ACKNOWLEDGMENTS

This manual represents the efforts of many people over a period of almost 18 years. Through the years, many states and other agencies have furnished reports, slides, and other information that has been incorporated into the course. Their assistance, and the assistance of many industry organizations who have also provided information, is greatly appreciated. These contributions have gone a long way in helping to keep the course comprehensive and up to date.

In the course of the development of this sixth edition, many people helped to revise the course's modules or, in some cases, rewrite them completely. The contributions of these individuals are gratefully acknowledged:

David G. Peshkin	Jon A. Epps
Kurt D. Smith	Eric D. Moody
Monty Wade	Todd V. Scholz

Reviewers who helped to evaluate the final draft include R. Gary Hicks and Carl L. Monismith. The update to the graphical materials was undertaken by Eric Henderson, and technical and editing support were tirelessly and cheerfully provided by Carol Chiappetta, Robin Patroni, and Laurie McCutcheon.

The Pavement's Division of the FHWA also provided invaluable assistance, from providing technical resources, slides, and advice, to performing technical reviews, usually with a very short lead time. This effort was spearheaded by Jason Harrington, and included help from Lee Gallivan, Roger Larson, Jim Sorenson, Rich Zamora, Mark Zitzka, and among others. Their input (and patience), along with the project leadership of Larry Jones, Pat Lees, and Pete Parsons of the NHI, is gratefully acknowledged.

Finally, a special acknowledgment is extended to those individuals who have contributed to the past editions of this notebook: Ernest Barenberg, Sam Carpenter, Michael Darter, Barry Dempsey, Bob Elliott, Lynn Evans, Sue Frazier-Morrow, Cynthia Good Mojab, Kathleen Hall, Moreland Herrin, Donald Janssen, Amy Mueller, Tom Nelson, Emmanuel Owusu-Antwi, Arti Patel, Elias Rmeili, Ronald Roman, Kelly Smith, Mark Snyder, Roger Smith, Tom Van Dam, Tom Wilson, Tom Yu, and Kathryn Zimmerman. This current edition only builds upon their significant contributions to previous efforts.

Stephen B. Seeds
Newton C. Jackson
John Ziegler

Nichols Consulting Engineers, Chtd.
January 1998

PREFACE

Development of the first version of the *Techniques for Pavement Rehabilitation* training course began in 1980. That original work along with work on four subsequent versions of the course was carried out by ERES Consultants (under the sponsorship of the Federal Highway Administration, National Highway Institute). Development of the latest version (Sixth Edition) of the course was carried out by Nichols Consulting Engineers (also under the sponsorship of the FHWA/NHI).

Over the years since its original development, the course materials have been improved to better address the needs of highway agencies on their specific maintenance and rehabilitation practices. Some of these improvements have included enhancements to the assortment of design and construction practices for the various existing pavement rehabilitation techniques. Other improvements have been aimed at adding new or more innovative techniques for existing pavement rehabilitation design and construction (e.g., dowel retrofit, fast track paving, hot in-place recycling, etc.) while still others (this latest version, in particular) have been targeted at making the process of evaluating the existing pavement structure more of a science.

Because of the maturity of the U.S. highway network, the demand for construction expenditures has switched dramatically over the years from new pavement construction to maintenance and rehabilitation. The number of times this course has been taught is approaching 200 with presentations in almost every State in the country, as well as numerous countries around the world.

The primary goals of this sixth edition of the course were to 1) once again update the recommended rehabilitation technology for both rigid and flexible pavements (especially for practices involving hot-mix asphalt), 2) reorganize the Reference Manual and course materials to deal with flexible and rigid pavement rehabilitation techniques separately and in logical sequence of minimum to maximum (most costly) treatments and 3) provide more emphasis on the understanding and need for a thorough project-level evaluation of the existing pavement. In addition, the course material were converted entirely to metric units. Activities associated with achieving these goals were carried out by a team of engineers with extensive experience in modern pavement evaluation and rehabilitation practices. The process involved a thorough search of the literature and contacts to many individuals in various States and private industry.

Although this effort is finally completed, identifying and implementing cost-effective means of maintaining and rehabilitating pavements is an on-going effort. Even as this manual goes to press, there are new and emerging practices and findings that are coming out of current research that must wait for the seventh edition. Nonetheless, this manual should be regarded as an accurate, informative, and up-to-date source of information on pavement evaluation procedures and rehabilitation techniques.

Any questions or comments on this manual should be directed to either:

National Highway Institute
Federal Highway Administration
901 N. Stuart Street, Suite 300
Arlington, Virginia 22203
(703) 235-0500
(703) 235-0593 *(fax)*

Nichols Consulting Engineers, Chtd.
1885 S. Arlington Ave., Suite 111
Reno, Nevada 89509
(702) 329-4955
(702) 329-5098 *(fax)*

Stephen B. Seeds, Newton C. Jackson and John Ziegler, Nichols Consulting Engineers, Chtd.
David G. Peshkin, Kurt D. Smith and Monty Wade, Applied Pavement Technology

January 1998

TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd)

BLOCK 5 SELECTION OF PREFERRED REHABILITATION ALTERNATIVES

LIST OF FIGURES

LIST OF FIGURES (cont'd)

LIST OF FIGURES (cont'd)

LIST OF FIGURES (cont'd)

LIST OF FIGURES (cont'd)

LIST OF FIGURES (cont'd)

LIST OF FIGURES (cont'd)

Figure **Page**

LIST OF FIGURES (cont'd)

LIST OF TABLES

LIST OF TABLES (cont'd)

LIST OF TABLES (cont'd)

BLOCK 1

INTRODUCTION

Few would argue with the statement that our roads and bridges are among the nation's most valuable assets... transportation (all types) comprises 18 percent of the Gross National Product, employs one-tenth of the work force, and accounts for 15 percent to 30 percent of the cost of agricultural products... the total replacement value of the nation's roads and bridges is estimated to be between $1 trillion and $3 trillion and... any measure that could improve their performance and durability by even one percent would result in savings of billions of dollars.[(1)]

The above statement, from a recent publication on implementing quality in the transportation field, not only stresses the importance of the highway infrastructure on the nation's economy, but also underscores the tremendous cost savings that could result if more effective methods could be employed to improve the performance and durability of the highway system. This refers to more effective methods employed in all phases of a highway project, including its design, construction, maintenance, and rehabilitation.

With 99 percent of the interstate highway system constructed, the biggest emphasis in recent years has been on the maintenance and rehabilitation of existing highway facilities. This trend is expected to continue, as fewer entirely new facilities will be constructed. Rather, existing highway facilities will continue to see an on-going cycle of pavement rehabilitation, consisting of restoration, resurfacing, recycling, and reconstruction for many years to come.

With most highway programs emphasizing pavement rehabilitation, it is important that effective pavement rehabilitation strategies be employed so that agencies can maintain—and perhaps improve—the performance of the highways. Through the use of effective rehabilitation strategies, highway facilities will provide higher levels of service for longer periods of time, with resultant savings to both the agencies and the users.

This training course is offered as a tool to assist pavement engineers in developing the most cost-effective and reliable rehabilitation alternatives for their highway pavements. The primary objective of the first block is to introduce the participant to this training course and to the 4R program. This block contains background information that will familiarize the participant with the general content of the course and provide information about the nontechnical side of the 4R program (such as legislation and funding). It also provides information on other closely related NHI courses. Subsequent blocks delve more deeply into specific pavement evaluation and rehabilitation methods.

---

(1) Afferton, K. C., J. Friedenrich, and R. M. Weed, "Managing Quality: Time for a National Policy," Transportation Research Record 1340, Transportation Research Board, 1992.

MODULE 1-1

INTRODUCTION AND COURSE OBJECTIVES

The first task for transportation is to maintain the assets that we have. If roads and bridges, subways, ports and waterways, and other transportation facilities are not kept in sound condition, they cannot support the level of service they are designed to handle. That means that the performance of the system declines: safe operating speeds drop, travel times rise, accidents increase and add further delays.... The longer that maintenance is deferred, the higher the eventual cost of restoration will be.... Over the long term, essential transportation facilities must be maintained on a continuing, timely basis.[1]

1. INTRODUCTION

It is conservatively predicted that travel on our highways is expected to increase 65 percent from 1991 to the year 2009.[2] How well the nation's existing highways and bridges perform in response to that increase in travel is a function of their current condition and the magnitude of capital and maintenance expenditures. In particular, adequate future highway service will be determined by how closely capital requirements and capital expenditures match.

A compilation of pavement condition data (as of 1995) for the nation's interstate highways and major arterial roadways is presented in figure 1-1.1.[3] Examination of this figure indicates that, as of 1995, roughly two-thirds of the nation's highway network (67.5 percent of the Interstate System, 66.1 percent of the other arterials) was in fair to very good condition. Although this seems to be an acceptable overall condition at first glance, it should be viewed in the context that an overall better and stable condition existed during the late 1980s when approximately four-fifths of the nation's highway network (76.2 percent of the Interstate System, 85.1 percent the other arterials) was in fair to good condition.[4] Passage of the original Intermodel Surface Transportation Efficiency Act of 1991 (ISTEA) and, more recently, the National Highway System Designation Act of 1995 provide funding to help re-establish a better overall condition for the highway network; however, more extensive use of improved design, management and construction practices will be essential to maximizing the return on these investments.

The abilities and engineering knowledge required to maintain and rehabilitate these existing pavements are much different than those that were required to originally design and build them, and this shift places new responsibilities on pavement engineers. As part of this shift in emphasis from design projects to rehabilitation projects, in the late 1970s the Federal Highway Administration (FHWA) recognized a critical need for technical training in the rehabilitation (resurfacing, restoration, and recycling) of transportation facilities. This manual and training course were developed to help meet that need for technology transfer and training. Together they provide the most up-to-date information available to assist highway maintenance staff, pavement design staff, construction staff, and policy makers to select and implement the most cost-effective pavement rehabilitation options available for their pavements. 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 need for this type of information has grown tremendously. Since 1980, this course has been presented over 160 times, in almost every State and to over 5,000 practicing engineers and technicians.

Table 1-3.
Highway Pavement Conditions: 1994 and 1995
(In percent)

Type of Road	Year	Poor	Mediocre	Fair	Good	Very Good	Total Miles Reported
Urban							
Interstates	1994	13.0	29.9	24.2	26.7	6.2	12,338
	1995	10.4	26.8	23.8	27.5	11.4	12,307
Other freeways and expressways	1994	5.3	12.7	58.1	20.9	2.9	7,618
	1995	4.8	9.8	54.7	20.4	10.3	7,804
Other principal arterials	1994	12.5	16.3	50.8	16.6	3.8	38,598
	1995	12.4	14.7	47.2	15.9	9.7	41,444
Rural							
Interstates	1994	6.5	26.5	23.9	33.2	9.9	31,502
	1995	6.3	20.7	22.3	36.9	13.9	31,254
Other principal arterials	1994	2.4	8.2	57.4	26.6	5.4	89,506
	1995	4.4	7.6	51.1	27.9	9.0	89,265
Minor arterials	1994	3.5	10.5	57.9	23.6	4.5	124,877
	1995	3.7	9.0	54.7	23.9	8.7	121,443

KEY:
Poor = needs immediate improvement.
Mediocre = needs improvement in the near future to preserve usability.
Fair = will be likely to need improvement in the near future, but depends on traffic use.
Good = in decent condition; will not require improvement in the near future.
Very good = new or almost new pavement; will not require improvement for some time.

NOTE: Interstates are held to a higher standard than other roads, because of higher volume and speed.
SOURCE: U.S. Department of Transportation, Federal Highway Administration. 1995 and 1996. *Highway Statistics.* Washington, DC. Table HM-64.

Figure 1-1.1. Condition of interstate and other arterial - 1995.[3]

Module 1-1

The course notebook has undergone four major revisions (1982, 1984, 1987, 1993, and 1997) to keep up with the rapidly expanding knowledge in pavement rehabilitation.

History of 3R/4R Legislation and Guideline Development

Prior to 1976, Federal-Aid Interstate funds could be used only for the initial construction of the system. All other non-maintenance work on the Interstate System was funded with Federal-Aid Primary or State funds. On the primary, secondary, and urban systems, Federal-Aid funds were used for a much broader range of activities including reconstruction.

The Federal-Aid Highway Act of 1976 established the Interstate 3R program, which placed emphasis on the use of Federal funds for restoration, resurfacing, and recycling. This redefinition of construction was aimed toward extending funding to lengthen the useful life and improve the safety of old pavements without costly reconstruction of vertical and horizontal alignments.

The Federal-Aid Highway Act of 1978 required that 20 percent of each State's primary, secondary, and urban Federal-Aid funds be spent on 3R projects. The Federal-Aid Highway Act of 1981 added the fourth R, reconstruction, so that existing facilities could be eligible for Federal funding.

The Surface Transportation Assistance Act of 1982 increased the Federal gas tax from 4¢/gal to 9¢/gal (1.1¢/L to 2.4¢/L). The Act also increased the amount of 4R money to be spent on primary, secondary, and urban routes from 20 percent to 40 percent. The Surface Transportation and Uniform Relocation Assistance Act of 1987 continued the funding of the 4R programs.

The Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) provided total funding of about $155 billion over a 6-year period (fiscal 1992-1997). This Act reclassifies the four Federal-Aid systems (Interstate, Primary, Secondary and Urban) into two Federal-Aid systems: the National Highway System (NHS) and the Non-NHS. Although the Interstate System is a part of the NHS, it retains its own identity and will receive separate funding. In addition, some of the program funds are distributed using procedures that are significantly different from the formulas of the past. Program eligibilities and requirements are also revised and States and local governments have increased flexibility in determining transportation solutions. The National Highway System Designation Act of 1995 formally launched the United States on its postinterstate mission and continued the funding initiated originally under ISTEA.

In June 1998, Congress passed the Transportation Equity Act for the 21st Century (TEA-21) to reauthorize 1991 ISTEA's transportation programs[6]. This program provides funding for improvements to rural and urban roads that are part of the NHS, including the Interstate System and designated connections to major intermodal terminals. Under certain circumstances, NHS funds may also be used to fund transit improvements in NHS corridors.

The NHS makes up 4.4 percent of the nation's total public roads, but carries over 42 percent of the travel.[3] Roughly 30 percent of the NHS consists of the interstate highway system. Table 1-1.1 shows the breakdown of mileage within the NHS (as of 1995) while figure 1-1.2[3] depicts its overall condition. (Again, the data indicate that the small key portion of the nation's highways that experience the most traffic are in need of improvement).

Availability of Funds

In the past, funds for 4R projects from either Federal or State sources have been very limited. Although the 1991 ISTEA bill authorized $121 billion over 6 years (approximately $20.2 billion per year) for surface transportation improvements, when compared to the capital needs, the capital commitments did not match the capital requirements.

However, the recent authorization of TEA-21 has reversed this trend. Due to the passage of TEA-21, funding is not available for surface transportation improvements. The critical element here is to use these funds in the most cost-effective manner possible. Agencies must be cautious in how they distribute these funds, and can do so by providing training to engineers in the latest pavement rehabilitation technology. This will help to ensure that funds are being spent wisely and efficiently. The 1998 TEA-21 bill authorizes the following:

NATIONAL HIGHWAY SYSTEM [7]							
Year	1997	1998	1999	2000	2001	2002	2003
Authorization	$3,600M	$4,112M	$4,749M	$4,793M	$4,888M	$4,968M	$5,061M

*Authorizations shown here will be augmented by a portion of Minimum Guarantee funds.

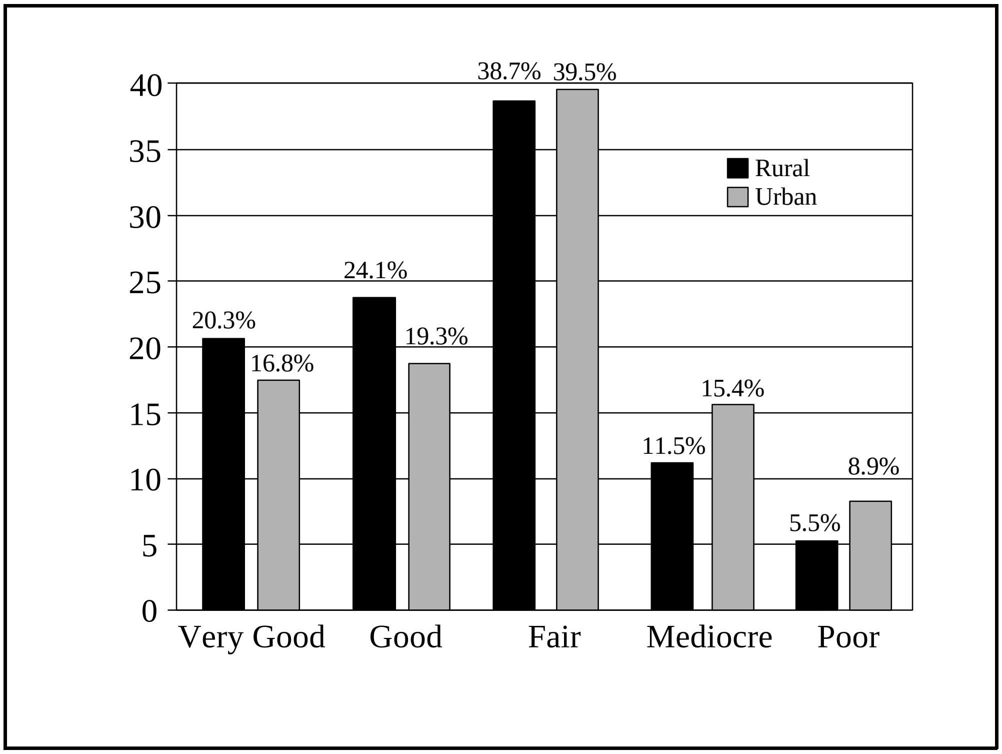
Federal funds are available for matching State and local funds to construct 4R projects. The Federal contribution often amounts to about 80 to 90 percent of the funding for the entire project, with the remainder coming from State and local funds. The formula for distribution of funds is based on each State's share of total national funding with appropriate adjustments. The participant should check with the State highway agency and FHWA division office as to the procedure that must be followed to obtain approval for Federal 4R funding.

2. COURSE OBJECTIVES

This training course is intended to assist pavement engineers in developing the most reliable and cost-effective rehabilitation alternatives for both flexible and rigid pavements. It addresses the problem of rehabilitation for both hot-mix asphalt and portland cement concrete pavements in a logical sequence involving existing pavement structural evaluation and condition assessment, distress mechanisms, needs assessment, assignment of feasible alternatives (from four categories of reconstruction, restoration, recycling, and resurfacing), overall design, selection of preferred alternatives (based on life-cycle costs and non-monetary factors) and construction.

Table 1-1.1. National Highway System mileage - 1995.[5]

NHS Mileage			
	Rural	**Urban**	**Total**
Interstate	32,580	13,164	45,744
Other NHS	97,948	52,796	150,744
Total NHS	130,528	65,960	196,488
NHS Percent of Total Mileage			
Interstate	0.8	0.3	1.2
Other NHS	2.5	0.7	3.2
Total NHS	3.3	1.0	4.3
NHS Travel (millions)			
	Rural	**Urban**	**Total**
Interstate	222,382	341,528	564,910
Other NHS	215,567	370,338	585,905
Total NHS	438,949	711,866	1,150,815
NHS Percent of Total Travel			
Interstate	9.1	13.9	22.9
Other NHS	7.9	11.7	19.5
Total NHS	16.9	25.5	42.5



The descriptive words used in the charts can be defined as follows:

Very Good -- New or almost new pavement; will not require improvement for some time.
Good -- In decent condition; will not require improvement in the near future.
Fair -- Will likely need improvement in the near future, but depends on traffic use.
Mediocre -- Needs near-term improvement to preserve useability.
Poor -- Needs immediate improvement to restore serviceability.

Figure 1-1.2. Condition of National Highway System - 1995.[3]

Upon completion of the course, the participants should be able to:

1. Describe typical performance of conventional pavements in terms of key types of deterioration.
2. Recognize common pavement distress types, identify their causes (mechanisms) and be familiar with procedures for project field surveys, including coring, lab testing and non-destructive deflection testing.
3. Describe design considerations and processes for pavement rehabilitation.
4. Recognize the principles and the importance of proper preparation of the existing pavement for resurfacing or restoration, including pavement recycling.
5. Develop, evaluate and select the most cost-effective 4R alternative for a given project.
6. Identify many types of rehabilitation that have worked well in the United States.

Although any individual involved with the pavement rehabilitation will benefit from this course, it is primarily intended for roadway design, construction and maintenance engineers who are responsible for developing and selecting the agency’s pavement resurfacing, restoration, recycling, and reconstruction alternatives.

3. COURSE ORGANIZATION

The overall objective of this training course is to develop a familiarity with the technical engineering concepts and information that are needed to design and construct resurfaced, restored, and recycled pavement projects. This information is essentially the rehabilitation process, which is summed up in figure 1-1.3.

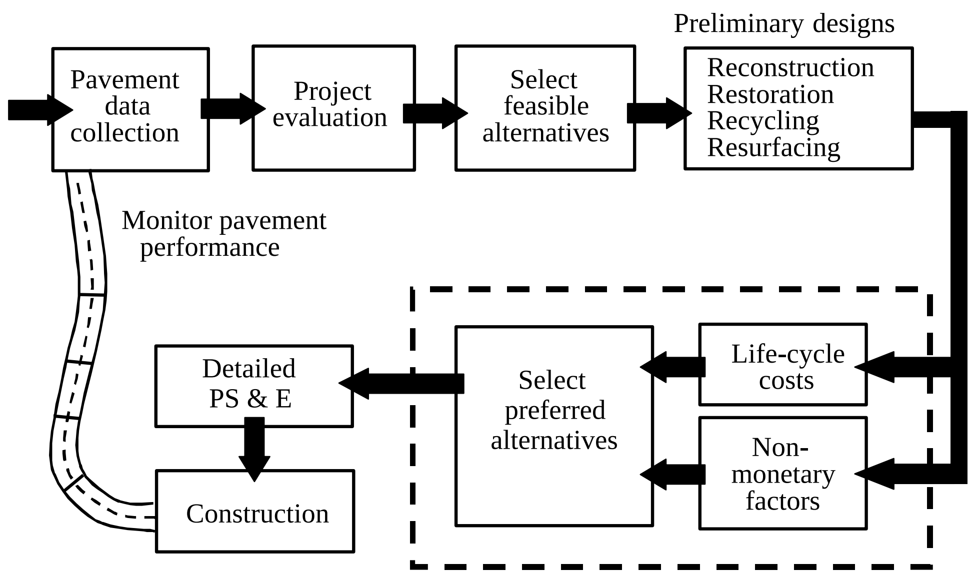


Figure 1-1.3. Steps in the pavement rehabilitation process.

In order to meet the overall course objectives, a large number of subjects are included in this course. Because of their length and complexity, not all of these technical subjects can be covered within the course's three-and-one-half-day time period. Therefore, the course presentation focuses on the most important major technical concepts and information that should be considered in the rehabilitation design process. Many more details and bibliographical materials are included in this notebook, which should serve as a lasting source of information to the participant.

In order to facilitate the presentation of the material and maintain the flexibility to present the material at different levels of interest and over different time periods, this course is divided into seven blocks, or major topics of instruction. The primary objective of this first block is to provide an introduction to this training course and to the 4R program. This block contains two modules of background information of interest to the participant. A brief description of each of the remaining blocks follows:

- **BLOCK 2: PROJECT SURVEY AND EVALUATION**—This block deals with the concepts and techniques of field surveys and material evaluations that must be conducted and the resulting information that should be evaluated to make it possible to identify the key distress mechanisms and determine/design the most cost-effective 4R alternative. It includes modules on:

2-1 Pavement Types
2-2 Condition Data Collection and Processing
2-3 Non-Destructive Data Collection and Interpretation
2-4 Laboratory Materials Characterization
2-5 Drainage Survey and Evaluation
2-6 Traffic Loading Evaluation
2-7 Overall Project Evaluation

- **BLOCK 3: FLEXIBLE PAVEMENT REHABILITATION TECHNIQUES**—This block provides detailed information on all rehabilitation activities under the 3R category (restoration, recycling, and resurfacing) related to existing flexible pavements. It also includes a module to help identify which alternatives are the most feasible. The modules include:

3-1 Asphalt Concrete Pavement Mixture Overview
3-2 Joint and Crack Sealing
3-3 Patching With Bituminous Mixtures
3-4 Cold Milling
3-5 Surface Rehabilitation Techniques
3-6 Recycling Overview
3-7 Hot In-Place Recycling
3-8 Cold In-Place Recycling
3-9 Hot Central Plant Recycling
3-10 Hot-Mix Asphalt Overlays
3-11 Identification of Feasible Alternatives

- **BLOCK 4: RIGID PAVEMENT REHABILITATION TECHNIQUES**—This block provides detailed information on all rehabilitation activities under the 3R category (restoration, recycling, and resurfacing) related to existing rigid pavements. It relies on some general concepts developed under block 3 and, like block 3, it concludes with a module to help identify which alternatives are the most feasible. These modules include:

4-1 PCC Pavement Overview
4-2 Joint Sealing for PCC Pavement
4-3 Pressure Relief Joints
4-4 Partial Depth Repairs
4-5 Full-Depth Repairs of PCC Pavements
4-6 Accelerated Rigid Paving Techniques
4-7 Slab Stabilization and Slab Jacking
4-8 Diamond-Grinding and Grooving
4-9 Load Transfer Restoration of PCC Pavements
4-10 Shoulder Rehabilitation considerations for PCC Pavements
4-11 Retrofitted Edge Drains
4-12 Recycling Concrete Pavements
4-13 PCC Overlays
4-14 Hot-Mix Asphalt Concrete Overlays for PCC Pavements
4-15 Identification of Feasible PCC Pavement Rehabilitation Alternatives

- **BLOCK 5: SELECTION OF THE PREFERRED ALTERNATIVE**—This block briefly covers non-pavement-related factors that have a bearing on the design of 4R projects. The factors discussed are: available funds, environmental concerns, right-of-way, safety, geometric design, and traffic control during construction. This block also deals with the development of cost-effective alternative 4R designs and the factors and procedures that should be considered in the selection of the preferred action.

Generally, each module begins with a list of learning objectives for the topic and concludes with a summary that provides a means for the participants to determine if they have learned the specific skills established for the module. Each module also contains a bibliography that provides a list of key references for those who wish to study the topics covered in the module in more depth. The training program is flexible and can be customized to different backgrounds, needs, or interests.

4. OTHER RELATED NHI TRAINING COURSES

Between the classroom presentations and this manual, a broad range of topics are covered in this course. Many of the topics that are introduced could themselves be separate training courses, and in fact there are several NHI training courses that overlap the content of this course and should be of interest to those seeking more information on these topics. These include courses on pavement deflection analysis, subsurface drainage design, pavement overlay design, and pavement analysis. A summary of the course content for each of these is presented below: additional information is available in the current version of the National Highway Institute's course catalog or by contacting the NHI at (703) 285-2776.

- *Pavement Deflection Analysis* (No. 13127), a 3-day course on backcalculation using the falling-weight deflectometer, provides extensive information on the use of the FWD to evaluate pavement condition. The material addresses both the testing program and the backcalculation and interpretation of the data, and should be of interest to those who use or are interested in using the FWD to collect and evaluate data on pavement condition at the project level.

- *Pavement Subsurface Drainage Design* (No. 13126) is a detailed 3-day course on the design of pavement drainage systems for both new design and rehabilitation projects. The course materials, developed in 1996, includes information on appropriate retrofitted subdrainage designs that are appropriate for both flexible and rigid pavements. It also addresses subdrainage requirements in the pavement overlay process.

- *AASHTO Pavement Overlay Design* (No. 13129) is a 3-day course on the structural design of all types of overlays for both flexible and rigid pavements, based on the AASHTO design procedures, With its detailed approach to pavement assessment and the selection of appropriate preconstruction repairs, this course's content will be of interest to those who seek more information on pavement rehabilitation with overlays.

- *Pavement Analysis and Design Checks* (No. 13130) presents both a methodology and the analytical tools needed to evaluate pavement designs and determine their adequacy. The content of this course requires a background in pavement design and analysis, but is applicable to those seeking more tools to evaluate the condition of their pavements or the adequacy of their pavement designs.

5. SUMMARY

Background information, historical information, and funding for 4R projects are discussed in this module. The capital requirements to maintain and improve the existing highway system are greater than capital commitments. Therefore, cost-effective rehabilitation techniques are of great interest to State highway agencies.

The use of Federal funds for the 3R program was established by the Federal-Aid Highway Act of 1976. Additional legislation through the years has established a comprehensive program for the restoration, resurfacing, recycling, and reconstruction of the nation's highways. The most recent legislation, ISTEA, establishes a National Highway System and provides more flexibility for State and local agencies in allocating funding. However, there are significant changes in the programming and distribution of funds, so agencies should consult with local FHWA division offices when contemplating Federal funding on a project.

An overview of the workbook and the training course are also presented in this module. This includes an review of the layout of the workbook and an overview of the format of the course. Because of the extensive information contained in this course, some “customization” is needed so that agencies are exposed to those topics of most interest to them.

6. REFERENCES

1. “Moving America, New Directions, New Opportunities: A Statement of National Transportation Policy Strategies for Action,” United States Department of Transportation, February 1990.

2. “1991 Status of the Nations Highways and Bridges: Condition, Performance and Capital Investment Requirements,” United States Department of Transportation, Federal Highway Administration, July 2, 1991.

3. “Our Nation’s Highways—Selected Facts and Figures,” FHWA-PL-95-028, Federal Highway Administration, 1995.

4. “Our Nation’s Highways— Selected Facts and Figures,” FHWA-PL-92-004, Federal Highway Administration, 1992.

5. “Condition and Performance,” 1997 Status of the Nation’s Surface Transportation System, Report to Congress, FHWA, Federal Highway Administration, 1997.

6. “Passage of Surface Transportation Legislation,” News Release, United States Department of Transportation, June 1998.

7. “TEA-21 - Transportation Equity Act for the 21^{st} Century,” Fact Sheet: National Highway System, United States Department of Transportation, July 10, 1998.

BLOCK 2

PROJECT SURVEY AND EVALUATION

Project surveys are performed to collect data on the existing condition of a pavement structure. Information collected on a particular project may include distress data, roughness data, surface friction values, deflection data, and drainage information. The data that collected can be used for a variety of purposes that generally fall into one of the following categories:

- Development and selection of rehabilitation alternatives.
- Prioritization of projects for rehabilitation action.
- Development of performance prediction models and curves.

This training course focuses on the first category, the development and selection of cost-effective rehabilitation techniques for a specific pavement section that has already been identified as needing rehabilitation. The other two topics are gear more towards network-level (as opposed to project-level) pavement management system (PMS) activities, which are not a part of this training course.

The selection and design of the appropriate rehabilitation techniques for a pavement requires consideration of many factors. For instance, the techniques must address the causes of distress, must provide for an acceptable level of service for a reasonable time, and should be cost-effective. Some factors that may be considered in this process include the existing distress, the overall pavement roughness, future maintenance requirements, existing and future traffic levels, climatic conditions, safety elements, and available funding levels.

Other information may also be required, depending upon the type of rehabilitation alternatives considered for a particular project. It is imperative that the engineer identify and obtain all information that is needed to develop the most reliable, cost-effective design for a given project.

In order to identify appropriate rehabilitation measures for a particular project, the performance of that pavement must first be quantified. Pavement performance may be divided into either structural performance or functional performance. Structural performance is an indication of how well a pavement is able to support traffic and environmental loadings, while functional performance is a measure of how well the pavement is performing its intended function of providing a smooth, safe ride to the user. In this block, procedures for the conduct of the different types of field surveys are described so that the pavement's structural and functional performance may be assessed. These results may be used to select feasible, cost-effective 4$ repair methods.

This block will familiarize the participants with field surveys needed for a pavement evaluation, including those related to distress, roughness, drainage, structural, traffic, and subgrade. The distress survey is always conducted first, and the other surveys are based upon the results obtained from the distress survey.

MODULE 2-1

PAVEMENT TYPES

1. INSTRUCTIONAL OBJECTIVES

This module provides background information on the role of each layer in a pavement structure and the key factors that affect pavement performance. It also discusses the different types of pavements and briefly describes their characteristics and typical performance trends. Recognition of basic characteristics, performance trends, and the distresses associated with each pavement type (including their underlying mechanisms) provides for a more accurate assessment of the causes of the pavement deterioration. This, in turn, assists in the identification of rehabilitation measures best suited to address the particular distresses. Upon successful completion of this module, the participant shall be able to accomplish the following:

1. Describe the role of each layer in the pavement structure.
2. Identify the key factors that affect pavement performance.
3. Identify the three major pavement classifications and the five specific pavement types they encompass.
4. Describe the characteristics of each pavement type.
5. Describe typical performance for each pavement type, as well as the mechanism that lead to their typical distress manifestations.

2. DEFINITIONS

Distress. In this module and throughout this course, this term will refer to some observable, measurable manifestation of poor pavement condition, e.g., cracking, rutting, faulting, etc.

Mechanism. This refers to the set of physical conditions and/or mechanical processes that lead to the development of distress. Distress mechanisms are important to understand if an appropriate rehabilitation treatment is to be selected.

3. INTRODUCTION

Because of their different inherent mechanisms for carrying and distributing load, pavements are generally classified into one of three categories: flexible, rigid, and composite. Flexible pavements either have a hot-mix asphalt (HMA) surface or a bituminous surface treatment (BST). All rigid pavements have portland cement concrete (PCC) as their surface layer, however, they differ significantly in terms of their reinforcement and joint design. The most common types of rigid pavements are jointed plain concrete pavements (JPCP), jointed reinforced concrete pavements (JRCP) and continuously reinforced concrete pavements (CRCP). Composite pavements basically refer to a combination of an HMA surface on a PCC slab. New pavements are sometimes constructed in a composite fashion, however, the term generally refers to pre-existing rigid pavements that have been overlaid with hot-mix asphalt. Each of these pavement types is unique in the way that it responds to traffic and environmental loadings. Consequently, it is important to understand these responses and the nature of their differences if an appropriate rehabilitation alternative is to be selected. This module (and overall course) will focus on flexible pavements consisting of an HMA surface layer, rigid pavements (all three basic types), and composite pavements where the surface layer is either a HMA or a rigid pavement overlay. Basic information on pavement design and performance is found in references 1, 2, and 3.

Before discussing each pavement type specifically, it is useful to develop a common understanding of the general role of each layer in the pavement structure and the key factors that affect pavement performance.

Role of Pavement and Soil Layers

The surface layer of a pavement exposed to vehicular traffic must be tough to resist distortion and provide a smooth riding surface. It must be waterproof and transversely sloped to shed surface water to the roadside, thus protecting the entire pavement structure and the subgrade soil from the detrimental effects of moisture. It must also resist wear caused by traffic an retain its anti-skid properties.

Although typically unbound, the base and subbase layers are considered structural elements of the pavement. In conjunction with the overlying surface, they distribute wheel loads over a relatively large area of the subgrade or foundation soil. A pavement "base" is defined as a support layer immediately underneath the wearing surface. The base is a deliberately constructed layer of greater strength than the natural soil. There can be more than one base course. The term "subbase" is generally used to describe the bottom-most base course. The natural soil on which the pavement is built is normally referred to as the "subgrade" or "roadbed" soil. The top six inches or so of the subgrade soil are usually compacted prior to placing the base or subbase.

The subgrade soil (although not considered a structural layer of the pavement) ultimately carries all traffic loads. Thus, the function of a pavement structure is to support a wheel load on the pavement surface and to spread or distribute the load to the subgrade soil without overtaxing its strength or that of various overlying pavement layers.

Figure 2-1.1 shows a wheel load being applied to a pavement surface (through the tire) at an approximately uniform vertical pressure. The pavement structure spreads the wheel load to the soil so that the maximum pressure on the subgrade soil is considerably less than the vertical pressure at the surface. Thus, when proper pavement materials are used and pavement layer thicknesses are adequate, the maximum pressure on the soil will be small enough to avoid overstress and permanent deformation (rutting).

To perform this function, bases and subbases must be built with adequate internal strength properties. Furthermore, they must retain these properties throughout the life of the pavement. (This is why good drainage and routine maintenance are extremely important.)

Unbound (unstabilized) aggregate bases and subbases are permeable pavement layers that will allow subsurface and/or surface moisture to enter the structure. This leads to wetter (weaker) materials and the possibility of soil particles infiltrating the base ad subbase layers, thereby reducing their long-term strength. If water enters the base/subbase layers and cannot drain away, severe hydrostatic pressures from wheel loads can develop. These pressures lead to undue stress on all pavement and soil layers and in flexible pavements, can result in alligator cracking, rutting and potholes. In rigid pavements, it can result in pumping, the development of voids beneath the slab and, of course, severe cracking and slab breakup.

Asphalt and other treated (stabilized) bases are not as affected by moisture as unbound aggregate bases. They are more stiff and have increased capacity for spreading loads and reducing the compression stress on the natural soil; thus, less total pavement thickness is required. However, some HMA mixes may be moisture sensitive and which over time can reduce their capacity for spreading loads, as designed.

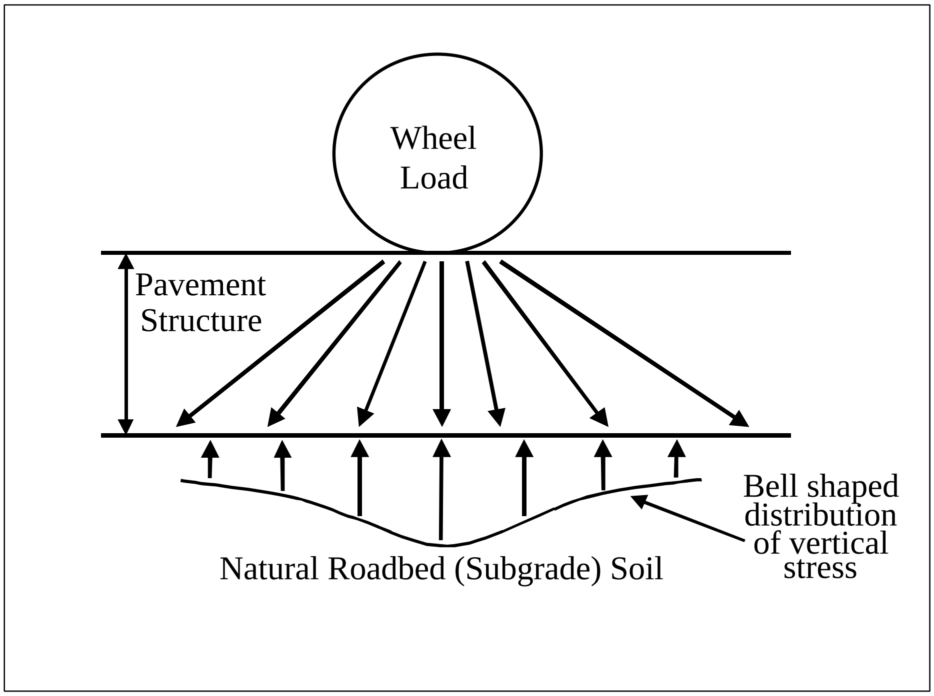


Figure 2-1.1. Distribution of Wheel Load to Subgrade Soil.

Factors Affecting Pavement Performance

A significant advance in highway engineering is the realization and demonstration that the problems of the structural design of pavements are similar to the problems of designing other complex engineering structures. When pavements were first introduced, the proper thickness was determined by guesswork, rule of thumb and/or opinion based on experience. Almost the same situation once prevailed in determining the dimensions of masonry arches and iron and steel structures. Those early techniques have long since yielded to more fundamental engineering analyses and, similarly, the structural design of pavements has evolved into more reliable engineering-based procedures. Research aimed at further refinements and a fully rational design procedure (such as the Long-Term Pavement Performance Program) is continuing.

The factors that affect pavement performance should be clearly understood when designing or evaluating pavements. For purposes of this discussion, these factors can be classified under the following seven categories:

- Traffic loadings.
- Subgrade soil support.
- Materials of construction.
- Structural characteristics.
- Construction and maintenance variation.
- Moisture.
- Maintenance and rehabilitation programs.

Following are more detailed discussions of the significance of each of these factors:

Traffic Loadings

This refers to the cumulative wheel loadings that are sustained by a pavement over the course of its life. It is a very important factor in that, without traffic, there would be little or no deterioration of the pavement. Thus, poor traffic analysis means that the useful life of the pavement may be significantly reduced if traffic is underestimated, or significantly "over designed" if traffic is overestimated.

The determination of a uniform value for traffic loading depends on several factors:

- Average daily traffic, ADT (initial number of vehicles per day).
- Future projection (annual growth rate by vehicle type).
- Distribution of trucks.
- Lane distribution (percent of trucks in design lane).
- Directional distribution (percent of trucks in design direction).
- Truck factors or load equivalency factors (means to convert the distribution of vehicle loads into an equivalent number of load applications that can be used for design).

All of these are combined with the design period (usually 10 years to 30 years) to derive the 80-kN equivalent single axle load (ESAL) applications that will be sustained by the "design lane" (i.e., that lane of the pavement that carries the most ESAL applications and for which the full-width pavement structure or overlay will be designed). More information on traffic loading is provided in module 2-5.

Subgrade Soil Support

All pavement structures must rely upon the natural subgrade soil for support. The function of pavement is to distribute wheel loads as much as possible, but the natural soil must ultimately carry all the load. Because of this, it is easy to understand why weak soils require thicker pavements than strong soils to carry the same traffic loadings. (Thicker pavement distribute load over a larger area, thereby minimizing the stress felt by the soil.)

From a pavement design and engineering standpoint, there are two general characteristics of the soil that are of interest, its classification and its strength. Soil classification is important since it gives the engineer a good idea of the gradation and constituents of the in-situ material. There are two systems which have generally been used to classify soils, the Unified System and the American Association of State Highway Officials (AASHO) System. For pavement design and analysis, the AASHO system is the most widely-used.

The second, and as indicated previously, most important soil characteristic is its strength. Measures of soil strength or resistance to deformation include California Bearing Ratio (CBR), stabilometer (R-value), soil support, unconfined compressive strength, stiffness coefficient, modulus of subgrade reaction and elastic (or resilient) modulus. Although many of these have been adequate in the past for strength characterization, the trend now is towards more fundamental engineering properties such as elastic (or resilient) modulus. These are generally more difficult to measure in the laboratory, but they are geared towards simulating the pavements behavior (response) under load and are better-suited for long-term pavement performance prediction. Furthermore, there are some nondestructive based "backcalculation" methods available now that can be used to determine their in-situ value without taking any samples. In addition, many States are beginning to use Dynamic Cone Penetrometers (DPC) to determine in-situ values.

If soil strength cannot be directly (or indirectly) measured (or if one strength measure is available but another desired), there are correlations available in the literature to make a reasonable estimate. It should be understood, however, that the use of such correlations does introduce increased uncertainty into the design process.

Materials of Construction

There are many types of materials that go into the construction of a pavement. The individual ingredients or constituents fall under one of six different categories:

- Asphalt (bitumen)
- Portland cement
- Aggregate
- Water
- Additives (lime, flyash, rubber, sulfur, superplasticizers, entraining agents, etc.)
- Steel (reinforcement and load transfer devices)

When various ingredients are appropriately combined, they produce mixes (hot-mix asphalt, portland cement concrete, stabilized bases/subbases, etc.) that ultimately make up the structural components of the pavement. Obviously, high quality materials, good mixing and construction practices, and quality control will help maximize the ultimate load-carrying capacity of the pavement.

Structural Characteristics

These refer to various physical characteristics of the pavement structure that have an impact on pavement performance. Structural characteristics include the number of layers and their thicknesses, lateral support, reinforcement type and percentage, joint spacing and geometry, and load transfer. They are properties that can be controlled during the design and construction process and provide another means through which the engineer can exercise some control over the long-term performance of the pavement.

Construction and Maintenance Variation

Another factor that affects pavement performance is the variation that occurs in construction and maintenance operations. Failure to obtain proper compaction, moisture conditions during construction, quality of paving materials, and as-built layer thicknesses all directly affect performance. During the AASHO Road Test, pavements were constructed under exceptionally stringent quality control procedures that probably have never been equalled. Even under these conditions the as-built thicknesses and quality of materials used, when evaluated after construction, varied surprisingly.

One of the best ways to address construction and maintenance variability is to develop and apply better specifications that encourage contractors to exercise better control over their operations. Performance-based and performance-related specifications are examples of the latest technology for achieving compliance, but warranty specifications and end-result specifications have also been used successfully.

Moisture

Moisture can have the most influential effect on pavement performance. Moisture can enter a pavement structure through cracks and holes in the surface, laterally through the subgrade soil, and from the underlying water table through capillary action. The presence of moisture in the soil and underlying layers of the pavement structure weakens those materials and thereby reduces their load-carrying capacity. Because of the way it will lubricate soil particles, it also increases the potential for slope failure on highway embankments. Free water also provides the medium through which fine-grained materials can be "pumped" from beneath rigid pavements leaving voids and non-uniform support. The presence of moisture in hot-mix asphalt surface layer can lead to a phenomenon known as stripping, which is the separation of asphalt from aggregate particles in the mix. Lastly, its presence in the soil in regions where freezing occurs can result in differential frost heave and thaw weakening. In addition, moisture changes in some clay subgrades cause volume changes and pavement distortion and roughness.

Because of the significant impacts of moisture, attention to drainage is provided throughout this document.

Maintenance and Rehabilitation Programs

The last major factor affecting pavement performance is the maintenance and rehabilitation program. The condition of a pavement is not constant; it deteriorates over time due to the factors previously discussed. Deterioration is also dependent on the levels and types of maintenance and rehabilitation that are conducted. Maintenance refers to any activity performed on the pavement that is intended to <u>preserve</u> its original service life or load-carrying capacity. Examples of maintenance activities include patching, crack or joint sealing, and seal coats. If these are performed on a routine basis and as the demand arises, the pavement should provide the desired service life (assuming that traffic and environment were adequately characterized). If these activities are ignored or deferred, the pavement will probably not last as long. On the other hand, if these activities are performed before the demand arises (preventive maintenance), the result will likely be both an extended service life and improved overall performance during the life. Many agencies are now finding more preventive maintenance activities to protect their pavement investments.

In contrast to maintenance, rehabilitation generally refers to an activity performed to extend the life or load-carrying capacity of the existing facility. Examples of rehabilitation activities include overlays, slab replacement, retrofitted load-transfer, grinding/milling, and undersealing/pressure grouting. Overlays are, by far, the most common means of rehabilitation. Some agencies rely on a policy or standard set of specifications for designing and building their overlays. This practice is usually acceptable for the lower functional classes (i.e., collectors, minor arterials, maybe even major arterials) but for high-volume facilities (interstate highways, State highways, principal arterials, freeways, etc.), the overlay should be designed using some rational overlay design procedure. An overlay design methodology which calls for a program of traffic and conditions surveys, non-destructive (deflection) testing and some laboratory testing is recommended, since it will maximize the benefit obtained from the overlay in a cost-effective manner. In general, the application of rational design procedures for overlays (and new pavements) will produce results that are much more certain and reliable than those derived from a policy.

To summarize, maintenance and rehabilitation are activities that are very important to the performance and overall level of service provided by an existing pavement facility. The cost of both are generally much less than the cost for reconstruction that would ultimately be required if they are ignored. Consequently, it is strongly recommended that they receive proper attention.

4. FLEXIBLE PAVEMENTS

Components

Flexible pavements consist of a bituminous surface course placed over a series of other structural paving layers. The flexible pavement design philosophy is twofold: to provide sufficient total pavement thickness above any given material (including the subgrade soil) to prevent permanent deformation, and to provide enough HMA surface, binder, and stabilized base thickness to limit the development of fatigue (alligator) cracking. This layered system distributes the load over the weaker materials in the lower layers of the pavement system, including the subgrade soil, as illustrated in figure 2-1.2. Materials that are commonly used in each layer include:

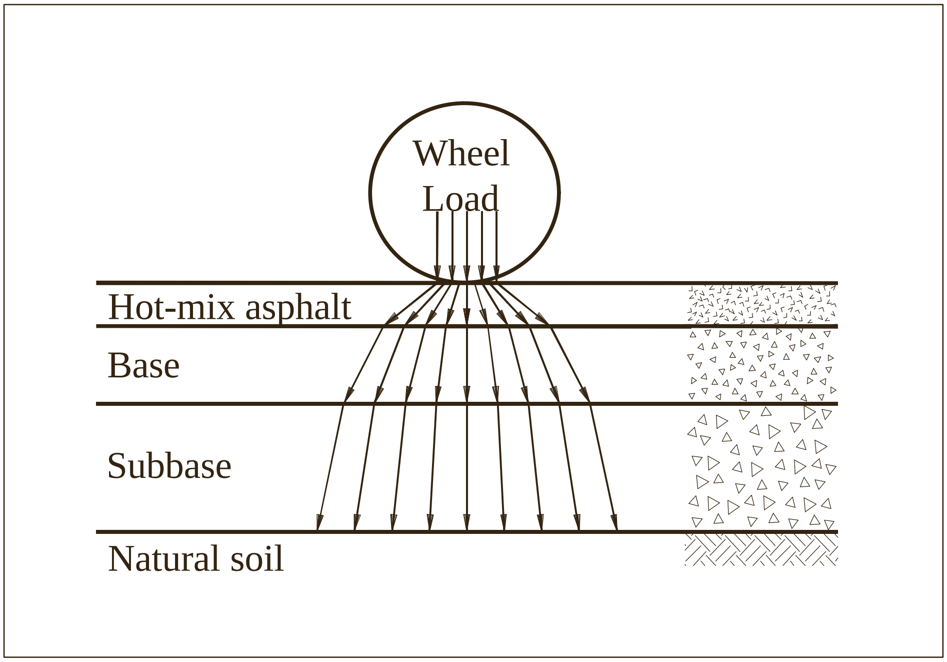


Figure 2-1.2. Distribution of wheel load in a flexible pavement.

Surface Layer	Base Layer	Subbase Layer
Hot-mix asphalt	Granular	None
Bituminous surface treatment	Stabilized (asphalt or cement)	Granular
		Stabilized (asphalt, cement, lime)

The specific types of materials used, their thicknesses, and their relative positions within the pavement structure have a great influence on the structural response of the pavement and, therefore, its performance.

Surface layers consisting of HMA are the most common type of pavement surface in the world. In fact, HMA is very effective in providing load-carrying capacity, resisting distortion, providing a smooth riding surface, minimizing the intrusion of moisture from the surface, resisting traffic wear and retaining its anti-skid properties. It is also comparatively economical and easy to construct. Bituminous surface treatments (BSTs) consist of one or two layers of a spray application of asphalt cement (or emulsion) followed by the imbedment of a crushed rock. BSTs are effective in many of these roles, however, they provide essentially no load-carrying capacity and are more prone to moisture infiltration than HMA surfaces. The best use of BSTs has been on pavements with lower traffic volumes.

Base layers (as previously discussed) generally consist of either granular or stabilized materials. Granular base courses are essentially unbound cohesion less layers of aggregate (or crushed rock) that are uniformly graded and compacted to optimum density. Another type of improved granular base layer is one in which the gradation is modified to permit improved subsurface drainage while maintaining the intended load distribution characteristics. These permeable bases are growing in popularity, especially in areas where subsurface moisture leads to problems with reduced layer strength and asphalt stripping.

A stabilized base course refers to a layer of select aggregates that is bound together by some type of bituminous material (i.e., asphalt cement, emulsin or cutback) or cementitious material (i.e., portland cement). The increased cost of the stabilization process is usually offset by the improved load distribution, structure capacity and moisture susceptibility characteristics.

Like base layers, subbase layers consist of either granular or stabilized materials. Because they are lower in the pavement structure where the wheel load stresses are significantly reduced, the overall quality requirements (strength, gradation, compaction, aggregate soundness, etc.) are not as strict. Both granular and stabilized subbase courses perform similar functions as their base course counterparts. However, stabilized subbase materials may be bound together with lime as well as either bituminous or cementitious materials.

Basic Distress Mechanisms

Four categories of pavement distress mechanisms have been identified for all pavement types:

- Load-related.
- Temperature-related.
- Moisture-related.
- Age-related.

The specific mechanisms vary depending on pavement type. For flexible pavements, the key distress mechanisms are discussed below.

Load-Related

Under the category of load-related distress mechanisms, flexible pavements are susceptible to two: fatigue and permanent deformation. Both of these can be described with the aid of figure 2-1.3.

Fatigue

The mechanism of fatigue refers to a progressive process whereby a bound layer in the flexible pavement structure undergoes so many repeated applications of stress (or strain) that it eventually cracks. No one application of stress is enough to exceed the strength of the bound material, yet the accumulation over time is enough to eventually wear out or fatigue the material. In figure 2-1.3, response 2 represents the maximum tensile strain at the bottom of the HMA surface layer resulting from the application of a single wheel load. The repeated application of this strain first causes the initiation of a crack at the bottom of the layer. Continued load repetitions ultimately causes the crack to propagate from the bottom to the surface. The photograph in figure 2-1.4 is one taken of a trench of an experimental section. Besides illustrating damage associated with frost heave, the photo depicts the various stages of fatigue crack progression (from the bottom to the surface). The photographs in figure 2-1.5 (a, b and c) illustrate the continued progression of fatigue cracking at the surface from a longitudinal hairline crack to severe, extensive alligator cracking in the wheel paths. The presence of fatigue cracking is an indication of the loss of structural (load-carrying) capacity in the pavement. It should be noted that fatigue cracking often develops at an almost exponential rate (see figure 2-1.6). Thus, if over one quarter of the pavement exhibits fatigue cracking, it is likely that the rest of the pavement is not far behind. This should have some bearing on the selection of an appropriate rehabilitation alternative.

It should also be noted that fatigue cracking does not always initiate in the HMA surface layer. If a stabilized base course exists. Its stiffness may be great enough such that tensile strains can be generated at the bottom of this layer and fatigue cracking may initiate there.

Permanent Deformation

Permanent deformation (or rutting) refers to a progressive process whereby the accumulation of small amounts of wheel load related permanent deformation in one or more layers ultimately leads to a significant depression of the pavement surface in its wheel paths (i.e., ruts). The main problem with wheel path rutting is that, during wet weather, it increases the likelihood of vehicle accidents associated with hydroplaning. Figure 2-1.7 (a and b) provides photos illustrating light/moderate and moderate/heavy rutting.

In figure 2-1.3, responses 1, 3, and 4 represent the vertical compressive stresses on the HMA surface, base and subgrade soil, respectively. Each of these vertical stresses can contribute to the permanent deformation of the layer(s) beneath them. If deformation exists only in the HMA surface layer, then any one or combination of the following could be the culprit:

- The HMA surface layer was overloaded.
- Loading was exerted during a hot period (when the HMA layer was “soft”).
- There was a problem with the stability of mix.
- There was a problem with the temperature susceptibility of the asphalt.

Figure 2-1.8 provides a photograph of an HMA surface with double-ruts from the dual tires. This is definitely a problem with the HMA (either mix design or construction).

If permanent deformation exists only in the base/subbase courses, then a different set of possible explanations exists:

- The HMA surface layer was too thin.
- The aggregate (or aggregate blend) in the base/subbase was unstable or inadequately designed.
- The layer was poorly constructed.
- The layer was exposed to excessive or prolonged moisture.

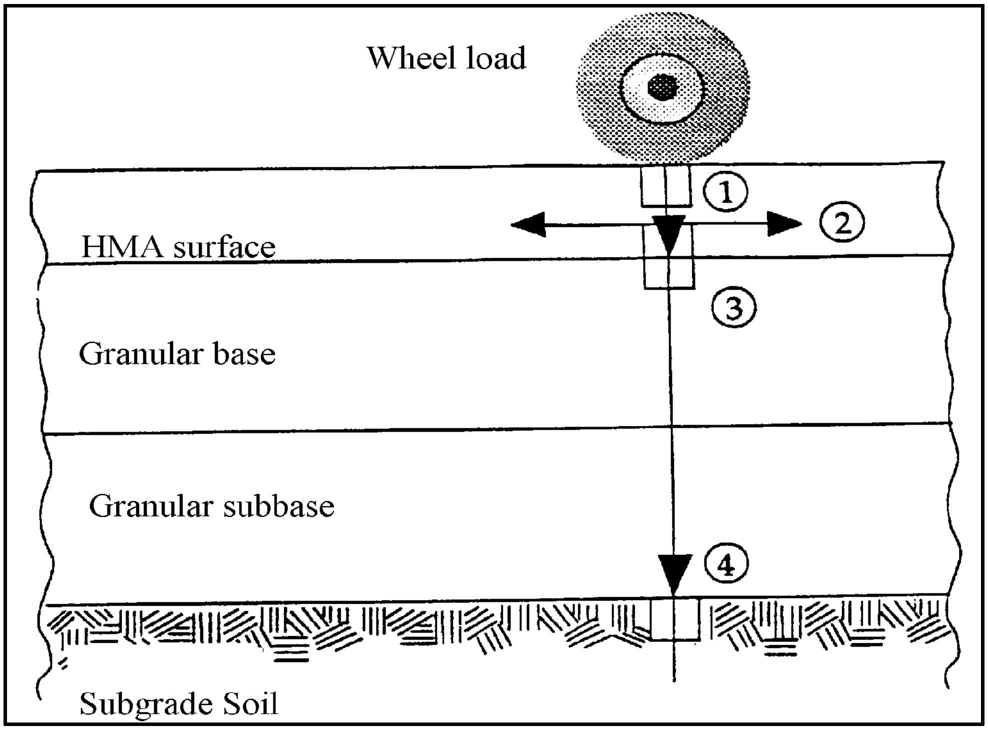


Figure 2-1.3. Critical stress / strain locations in asphalt concrete pavement with granular base.

Figure 2-1.4. AASHO road test section exhibiting fatigue and frost heave damage.

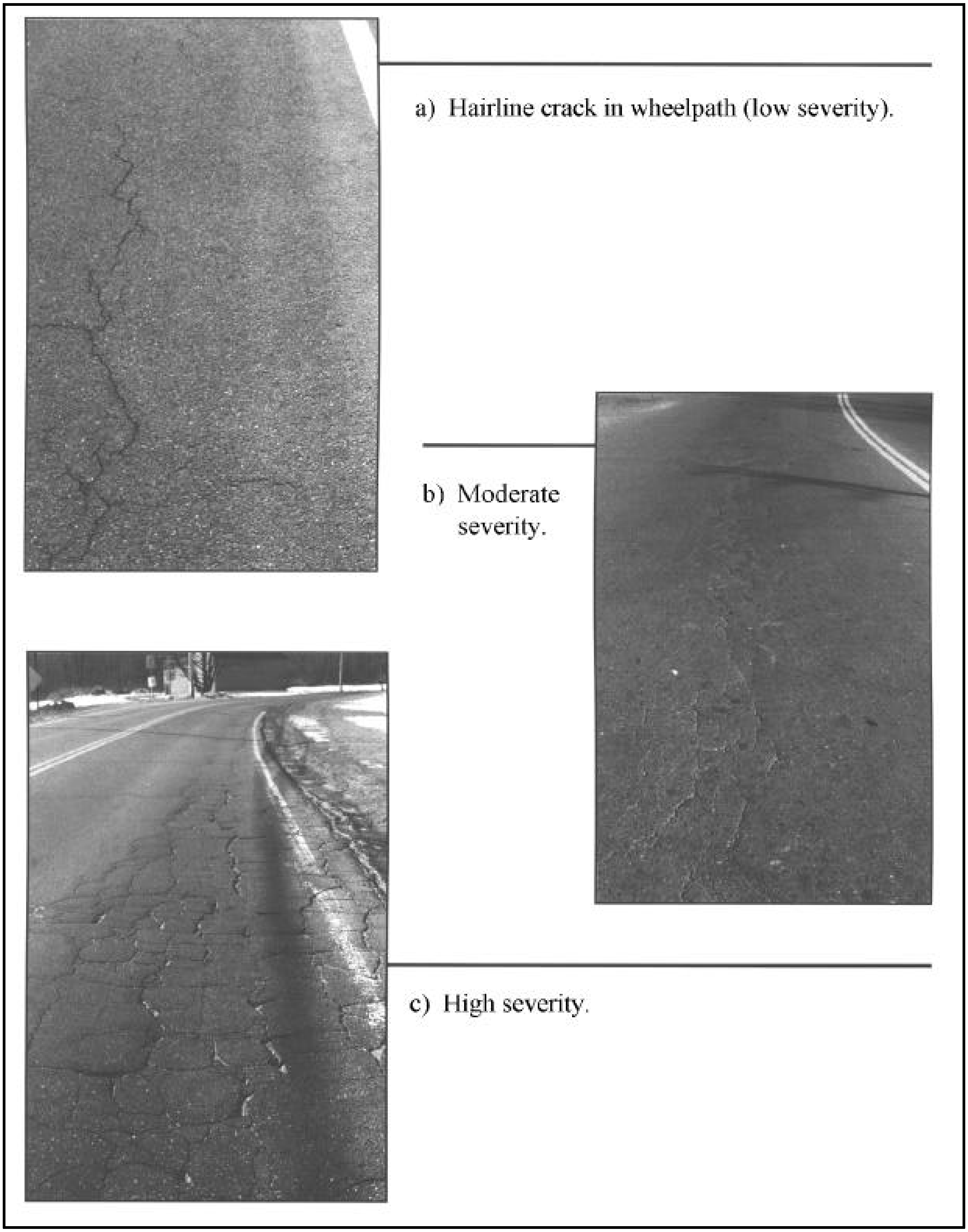


Figure 2-1.5. Typical progression of fatigue (alligator) cracking in HMA surface.[26]

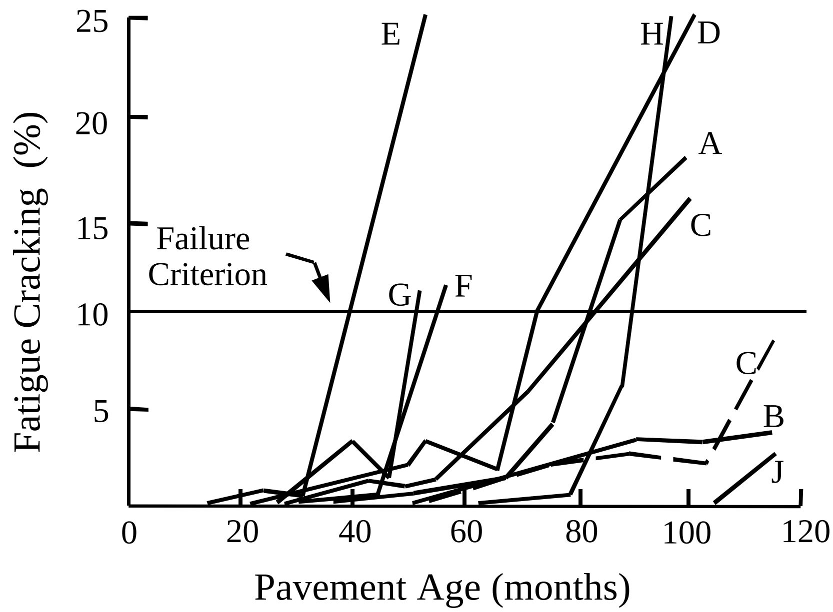


Figure 2-1.6. Rate of structural deterioration of several asphalt pavement sections in California (letters represent different pavement sections).[4]

If the total rutting at the surface exists in all layers down to the subgrade, then the problem would likely be attributed to:

- A pavement structure that is too thin for the applied loads.
- A naturally weak soil (or a soil in which high moisture contents exist routinely).

Temperature -Related

This section focuses on the distress mechanisms related to low-temperature conditions. High-temperature conditions increase the potential for permanent deformation in the HMA surface layer, as previously discussed.

Thermal Cracking

For flexible pavements with an HMA surface, thermal cracking is the primary distress concern related to temperature. It has also been referred to as low-temperature cracking, environmental cracking and transverse cracking. Figure 2-1.9 provides an example of a medium severity transverse crack. Its width is about 3 mm and is showing some signs of spalling.

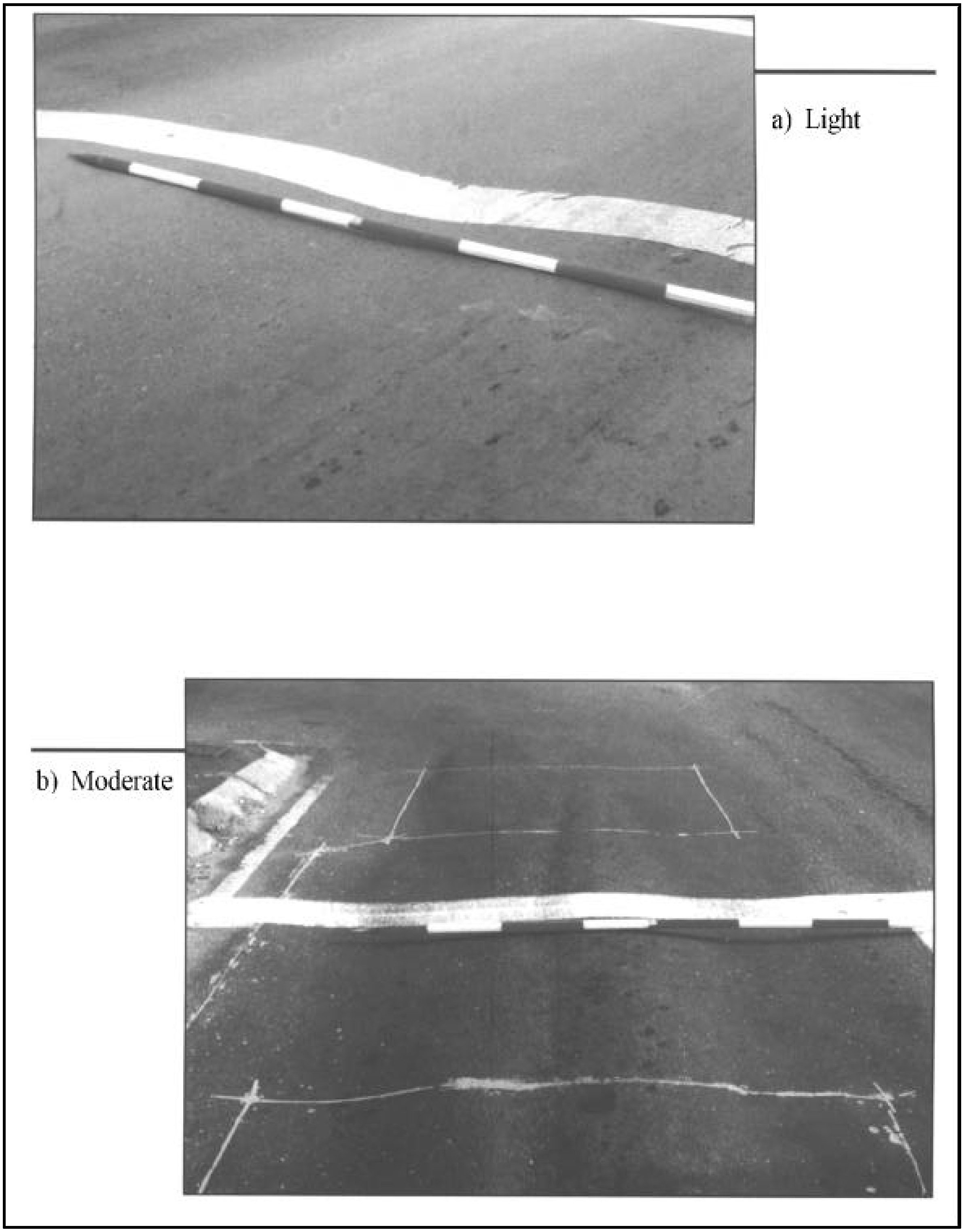


Figure 2-1.7. Photographs of typical permanent deformation (rutting) in HMA surface.[26]

Figure 2-1.8.   Example of HMA surface exhibiting double ruts in each wheelpath.

Thermal Cracking

For flexible pavements with an HMA surface, thermal cracking is the primary distress concern related to temperature. It has also been referred to as low-temperature cracking, environmental cracking and transverse cracking. Figure 2-1.9 provides an example of a medium severity transverse crack. Its width is about 3 mm and is showing some signs of spalling.

Figure 2-1.9.   Medium severity transverse crack in HMA surface.

Thermal cracking develops in HMA surface layers as the pavement undergoes one (or more) temperature-drop cycles. During a given daily temperature drop cycle, the HMA surface layer will naturally tend to contract (see figure 2-1.10), with greater temperature drops producing greater potential for contraction. Restraint to contraction is supplied by 1) friction on the bottom of the HMA surface and 2) the continuity of the HMA layer itself. The combination of contractive forces and restraint causes the tensile stress to build up to such a point that it can exceed the tensile strength of the HMA layer and the crack develops. Even if the stress does not exceed the strength during any one cycle, the transverse crack can come about as a result of the accumulated damaging effects of multiple temperature drop cycles.

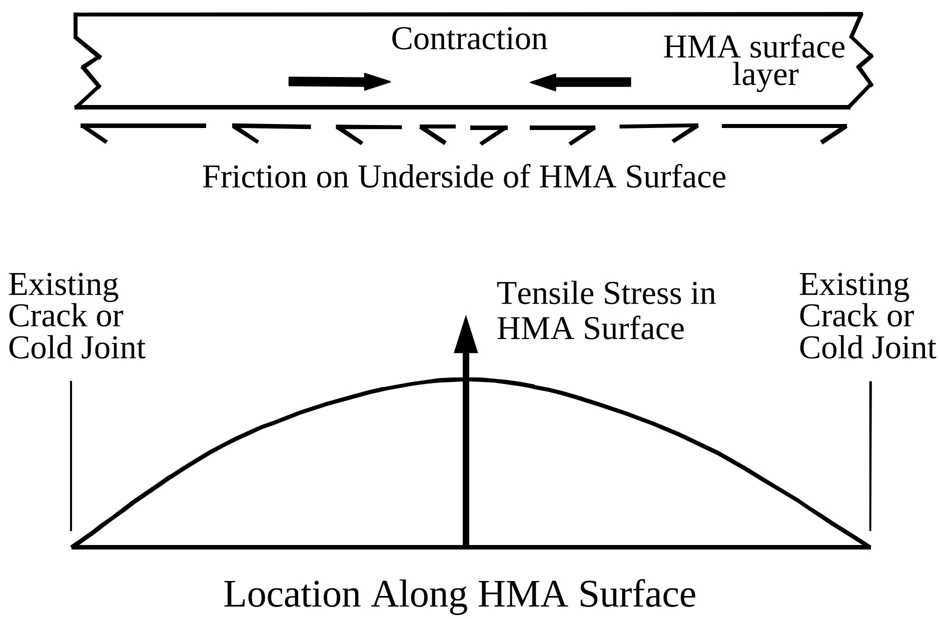


Figure 2-1.10. Development of tensile stress in an HMA layer as it undergoes a temperature-drop.

After the crack develops, continued temperature drop cycles can interact with other environmental and load-related forces to increase the severity. Manifestations of the higher severity states include:

- Wide Cracks. As crack width increases, the potential for moisture infiltration from the surface increases. Moisture, in turn, weakens the underlying layers.

- Spalling. If the transverse cracks are not sealed, they can be infiltrated by incompressible fines materials as well as moisture. These incompressible fines keep the crack from closing as the temperature rises and the pavement expands. Ultimately, the top edges of crack can chip, spall, or break away, resulting in poor ride quality.

- Tenting. If the HMA material at the edge of the transverse crack does not spall as the pavement expands, it can buckle upwards and create a small "tent." The roughness associated with this phenomenon is not only perceptible but unpleasant to the road user.

In general, the best maintenance treatment for a transverse crack (after the fact) is crack sealing. For higher severity cracks, patching may be required.

Another consideration which can be addressed during the rehabilitation design phase (before a new pavement or overlay is constructed) is the selection of an appropriate binder (asphalt cement) for use in the mix. Some asphalt cements are less temperature susceptible (i.e., prone to contraction and/or cracking) than others. One of the products of asphalt studies under the Strategic Highway Research Program (SHRP) was a method of performance grading asphalt cements that permits design engineers to specify a material that is best suited to the environment (see module 3-1).

Frost Heave

The distress associated with this mechanism actually requires a combination of:

- Prolonged low-temperature exposure.
- Moisture availability.
- Frost susceptible materials.

If this combination does not exist, then frost heave will not occur. Figure 2-1.4 provided a photograph of pavement that had undergone significant frost heave.

It is useful to note that frost heave is only a major problem if it impacts the pavement surface profile randomly or differentially. In other words, if a pavement heaves uniformly, then there will be little effect on the surface profile or ride quality. Even if a pavement does experience a uniform heave upwards, however, there will still be a problem at bridge abutments.

Frost heave takes place when moisture existing in an underlying fine-grained layer material is frozen by the presence of extended freezing temperatures. An ice lens develops that tends to attract more moisture and grow as the low-temperature conditions prevail. The growth is ultimately translated into an upwards vertical movement at the surface.

Besides the problem with heaving of the surface, these pavements will also eventually undergo a period of thaw weakening as the ice lenses melt. The damaging effects of wheel loads can be increased tenfold during this period.

In new pavements, the problem of frost heave is addressed through the use of insulating layers and/or the replacement of fine grained natural materials with non-frost susceptible materials. For rehabilitation projects, increased overlay thickness will help insulate the pavement from frost penetration. Also, there may be some cost-effective drainage solutions that can help control the moisture availability.

Moisture-Related

Moisture affects flexible pavements in three distinct fashions:

Strength Loss

The primary effect of moisture is that by "lubricating" the particles in given materials, it makes the materials weaker and less able to distribute applied loads. Thus, the mechanism of fatigue and subsurface rutting are magnified.

In general, moisture effects on pavement and subsurface layer strength are controlled by focusing on improved drainage. Among these methods are:

- Increased cross slope.
- Drainable bases.
- Improved shoulders.
- Improved ditches.

Stripping

This refers to a phenomenon which takes place in an asphalt bound layer whereby the presence of a prolonged high-moisture condition (together with an aggregate with a high-stripping potential) leads to the debonding of the asphalt binder from the aggregate particles. This loss of bond reduces the ability of the asphalt bound layer to carry tensile strains and generally reduces the overall load-carrying capacity of the pavement. In severe cases of stripping (see figure 2-1.11), the HMA or asphalt bound layer stops behaving like a bound layer and actually behaves more like an unbound layer. In fact, in designing a rehabilitation treatment (such as an HMA overlay) that exhibits significant stripping, it is recommended that the "stripped" HMA layer be characterized in terms of the strength as an unbound aggregate base layer.

Figure 2-1.11.   Example of "stripped" HMA layer.

It should be understood that high precipitation levels are not the only way in which the bound layer can be subjected to high moisture. In Nevada, for example, where a desert climate exists, moisture saturation levels can be achieved through a capillary action in the underlying soil and support layers. Various types of surface seals which prevent moisture vapor from escaping can cause or accelerate the problem.

From a mix design standpoint, stripping is usually controlled through the use of aggregates (in asphalt mixes) which do not have a strip potential and/or through the addition of an anti-strip agent (typically lime) to the mix. Permeable base courses are also effective if they keep the asphalt bound layers from prolonged exposure to moisture (from above) or moisture vapor (from below).

Raveling of an asphalt-bound surface layer can be considered one manifestation of stripping if the mechanism that leads to the loss of the surface aggregate particles (chips) involves a debonding of the particles from the asphalt binder as a result of moisture exposure. However, the more common form of stripping is one that takes place below the surface. In fact, stripping can reach an advanced stage of development below the surface without showing any significant signs of distress at the surface. (It is analogous to a termite infestation). Figure 2-1.12 provides a photograph of a "dimple" that can exist at a transverse crack in which stripping exists below.

Figure 2-1.12. Surface distress of HMA pavement surface layer with underlying stripping problem.

The implications of an undetected stripping problem are critical when developing a rehabilitation design for an existing flexible pavement. If an HMA is placed, the likelihood of it serving its intended life will be diminished because of the inherent weakness in a "stripped" layer. Examination of pavement cores is the best way to determine if a stripping problem exists; however, analysis of non-destructive pavement deflection measurements can also provide a clue. If cores are extracted, care should be taken since the process associated with "wet" coring tends to make the stripping problem look worse than it is.

Soil Swelling

Soil swelling (or roadbed swelling as it is referred to in the *1993 AASHTO Guide for Design of Pavement Structures*) refers to a phenomenon whereby the absorption of available moisture by an underlying expansive natural material causes that layer to swell, affect the pavement surface profile and, therefore, decrease the ride quality. Usually, the swelling activity takes place very rapidly during the first few years after a pavement is constructed and the subsurface moisture conditions approach equilibrium. The phenomenon is related primarily to the potential for a given sub-layer to expand as it absorbs moisture and can take place under both flexible and rigid pavements.

Fine-grained (clayey) soils with high plasticity (i.e., plasticity index values greater than 25) are the most susceptible to swelling. The problem is compounded when the thickness of the expansive layer exceeds one meter and when the moisture is absorbed non-uniformly along the pavement profile. (The latter leads to more differential vertical movements which have the greatest negative impact on ride quality). States such as Texas and Colorado with a predominance of expansive soils have experienced swelling activity greater than 300 mm in some locations where the the natural soil is highly expansive and thick.

From the standpoint of designing a new pavement, emphasis should be placed on providing good drainage and/or building a flexible pavement that can withstand the bulk of the swelling and then easily be milled and resurfaced to restore a smooth profile. From a rehabilitation design standpoint, the concerns should primarily be about activities during construction that could change the moisture equilibrium and cause the swelling to repeat after the rehabilitation is completed. An example of an activity that could destabilize the moisture equilibrium is removal of the entire pavement surface layer. (Under certain conditions, this action could result in drying or dessication of the expansive layer and, therefore, the possibility of swelling recurrence).

Intrusion of Fines

In pavement structures where unbound granular layers are placed directly on top of a fine-grained soil, there is a possibility that, with time and available moisture, fines from the natural soil will move vertically and infiltrate the voids in the granular layer. Intrusion of these fine materials break down the aggregate interlock in the granular layers and causes them to lose their strength and load distribution characteristics. The process is accelerated by increased axle loading.

The more common methods used to control the intrusion of fines in the base layers, may be the use of dense-graded, (but slow-draining) granular bases, the use of a filter course to separate more open-graded bases from fine grained soils, or by stabilizing a clayey subgrade. The filter courses may be either a filter fabric or a layer of finer dense-graded gravel. The use of filter layers is discussed in more detail in module 2-5 and very thoroughly in the NHI Course 13213 on Drainage.

Subsurface sampling is required to determine the nature of the fine-grained subgrade soils. Both sieve and hydrometer analysis are required design the filter layer for either a granular or cloth filter.

Age-Related

In flexible pavements, there is only one significant age-related distress mechanism. Actually, it has more to do with exposure to the elements than it does with age.

Oxidation

With time, prolonged exposure of an HMA layer to ultraviolet rays from the sun causes the asphalt (bitumen) to oxidize, lose its aromatics, become stiffer and more susceptible cracking. Figure 2-1.13 provides a photograph depicting a heavily oxidized pavement.

Figure 2-1.13. Example of heavily oxidized HMA surface layer.

Various types of surface treatments (chip seals, fog seals, cape seals, etc.) are effective at interrupting and slowing down the oxidation process. For an oxidized pavement in which there is a significant risk of existing cracks reflecting through a subsequent overlay, there are a variety of recycling techniques (i.e., heater scarification, cold in-place recycling, hot in-place recycling, etc.) in which asphalt, reclamite, or some other recycling agent is added to restore flexibility. HMA recycling is covered in much more detail in module 3-9.

5. RIGID PAVEMENTS

Components

Rigid pavements consist of a PCC surface layer placed typically over some type of granular (or stabilized) base/subbase layer. The PCC surface layer is placed in a particular design configuration that is intended to match the most appropriate materials and construction practices with the prevailing traffic, environmental, and contractor experience conditions. Like flexible pavements, the rigid (PCC) pavement structure is intended to protect the underlying subgrade soil from overstress while minimizing the rate at which it deteriorates. Unlike flexible pavements, the bulk of the applied axle loads are carried by the PCC slab itself. In engineering mechanics terms, this method of load-carrying is referred to as bending and it is very similar to the way a beam or bridge carries its load. Since most of the load is carried by the PCC slab (see figure 2-1.14), it is difficult to justify the need for (or cost of) a base or subbase layer from a structural standpoint. However, when the effects of environment and its interaction with traffic loads are considered, the benefits of an underlying base/subbase layer are undeniable. Basically, they 1) help maintain uniform support of the slab, 2) minimize the damaging effects of prolonged moisture exposure, and 3) serve as an additional layer of insulation against frost penetration. As with the case of flexible pavements, stabilized base courses are also used effectively to increase the load-carrying capacity of rigid pavements. Furthermore, the fact that the layer is bound means that the potential for material erosion is reduced even further, although not totally.

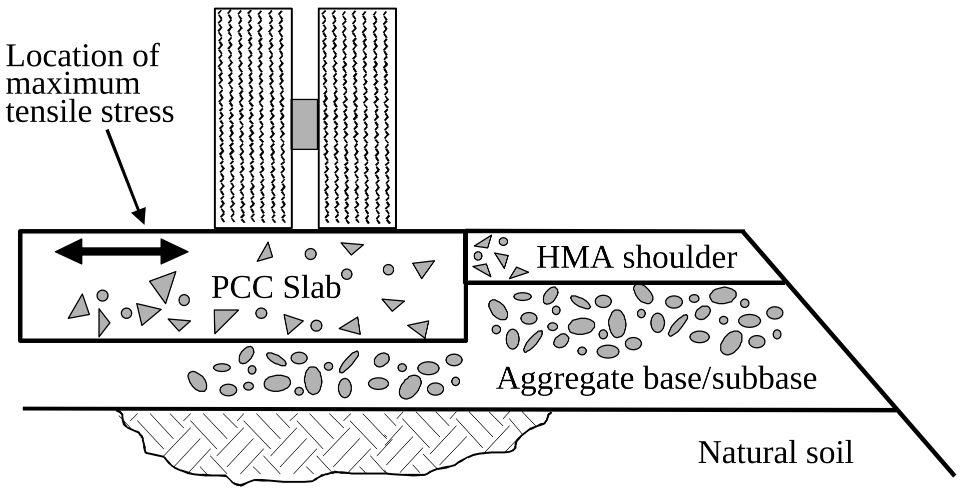


Figure 2-1.14. Illustration of the behavior of a typical PCC slab (with flexible shoulder) when loaded near the slab edge.

The behavior of rigid pavements and their response to various traffic and environmental loading introduces other design and construction related factors:

- Reinforcement. The primary function of steel reinforcement (typically deformed bars or welded wire fabric) is to help hold together (or keep tightly closed) any midslab cracks that may develop. Reinforcement is not intended to carry any of the stresses associated with traffic loading. Deformed bars or deformed welded wire are preferred over smooth welded wire.

- Joint Configuration. Rigid pavements experience the same kind of expensive and contractive stresses that flexible pavements do as they undergo changes in temperature. Transverse joints in rigid pavements represent predefined locations where horizontal movements can take place to alleviate these stresses. These joints are typically established by partial depth sawing soon after concrete placement. The key concern is their spacing. Longer joint spacings increase the potential for midslab cracking. A secondary concern (at least from past experience) is skewness. Joints were skewed to avoid an abrupt transfer of axle loading from one slab to the next. Experience indicates, however, that rigid pavements with skewed joints do not necessarily perform any better than those with perpendicular joints. Perpendicular joints are preferred with doweled joints because of the construction problems associated with skewed joints and dowels.

- Load Transfer. This refers to the method used to achieve a smooth transfer of wheel load from one side of a joint (or crack) in a rigid pavement to the other. Typically, this is accomplished through the use of steel dowel bars placed at mid-depth across transverse joints.

Combinations of these different factors are what lead to the three typical rigid pavement types: JPCP, JRCP, and CRCP. The attributes of these rigid pavement types are discussed later in this module.

Following is a summary of the basic mechanisms which lead to the more common types of distress observed.

Basic Distress Mechanisms

Of the four categories of pavement distress mechanisms identified for flexible pavement, only three actually apply for rigid pavements:

- Load-related.
- Temperature-related.
- Moisture-related.

Age-related mechanisms are not a major factor in the performance of rigid pavements and will not be discussed here.

Load-Related

Axle loadings are considered to be the largest single contributor to the deterioration of rigid pavements, both by themselves and in combination with other factors. Following is a discussion of the two primary load-related distress mechanisms.

Fatigue

The mechanism of fatigue in a rigid pavement refers to a progressive process whereby the PCC slab undergoes so many repeated applications of stress that it eventually cracks. The mechanism is essentially the same as that described for flexible pavements. The difference, however, is that maximum critical stress is not always at bottom of the slab. In fact, because the impact of the pavement edge and transverse joints is so significant in rigid pavements, the critical stress usually exists in the top of the slab. The reason for this is that the rigid pavement behaves more like a cantilever slab when load near the pavement edge or corner. (This is depicted in figure 2-1.14.) The “cantilever” effect” is even worse when voids exist beneath the slab.

Methods of limiting fatigue in new design include thicker slabs, improved joint load transfer and the use of a widened (e.g., 4.6 m) truck lane or tied PCC shoulders. For rehabilitation, control methods include overlays, retrofit load transfer, undersealing and, possibly, retrofit tied shoulders.

Faulting

This identifies a mechanism in which the end-result is a net differential vertical displacement between on slab and the next. Faults can occur at both joints and cracks in rigid pavements. Conditions that contribute to the rigid pavements. Conditions that contribute to the rapid development of faults include excessive moisture contents in the underlying support layer (i.e., base or subbase) and poor load transfer across the joint or crack.

The mechanism is depicted in figure 2-1.15. As an axle load travels down the pavement, pore water pressures will build up in the saturated support layer. In the case of faulting, the build-up of pressure as the axle load moves from the approach (upstream) slab to the leave (downstream) slab, that it causes suspended fine materials underneath the leave slab to be “injected” underneath the approach slab. The continuous build-up of fines causes the approach slab to lift or displace upwards relative to the leave slab. Faults as little as 2 mm are perceptible to the road user.

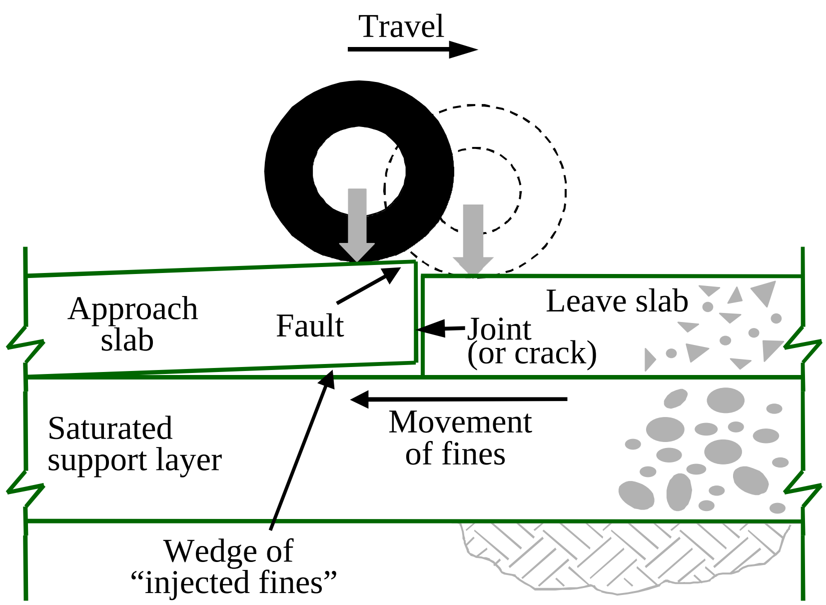


Figure 2-1.15. Illustration of faulting mechanism.

Grinding is the most popular method of removing faults as part of a rehabilitation operation. However, it does not address the mechanism. Attention to improved drainage and/or minimizing the infiltration of moisture beneath the slab (e.g., joint sealing) will help reduce the rate of faulting. Also, retrofit load transfer devices can be effective at improving load transfer and avoiding the sudden increase in pore pressure under the leave slab that causes the fines to migrates.

Temperature-Related

Rigid pavements can experience distress as a result of significant decreases and increases in slab temperature from the temperature at which it was constructed. In both cases, the distress comes about as the slab tries to contract or expand.

Low-Temperature Midslab Cracking

This mechanism is basically identical to the thermal cracking mechanism described for flexible pavements (see figure 2-1.10). As the slab undergoes a drop in temperature, it will try to contract. The contraction is restrained by friction along the slab-base interface. If the frictional restraint is high enough and/or the slab is long enough, the tensile stress generated at midslab can exceed the strength of the concrete and a crack will develop. If no reinforcement exists to hold the crack together, its severity in terms of width and spalling can increase very rapidly. (This is the basis for the differences in design between JPCP, JRCP, and CRCP).

In terms of rehabilitation, these are limited methods available for addressing a midslab cracking, most commonly either full-depth repair or dowel retrofit. The severity of the crack distress should at least be moderate to justify the expense.

High-Temperature Joint/Crack Distress

In contrast to low temperature effects, high pavement temperatures and the associated slab expansion can lead to distresses that are minor (spalling) to catastrophic (blowups). Factors that further contribute to the development of these distresses include:

- Longer joint/crack spacings.
- The use of expansive coarse aggregates in the PCC mix.
- Poor joint sealing practices.

Longer joint/crack spacings and expansive coarse aggregates increase the horizontal movement potential. Poor joint sealing practices increase the likelihood that incompressible materials will infiltrate the joints or cracks and restrain the expansion of the slab.

Spalling is primarily attributed to the localized overstress conditions that exist when incompressibles infiltrate the upper portion of the slab. Joint crushing and blowups can take place when the incompressible penetrate the full depth of the slab. In terms of rehabilitation options, spalling is typically addressed through partial depth repairs, while joint crushing and blowups call for full depth repairs. Pressure relief joints have also been used as a preventive measure (because they provide for more slab growth), however, they can create problems with loss of load transfer in rigid pavements which rely on aggregate interlock.

Moisture-Related

Moisture has adverse impacts on rigid pavement performance, primarily in the way it compounds other distress mechanisms:

In the case of fatigue, it weakens the underlying pavement layers and increases the load associated stresses that must be carried by the PCC slab. In the case of faulting, it is the medium that suspends the fine materials and allows them to be transported from one side of the slab to the other. For the case of high temperature joint/crack distresses, it helps to wash the incompressible materials down into the existing joints and cracks.

There are two other mechanisms that are considered to be more moisture-related, even though there are other interactive factors involved.

Pumping

This mechanism is very similar to the one that leads to faulting. The differences are that 1) the pumping action results in the fine-grained materials being “pumped” to the surface, 2) the pumping action can take place at lower pore pressures (and, therefore, throughout the length of the pavement), and 3) the process usually results in a void beneath the slab.

The primary problem with pumping is the effect the resulting voids can have on slab deterioration. As the voids develop, the slab becomes less and less supported by the underlying materials and the “cantilever effect” magnifies. Also, because of the rigid nature of the slab, it does not take a very large void to cause the pavement to wear out faster. Corner breaks and punchouts are the most critical distresses associated with this mechanism.

From the standpoint of rehabilitation, undersealing is the most common method used for filling any voids. However, preventive measures such as improved drainage, joint and crack sealing, dowel retrofit, and the use of less erodible base courses (in new design) are more dependable and cost-effective.

D-Cracking

Durability cracking is primarily a climate/materials related mechanism. Concrete pavements in northern (freezing) climates made with coarse aggregates that have a significant amount of voids tend to be the most susceptible to D-cracking. Moisture from various sources saturates the concrete and fills the voids in the coarse aggregate particles. Later, when frost penetrates the slab, the water in these voids freezes, creates internal expansive forces, and causes both the aggregate particles and surrounding concrete paste to disintegrate. D-cracking can take place throughout the slab, both horizontally and vertically. However, it tends to be focused more at existing joints (or cracks) and usually works its way up from the bottom of the slab.

As part of the design for a new concrete pavement (or concrete overlay), the mechanism of D-cracking can be addressed by avoiding the use of susceptible aggregates or keeping the maximum coarse aggregate size below 19 mm. In treating an existing pavement that has a D-cracking problem, it is possible to use full depth PCC patches at joints or cracks if they are the limit of the D-cracking problem. If the D-cracking problem is more extensive, it may be better to reconstruct, recycle, or crack/break and seat the pavement structure.

Alkali-Silica Reactivity (ASR)

Alkali-Silica reactivity may occur in pavements made with coarse or fine aggregate with a high silica content. The aggregates may react in alkali solutions. The cement and water in a concrete mix provides the soluble alkali source for a reaction. The alkali-silica reaction forms a gel that creates internal pressures int he concrete leading to extensive map cracking. The cracking usually grows over time as the gel continues to form. The compressive pressures from the gel formation also cause increased blowup problems and considerable pressure against bridge abutments which in a few cases have pushed over bridge abutments.

Alkali-Silica reactivity is reduced through the use of aggregate with lower silica contents. The use of low-alkali type II cement, lower water cement ratios, and adding a type F fly ash may also reduce the potential for ASR.

Rigid Pavement Types

Rigid pavements consist of a portland cement concrete pavemnet placed over a series of other paving layers. Although some agencies are experimenting with the use of post-tensioned or prestressed concrete pavements, the vast majority of rigid pavements in the United States fall under one of three categories:

- Jointed plain concrete pavement (JPCP).
- Jointed reinforced concrete pavement (JRCP).
- Continuously reinforced concrete pavement (CRCP).

Consequently, the emphasis on rigid pavement rehabilitation in this maual is on the basic types.

Jointed Plain Concrete Pavement (JPCP)

Jointed plain concrete pavements do not contain steel reinforcing mesh in the slab. The only steel in the pavement is the tiebars that are placed across the longitudinal lane joint and dowel bars present at the transverse joints. Because there is no steel to control any transverse slab cracking, joint spacing for this design is kept short, typically 3.7- to 6.1-m. The shorter joint spacings also assist in maintaining aggregate or grain interlock load transfer at the transverse joints if dowel bars are not included. Slab thicknesses historically have ranged from 150- to 250-mm, although newer pavements are being constructed 300 mm or more in thickness.

The basic concept of JPCP design is to provide a slab with adequate thickness to prevent fatigue cracking (transverse midslab cracking and corner breaks). Short joint spacing is necessary to minimize midslab cracking and to prevent excessive joint openings that would reduce transverse joint load transfer in the absence of dowel bars. Short joint spacing is also required to reduce warping stresses, particularly when the rigid pavement is placed on a stabilized base.[27,28]

Transverse joints in JPCP are often weakened-plane contraction joints. Dowel bars typically are 38 mm diameter, smooth round steel bars, 460 mm long, and spaced every 300 mm across the lane width. The subbase beneath the slab may either be granular, stabilized (with asphalt or cement), or lean concrete. The typical subbase thickness is 100- to 300-mm, depending upon pavement support and drainage requirements.

The performance of JPCP is highly dependent upon transverse joint load transfer and the erodibility of the base. Load transfer at transverse joints in JPCP without dowels is primarily dependent upon stiff base support and aggregate interlock between the two abutting joint faces. Loss of aggregate interlock across the joint face may occur due to the thermal contraction of the slab. This will create large corner deflections under heavy truck loadings that, in conjunction, with an erodible base and excess moisture, cause pumping and faulting of the transverse joints. Continued pumping and faulting eventually lead to loss of pavement support, which can cause transverse cracking and corner breaks. JPCP that contain dowel bars resist faulting much longer than non-doweled JPCP because of reduced corner deflections. The amount of free water beneath the slab also affects the potential for erosion and faulting.

Repeated traffic loading can also create deterioration in JPCP in the form of midslab fatigue cracking. Midslab transverse cracking may also develop due to thermal gradient stresses if the joint spacing is too long or if the joints lock-up and are no longer functioning. Such transverse cracks serve as entry points for water infiltration, which can lead to additional deterioration. They also will fault, spall, and break down under traffic, thereby increasing roughness.

Figure 2-1.16 illustrates the general manner in which cracking, faulting, pumping, joint deterioration (spalling), and loss of serviceability progress in jointed concrete pavements. Figure 2-1.17 illustrates the progression of faulting for a non-doweled pavement with (9.1 m) joint spacing, a non-doweled pavement with (4.6 m) joints, and a doweled pavement with (4.6 m) joint spacing. The benefits of shorter joint spacing and of dowel bars are clearly shown. Based on faulting and mid panel cracking experience the 9.1m joint spacing is not recommended.

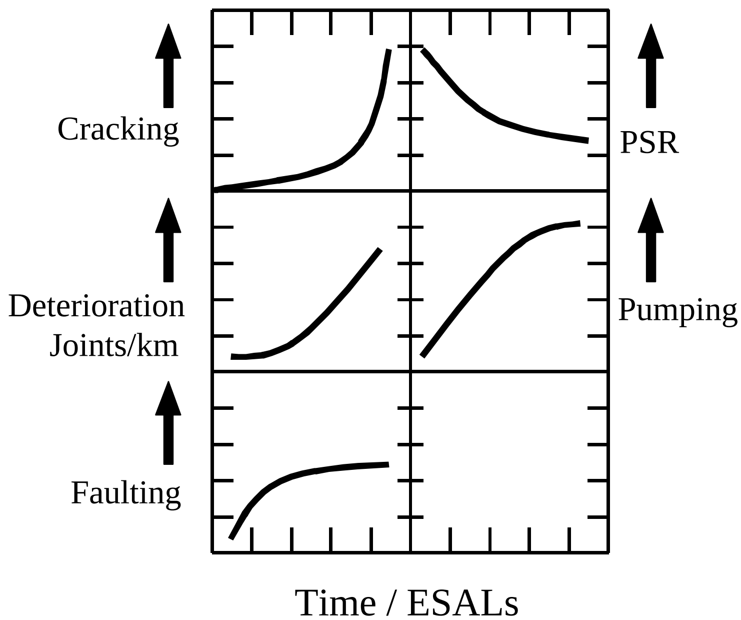


Figure 2-1.16. Progression of key distresses and loss of serviceability in jointed concrete pavement.[15]

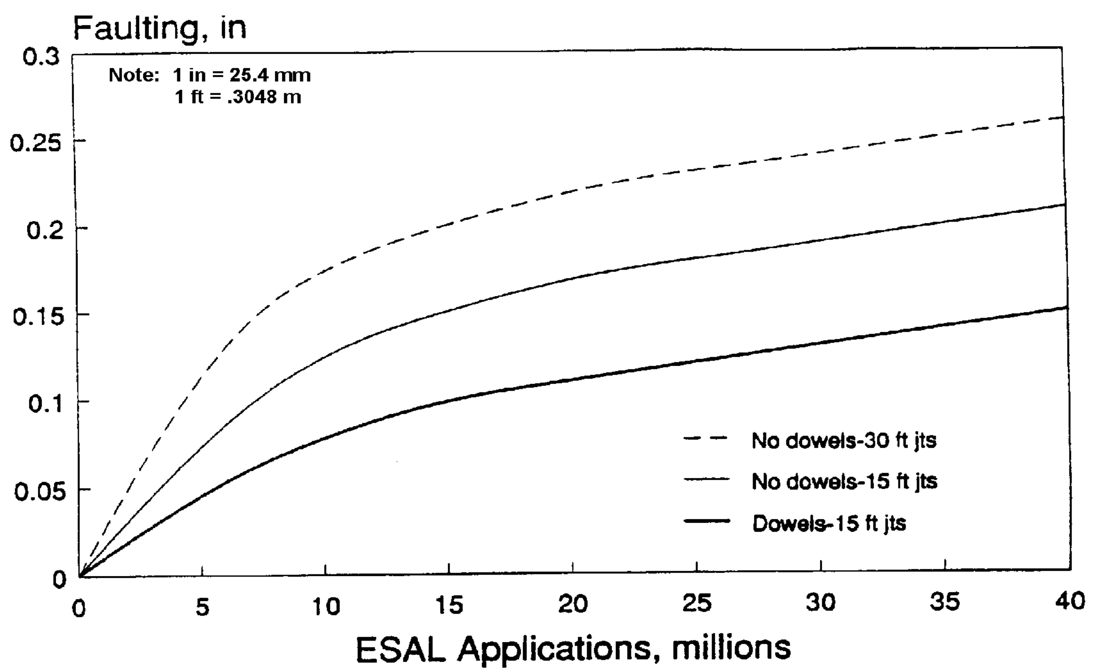


Figure 2-1.17. Effects of dowels and joint spacing on development of transverse faulting.[16]

Figure 2-1.18 shows the predicted transverse cracking for three concrete pavement designs with joint spacings of 3.0 m, 4.6 m, and 6.1 m, respectively. Shorter joint spacings result unless cracking due to the reduced thermal curling stress.

Typical JPCP distresses are defined in references 5 and 6. Additional information on the performance of JPCP is provided in references 15 through 19.

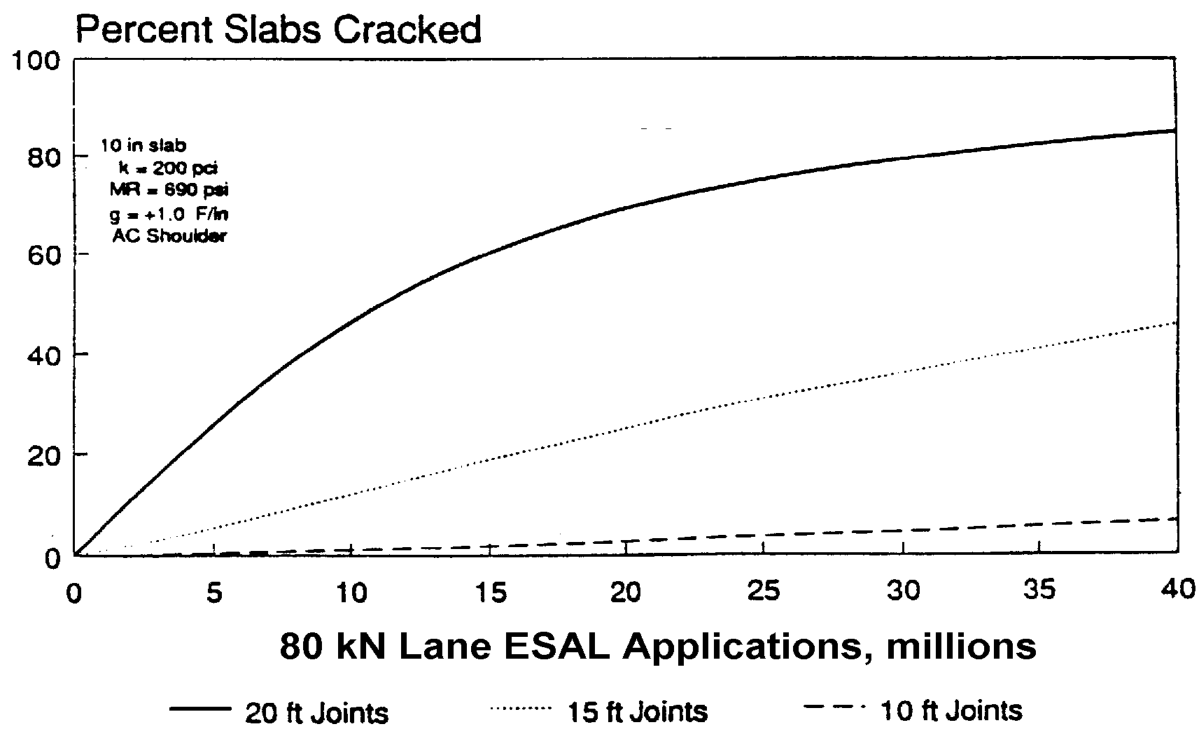


Figure 2-1.18. Effects of joint spacing on development of JPCP slab cracking.[16]

Jointed Reinforced Concrete Pavement (JRCP)

This type of concrete pavement consists of slabs ranging in length from about 8- to 30-m. Because of the longer slab lengths, steel reinforcement is placed in the slab to help control the transverse cracks that will develop. The reinforcement is normally welded wire fabric, with longitudinal wire typically 8.4 mm in diameter and placed on 150 mm centers, and the transverse wires typically 5.8 mm in diameter and placed on 300 mm centers.

The amount of reinforcing steel is quite small, typically 0.10 percent to 0.20 percent of the slab cross-sectional area. The purpose of the reinforcing steel is to hold cracks together so that aggregate interlock will exist across cracks. The steel is not intended to take tensile stress from traffic loadings. Slab thicknesses range from 150- to 250-mm, with the underlying subbases typically being a granular material, although stabilized bases also have been used. The transverse joints are typically weakened-plane contraction joints that contain smooth dowel bars.

The basic design concept for JRCP is to provide adequate slab thickness to prevent fatigue cracking and to provide adequate steel content to hold transverse cracks tightly together. The joint design is also critical, as it must provide adequate load transfer to minimize faulting, and also maintain an adequate seal to minimize the infiltration of water and incompressibles.

Within the first few years after construction, several transverse cracks develop in JRCP from a combination of concrete shrinkage, thermal curling, and traffic loadings. Since most JRCP are located in areas where deicing salts are used extensively, corrosion of the steel dowel bars is common and can result in a lockup of “freezing” of the joint. If that happens, considerable stress will be placed on the longitudinal steel at transverse cracks, forcing the midslab cracks to open. Chlorides from deicing salts then can seep into the crack and corrode the reinforcement. Heavy repeated loads, pumping, and corrosion of the steel can break down the transverse cracks, which results in their faulting and spalling and creating increased roughness. A minimum of 75mm of concrete cover is required to minimize these effects.

The deterioration of JRCP follows the same general trend as that of JPCP, as illustrated in figure 2-1.16. In JRCP, however, the major distress is the deterioration of the transverse joints. One of the major causes of joint deterioration is the infiltration of incompressibles into the joint. This can create large expansive pressures in the slab that can result in spalling and blowups in the pavement. This occurs more often on JRCP with long joint spacing (i.e., 12 m or greater). Figure 2-1.19 shows an example of the rate of joint deterioration for JRCP. The effect of joint spacing on the development of deteriorated joints and deteriorated cracks is shown in figure 2-1.20.

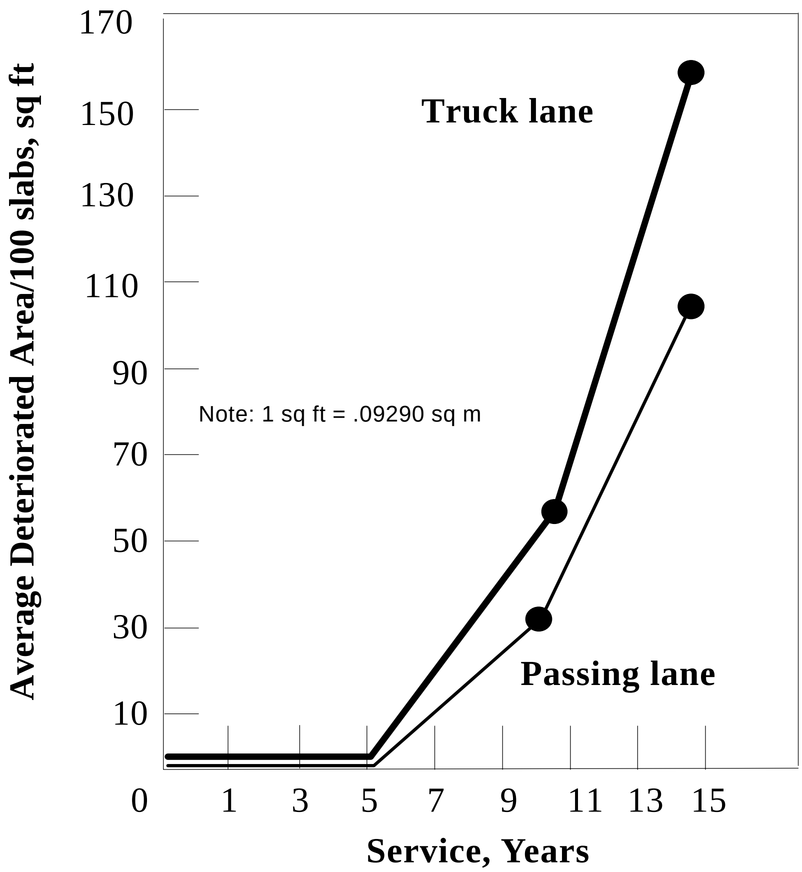


Figure 2-1.19. Illustration of the rate of joint deterioration of JRCP in Michigan.[20]

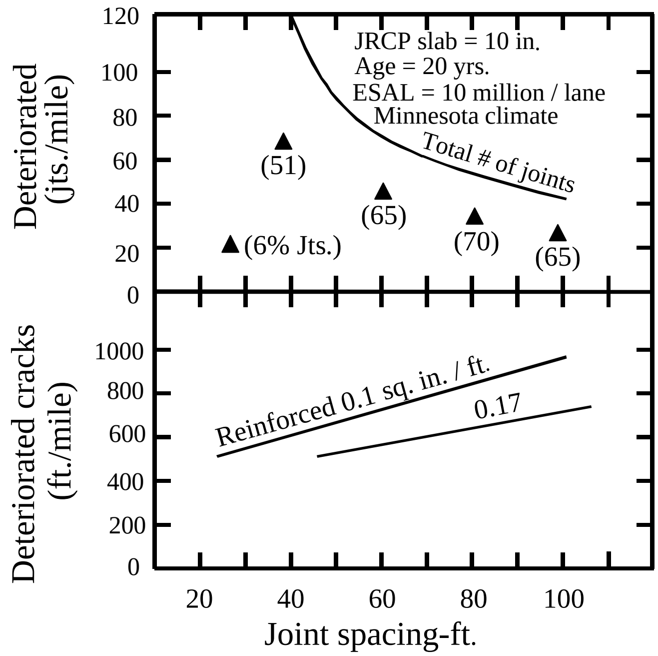


Figure 2-1.20. Effect of joint spacing on JRCP joint spalling and transverse cracking.[15]

Typical JRCP distresses are described in references 5 and 6. Additional information on JRCP performance is provided in references 15 through 20.

Continuously Reinforced Concrete Pavement (CRCP)

Continuously reinforced concrete pavements are concrete pavements constructed without regularly-spaced transverse joints. CRCP contain larger amounts of steel reinforcement, typically 0.5 percent to 0.8 percent of the slab cross-sectional area, and the steel is intended to force the slab to crack at short intervals in the range of 1.0- to 2.5-m. The only joints placed in CRCP are construction joints or joints placed at structures.

The longitudinal steel reinforcement is made continuous by overlapping the steel rebars. The size and type of the reinforcing bars vary considerably from agency to agency, but typically the longitudinal steel consists of 16 mm diameter bars spaced on 150 mm centers. If transverse steel is used, it typically consists of 13 mm bars placed on 1.2 m centers.

In the past, CRCP slab thicknesses have been about 25- to 75-mm thinner than the conventional jointed design for the same traffic level. However, recently the practice has been to construct CRCP of the same thickness as jointed concrete pavements for the best performance. Bases for CRCP have normally consisted of stabilized materials. A high type of HMA, or a lean concrete or cement-treated base, with a smooth surfaced bond breaker has been found to provide the best combination of non-erodible support as well as moderate frictional resistance for proper crack development. The basic concept of CRCP design is to provide adequate slab thickness to resist deflections and high stresses. In addition, sufficient steel must be included to hold the transverse cracks tight and a non-erodible base should be used to provide stiff uniform support and to resist pumping and erosion.

The performance of CRCP is particularly susceptible to the quality of construction. Poor concrete consolidation, inadequate steel laps, and improper steel reinforcement depth have often led to premature pavement deterioration. In addition, some projects have experienced spalling distress caused by corrosion of the steel rebar, similar to bridge deck deterioration.

Figure 2-1.21 shows the truck loading situation that leads to the structural deterioration of CRCP (punchouts). Heavy repeated loadings and free water beneath the slab often lead to pumping, erosion, and loss of support, which can cause a breakdown of the transverse cracks and, finally, edge punchouts, as shown in figure 2-1.22.

The key factor in the breakdown of the cracks is the crack width. If the cracks open significantly, deterioration of the crack will soon follow, resulting in a punchout. Low steel content, steel placed too deep in the slab, and large deflections all contribute to crack opening.

A study in Illinois compared the performance of CRCP and JRCP.(22) These results, illustrated in figure 2-1.23, indicate that an 8-in (200 mm) CRCP had the same service life as the 10-in (250 mm) JRCP. However, it should be noted that the performance of the 10 in (250 mm) JRCP was adversely affected by excessive joint spacing and corroded dowel bars. The Illinois study also notes that D-cracking has a severe effect on the performance of CRCP, and that, after slab thickness, the base type and the amount of reinforcement have the most significant effect on the performance of the CRCP.(22) Widening the outside lane to 4.6m to reduce deflections and pumping also improves performance.

Typical CRCP distresses are defined in references 5 and 6. Additional information on CRCP performance is found in references 21 through 25.

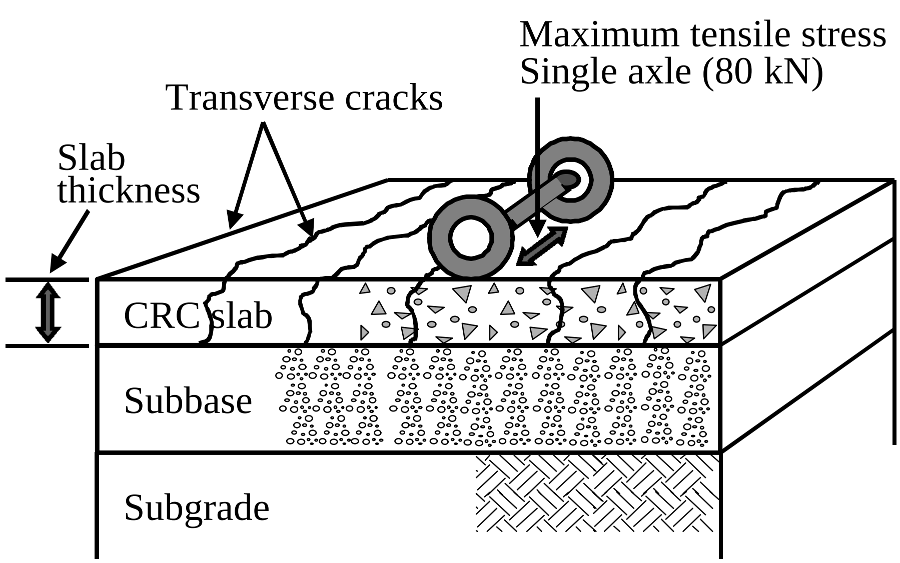


Figure 2-1.21. Diagram of truck loading that produces critical tensile stress in top of CRCP.(21)

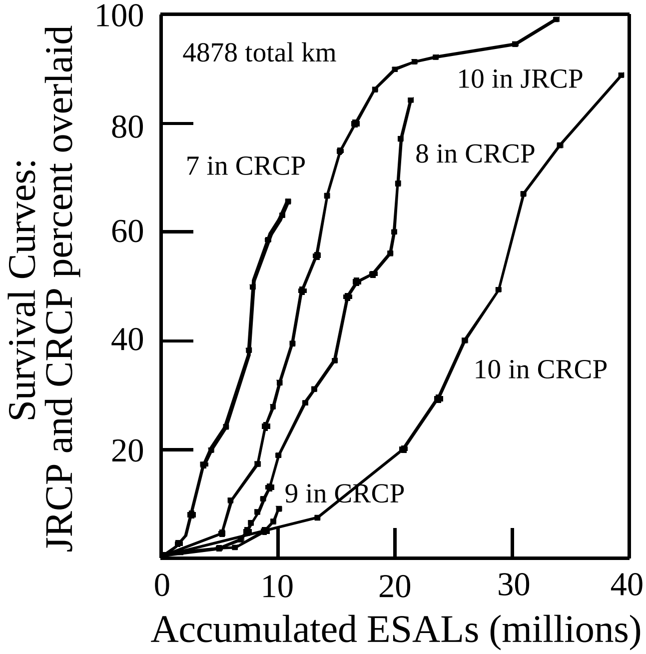


Figure 2-1.22. Illustration of edge punchout in CRCP.(21)

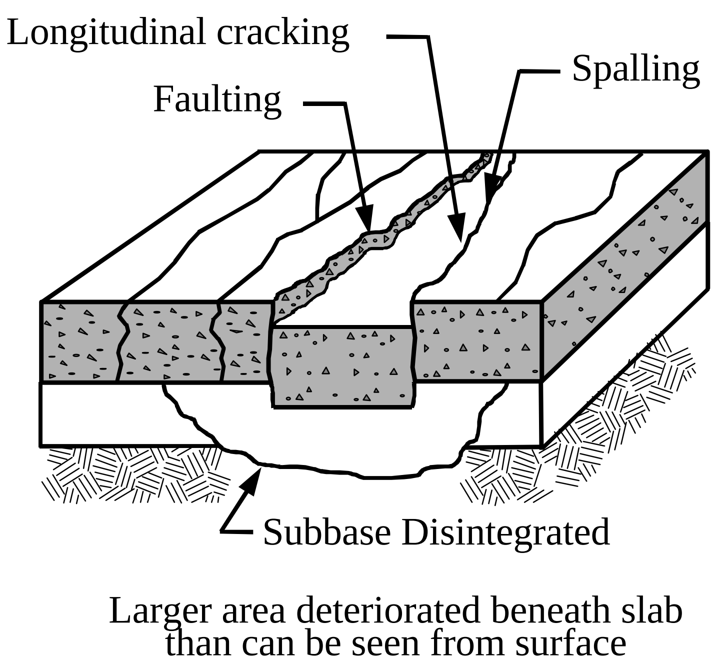


Figure 2-1.23. Survivial curves for 7-,8-,9-, and 10-in CRCP and 10-in JRCP.(22)

6. ASPHALT/CONCRETE COMPOSITE PAVEMENTS

Composite pavements, consisting of an HMA surface layer over a PCC layer, can be built as new construction or may result when an HMA overlay is placed on an existing PCC pavement. The basic goal of HMA/PCC composite pavement design is to provide sufficient combined thickness of HMA and PCC to eliminate cracking in the HMA layer. The joint system in the underlying concrete slab may range from no joints (CRCP) to joints spaced from 4- to 30-m apart. A design procedure for composite pavements is presented in reference 7.

Typical load-associated fatigue cracking is not common on this type of pavement due to the structural presence of the underlying concrete slab. Rutting in the HMA can occur, however, especially if the pavement is subjected to heavy traffic loading or if stripping occurs at the HMA/PCC interface. If alligator cracking occurs, there is likely to be stripping at the HMA/PCC interface or the bond between the layers has been broken.

The most common problem with this pavement type is reflection cracking from joints and cracks in the underlying concrete slab. This is the result of a combination of horizontal thermal movement of the underlying slab, and vertical differential movement from heavy loads crossing the joints and cracks. Infiltration of water into the cracks, along with freeze-thaw cycling and repeated loadings, usually result in breakup or spalling of the HMA surface. This spalling can also result from deterioration in the underlying concrete slab. Additional information on reflection cracking is provided in references 8 through 14 and in module 4-14.

If the PCC slab experiences severe deterioration due to D-cracking, localized failures may occur in the HMA overlay and severe rutting and potholes can develop.

7. SUMMARY

This module gives a summary of the various pavement types, their behavior under various loading conditions and their typical performance trends. It is very important that the engineer obtain the as-built plans of the pavement project under consideration and become thoroughly familiar with the details of its design. A knowledge of the existing design and an understanding of its typical performance and underlying distress mechanisms is needed to identify the causes of distress so that the most appropirate rehabilitation techniques can be identified.

8. REFERENCES

1. Yoder, E.J. and M.J. Witczak, Principles of Pavement Design, Wiley Publishers, 1975.

2. "Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials, 1986.

3. Carpenter, S.H., M.I. Darter, A.L. Mueller, M.B. Snyder, K.D. Smith, K.T. Hall, "Pavement Design-Principles and Practices," Course Notebook, Federal Highway Administration/National Highway Institute, September 1987.

4. Zube, E. and J. Skag, "Final Report on the Zaco-Wigmore Asphalt Test Road," Proceedings, Association of Asphalt Paving Technologists, Vol. 38, 1969.

5. Smith, R. E., M.I. Darter and S.M. Herrin, “Highway Pavement Distress Identification Manual for Highway Condition and Quality of Highway Construction Survey,” FHWA-RD-79-66, Federal Highway Administration, March 1979.

6. “Distress Identification Manual for the Long-Term Pavement Performance Studies,” SHRP-LTPP/FR-90-001, Strategic Highway Research Program, October 1990.

7. George, K.P., J.S. Rao, and K. Fredrickson, “Composite Pavement Design Procedure, Volumes 1 and 2,” FHWA/RD-87/012 and FHWA/RD-87/013, Federal Highway Administration, 1987.

8. Jayawickrama, P.W., R.E. Smith, R.L. Lytton, and M.R. Tirado-Crovetti, “Development of Asphalt Concrete Overlay Design Equations, Volumes I and II,” FHWA/RD-87/077 and FHWA/RD-87/078, Federal Highway Administration, 1987.

9. Majidzadeh, K., G. J. Ilves, and V. R. Kumar, “Improved Methods to Eliminate Reflection Cracking,” FHWA/RD-86/075, Federal Highway Administration, 1985.

10. Thompson, M.R., “Breaking/Cracking and Seating of Concrete Pavements,” Synthesis of Highway Practice 144, Transportation Research Board, March 1989.

11. Kilareski, W.P., and R.A. Bionda, “Structural Overlay Strategies for Jointed Concrete Pavements, Volume I—Sawing and Sealing of Joints in AC Overlays of Concrete Pavements,” FHWA-RD-89-142, Federal Highway Administration, June 1990.

12. Kilareski, W.P., and S.M. Stoffels, “Structural Overlay Strategies for Jointed Concrete Pavements, Volume II—Cracking and Seating of Concrete Slabs Prior to AC Overlay,” FHWA-RD-89-143, Federal Highway Administration, June 1990.

13. G.F. Voigt, S.H. Carpenter, and M.I. Darter, “Rehabilitation of Concrete Pavements, Volume II—Overlay Rehabilitation Techniques,” FHWA-RD-88-072, Federal Highway Administration, July 1989.

14. Sherman, G., “Minimizing Reflection Cracking of Pavement Overlays,” Synthesis of Highway Practice 92, Transportation Research Board, September 1982.

15. Darter, M.I., J.M. Becker, M.B. Snyder, and R.E. Smith, “Concrete Pavement Evaluation System (COPES),” NCHRP Report 277, Transportation Research Board, 1985.

16. Smith, K.D., A.L. Mueller, M.I. Darter, and D.G. Peshkin, “Performance of Jointed Concrete Pavements, Volume II—Evaluation and Modification of Concrete Pavement Design and Analysis Models,” FHWA-RD-89-137, Federal Highway Administration, July 1990.

17. Darter, M.I. and E.J. Barenberg, “Zero Maintenance Pavement: Results of Field Studies on the Performance Requirements and Capabilities of Conventional Pavement Systems,” FHWA-RD-76-105, Federal Highway Administration, 1976.

18. “Joint Related Distress in PCC Pavement,” Synthesis of Highway Practice 56, Transportation Research Board, 1979.

19. Smith, K.D., D.G. Peshkin, M.I. Darter, A.L. Mueller, and S.H. Carpenter, “Performance of Jointed Concrete Pavements, Volume I—Evaluation of Concrete Pavement Performance and Design Features,” FHWA-RD-89-136, Federal Highway Administration, March 1990.

20. Oehler, L.T., “Performance of Michigan’s Postwar Concrete Pavement, Research Report R-74, Michigan State Highway Commission, 1970.

21. LaCoursiere, S.A., M.I. Darter, and S.A. Smiley, “Performance of CRCP in Illinois,” FHWA-IL-UI-172, Illinois Department of Transportation, 1978.

22. Dwiggins, M.E., M.I. Darter, J.P. Hall, C.L. Flowers, and J.B. DuBose, “Pavement Performance Analysis of the Illinois Interstate Highway System,” FHWA-IL-UI-220, Illinois Department of Transportation, July 1989.

23. “Failure and Repair of Continuously Reinforced Concrete Pavement,” Synthesis of Highway Practice 60, Transportation Research Board, 1979.

24. Neff, T.L. and G.K. Ray, “CRCP Performance: An Evaluation of Continuously Reinforced Concrete Pavements in Six States,” Concrete Reinforcing Institute, 1986.

25. Zollinger, D.G. and E.J. Barenberg, “Continuously Reinforced Pavements: Punchouts and Other Distresses and Implications for Design,” FHWA/IL/UI 227, Illinois Department of Transportation, March 1990.

26. “Pavement Distress and Rehabilitation Manual,” Massachusetts Highway Department, December 1993.

27. M.I. Darter, K.T. Hall, and Chen-Ming Kuo, “NCHRP Report 372 Support Under Portland Cement Concrete Pavements,” Transportation Research Board, National Academy Press, Washington, DC, 1995.

28. K.T. Hall, M.I. Darter, T.E. Hoerner, and L. Khazanovich, “LTPP Data Analysis Phase 1: Validation of Guidelines for K-Value Selection and Concrete Pavement Prediction,” FHWA Report No. FHWA-RD096-198 January 1997, FHWA McLean, VA.

MODULE 2-2

CONDITION DATA COLLECTION AND PROCESSING

1. INSTRUCTIONAL OBJECTIVES

This module describes the surveys and equipment commonly used to assess the functional performance of a pavement section being considered for rehabilitation as well as techniques for evaluating the data such that feasible, cost-effective rehabilitation alternatives can be identified. In particular, it describes the procedures for the measurement and assessment of pavement distress, pavement roughness, and pavement surface friction. At the conclusion of this module, the participant should be able to accomplish the following:

1. Describe the three factors required to properly characterize distress.
2. List the field procedures necessary to perform a project distress survey.
3. Describe the automated equipment available to perform distress surveys.
4. Describe the information that can be obtained from a roughness survey and why roughness is an important aspect of pavement condition.
5. Describe the types of equipment commonly used to measure pavement roughness.
6. Describe the information that can be obtained from surface friction surveys and why surface friction is an important aspect of pavement condition.
7. Describe the types of equipment commonly used to measure surface friction.
8. Describe the process of the way information obtained from the condition surveys can be used to aid in selecting appropriate rehabilitation strategies to optimize performance, funding levels, and safety.

2. DEFINITIONS

<u>Pavement Condition</u>. The state or “health” of a pavement. Distress, roughness, and surface friction are components of pavement condition.

<u>Pavement Distress</u>. Pavement distress can be defined as flaws such as cracks, potholes, material loss, etc. in a pavement.

<u>Pavement Roughness</u>. Roughness can be defined as the distortion (longitudinal and tranverse) of the road surface.

<u>Pavement Surface Friction</u>. The frictional force developed at the tire-pavement interface.

3. CONDITION SURVEY OVERVIEW

A distress survey is typically the first step in a series of surveys to assess the condition of a pavement. These surveys record the distress that is visible on the surface of the pavement; by knowing that distress, much information can be inferred as to the underlying causes of the deterioration. For example, the appearance of pumping at the joints of a jointed concrete pavement is an indication that, in the vicinity of the joint, excessive deflections are occurring, erosion of one of the underlying layers is occurring, and excess moisture is present beneath the slab.

Such distress information can suggest further investigations, such as deflection testing, coring, or other investigative actions, in order to verify the and extent of the deterioration. By knowing this information, appropriate rehabilitative measures can be identified.

In conjunction with distress surveys, roughness is also commonly measured on pavements in order to obtain an indication of how well the pavement is serving the user. Excessive roughness can create user discomfort and irritation and can lead to increased vehicle operating costs. Roughness is a key indicator of pavement performance from the user's perspective, and is often used as a trigger to indicate that some sort of rehabilitation is required because the pavement rideability has become unacceptable.

Surface friction is also commonly measured on pavements as part of the condition evaluation to identify pavement segments that have surface friction. Surface friction values along with wet weather accident rates are used to identify potential sections of pavement where low surface friction may contribute to wet skidding accidents. Inadequate surface friction can lead to accidents and therefore identification of deficient areas is an important part of the condition evaluation of the pavement.

The following sections describe how distress, roughness, and surface friction are typically measured. Following these sections is a section describing how the data obtained from the surveys can be reduced and used to aid in identifying rehabilitation strategies.

4. DISTRESS SURVEYS

The most basic measure of pavement condition is existing distress. There are many types of distresses and different severity levels for each type. Pavement distresses are the result of one or more factors which, when known, provide insight into the causes of pavement deterioration. Distresses are identified by considering the following three factors:

- Type–The type of distress is determined primarily by similar mechanisms of occurrence and appearance.

- Severity–The severity of distress represents the criticality of the distress in terms of progression; more severe distresses will require more extreme rehabilitation measures.

- Amount–The quantity of each type and severity level must be measured and expressed in convenient terms.

Any method of distress measurement that ignores one of these three factors will not provide adequate information for design of a rehabilitation project.

In recent years significant progress has been made in the standardization of distress identification in the form of distress identification manuals. Several have been developed for different applications including highways,[1,2] streets,[3] and airports.[4] The recently-completed *Distress Identification Manual for the Long-Term Pavement Performance Project*[2] was developed under the Strategic Highway Research Program (SHRP) to provide a uniform basis for the identification and measurement of pavement distresses for the SHRP Long-Term Pavement Performance Program (LTPP) research study. The manual includes distresses for the following pavement types:

- Asphalt concrete pavements (including asphalt overlays).
- Plain and reinforced jointed concrete pavements (including concrete overlays).
- Continuously reinforced concrete pavements.

The SHRP distress identification manual[2] contains approximately 16 distresses for each of the above pavement types. The manual describes each distress type, it defines the severity levels so that different levels of deterioration can be gauged, and it explains the standard units in which the distress is measured. An example description from the manual for fatigue cracking in asphalt concrete pavements is provided in table 2-2.1.

Figures and photographs of the distress type at various levels of severity are also provided in the manual to aid the surveyor in the distress identification process. Figure 2-2.1 provides an example figure for fatigue cracking in asphalt concrete pavements.

Table 2-2.1. Example description for fatigue cracking in asphalt concrete pavements.[2]

Fatigue Cracking

Description

- Occurs in areas subjected to repeated traffic loadings (wheelpaths).
- Can be a series of interconnected cracks in early stages of development. Develops into many-sided, sharp-angled pieces, usually less than 0.3 m (1 ft) on the longer side, characteristically with a chicken wire/alligator pattern, in later stages.
- Must have a quantifiable area.

Severity Levels

- **Low**–An area of cracks with no or only a few connecting cracks; cracks are not spalled or sealed; pumping is not evident.
- **Moderate**–An area of interconnected cracks forming a complete pattern; cracks may be slightly spalled; cracks may be sealed; pumping is not evident.
- **High**–An area of moderately or severely spalled interconnected cracks forming a complete pattern; pieces may move when subjected to traffic; cracks may be sealed; pumping may be evident.

How to Measure

- Record square meters (square feet) of affected area at each severity level.
- If different severity levels existing within an area cannot be distinguished, rate the entire area at the highest severity present.

Pavement distress surveys traditionally have been conducted manually (i.e., with hand-held tools and survey forms). More recently, however, automated equipment has become available to aid in the survey process. The following paragraphs describe manual distress surveys as well as introduce the participant to automated distress survey equipment.

Manual Distress Surveys

Manual distress surveys have been performed for many years and serve as a cornerstone in the development of rehabilitation alternatives. With adequate field training of the survey crew, these surveys provide reliable and consistent results. Equipment needed for a manual distress survey is readily available and should include:

- Hand odometer (measuring wheel) or tape measure (at least 30 m) for measuring distances.
- Stringline or straightedge between 1 m and 2 m for measuring rut depth and/or dropoff.

- Small scale or ruler for fine measurements.
- Mid- to full-sized vehicle.
- Faultmeter or other means for measuring joint and/or crack faulting.
- Data sheets (and clipboard) for recording pavement distresses.
- Distress Identification Manual.
- Camera for photographing representative distress.
- Hard hats and safety vests.
- Traffic control measures (moving set up with arrow board).

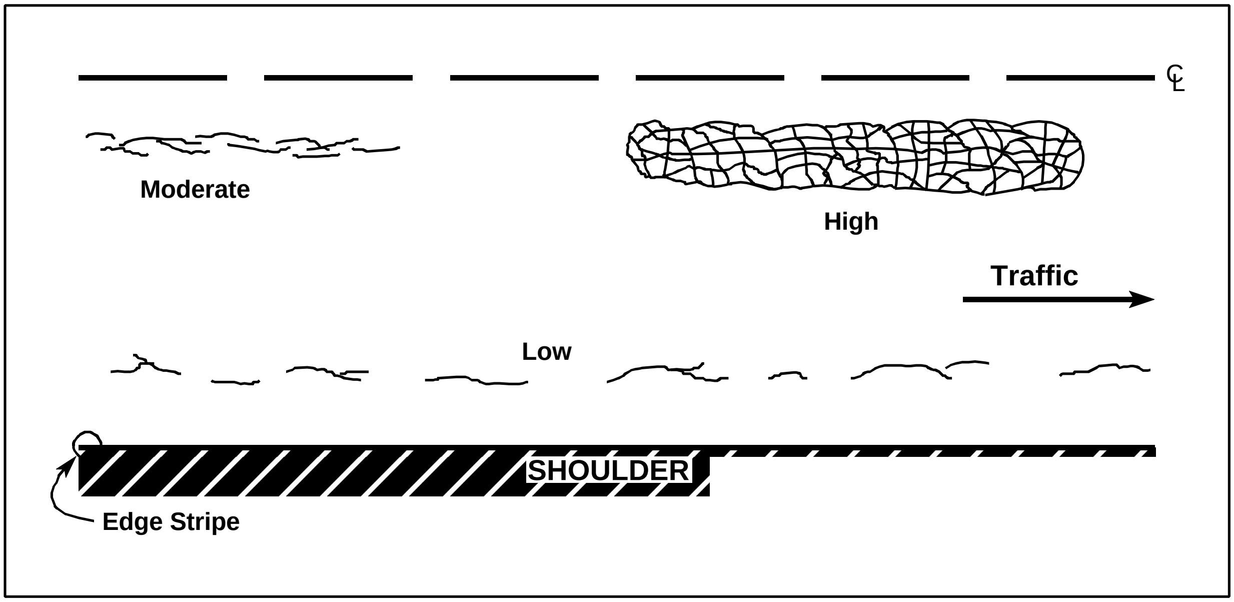


Figure 2-2.1. Example figure of fatigue cracking in asphalt concrete pavements.[2]

The distress survey described below is for the purpose of determining the condition of a specific project so that appropriate rehabilitation alternatives can be identified. Elements of a manual distress survey are provided in the following sections.

Presurvey Activities

A distress survey usually requires a two-person crew. The members of the survey crew should be thoroughly familiar with the distress identification manual in present use. Prior to the actual survey, the limits of the project should be identified. The limits generally correspond to a specific construction project, although it is possible that several projects are to be evaluated for rehabilitation. If available, information from previous surveys should be reviewed to help in the identification of distresses and where these are located.

Windshield Survey

A "windshield survey" should be conducted prior to the detailed survey. In simple terms, the entire project should be driven in each lane in both directions at posted speed limits to get an overall "feel" or impression for how the pavement is performing. During these passes, any swells, depressions, or other sources of discomfort should recorded and any locations of medium- to high-severity swells or depressions should be noted by mile post.

Detailed Distress Survey

In a manual distress survey, the survey crew walks along the shoulder of the project measuring and recording all distresses in accordance with instructions provided in the distress identification manual. In most cases, travel lanes and shoulders should be examined. If the project is on a busy road (e.g., a highway), traffic control should be implemented for the safety of the survey crew.

The data 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. Example data sheets from the SHRP Distress Identification Manual[2] for asphalt concrete pavements are shown figure 2-2.2, figure 2-2.3, and figure 2-2.4. Others can be found in reference 5 and reference 6.

Data also can be recorded through use of portable computers. These can be very convenient for reducing paperwork and, therefore, the amount of data handling. The information can later be downloaded for evaluation.

Aerial photography (on a scale of 1 cm = 6 m or greater) can also be used to obtain distress information. This information can then be used in the evaluation of the project as well as in preparation of contract documents showing where repair work should be performed.

As part of a distress survey, it is strongly recommended that photographs of typical conditions be taken during the survey. These serve as a permanent record of the pavement condition at the time of the survey and can be extremely helpful in communicating the condition of the pavement to management.

Automated Distress Survey Equipment

In recent years, tremendous advances have been made in the development of automated equipment that can be used to collect, store, and process distress data. This equipment is capable of measuring most pavement distress types at higher speeds. Other advantages of automated distress survey equipment include:[7]

- More consistent measurements.
- Increased safety.
- No disruption to traffic flow.
- Predictable productivity.
- Highly objective output.
- Increase in sample size.
- Cost savings over manual, long term.
- Less traffic control and less man hours.

Revised December 1, 1992

SHEET 1

DISTRESS SURVEY

LTPP PROGRAM

STATE ASSIGNED ID __ __ __ __

STATE CODE __ __

SHRP SECTION ID __ __ __ __

DISTRESS SURVEY FOR PAVEMENTS WITH ASPHALT CONCRETE SURFACES

DATE OF DISTRESS SURVEY (MONTH/DAY/YEAR) __ __/__ __/__ __

SURVEYORS: __ __ __, __ __ __ PHOTOS, VIDEO, OR BOTH WITH SURVEY (P, V, B) __
PAVEMENT SURFACE TEMP - BEFORE __ __ __ __°C; AFTER __ __ __ __°C

DISTRESS TYPE	SEVERITY LEVEL LOW	MODERATE	HIGH
CRACKING			
1. FATIGUE CRACKING (Square Meters)	__ __ __.__	__ __ __.__	__ __ __.__
2. BLOCK CRACKING (Square Meters)	__ __ __.__	__ __ __.__	__ __ __.__
3. EDGE CRACKING (Meters)	__ __ __.__	__ __ __.__	__ __ __.__
4. LONGITUDINAL CRACKING (Meters)			
4a. Wheel Path	__ __ __.__	__ __ __.__	__ __ __.__
Length Sealed (Meters)	__ __ __.__	__ __ __.__	__ __ __.__
4b. Non-Wheel Path	__ __ __.__	__ __ __.__	__ __ __.__
Length Sealed (Meters)	__ __ __.__	__ __ __.__	__ __ __.__
5. REFLECTION CRACKING AT JOINTS Number of Transverse Cracks	__ __ __	__ __ __	__ __ __
Transverse Cracking (Meters)	__ __ __.__	__ __ __.__	__ __ __.__
Length Sealed (Meters)	__ __ __.__	__ __ __.__	__ __ __.__
Longitudinal Cracking (Meters)	__ __ __.__	__ __ __.__	__ __ __.__
Length Sealed (Meters)	__ __ __.__	__ __ __.__	__ __ __.__
6. TRANSVERSE CRACKING Number of Cracks	__ __ __	__ __ __	__ __ __
Length (Meters)	__ __ __.__	__ __ __.__	__ __ __.__
Length Sealed (Meters)	__ __ __.__	__ __ __.__	__ __ __.__
PATCHING AND POTHOLES			
7. PATCH/PATCH DETERIORATION (Number)	__ __ __	__ __ __	__ __ __
(Square Meters)	__ __ __.__	__ __ __.__	__ __ __.__
8. Potholes (Number)	__ __ __	__ __ __	__ __ __
(Square Meters)	__ __ __.__	__ __ __.__	__ __ __.__

Figure 2-2.2. Distress survey form, sheet 1 of 3.[2]

Revised December 1, 1992

SHEET 2

DISTRESS SURVEY

LTPP PROGRAM

STATE ASSIGNED ID _ _ _ _

STATE CODE _ _

SHRP SECTION ID _ _ _ _

DATE OF DISTRESS SURVEY (MONTH/DAY/YEAR) _ _/_ _/_ _

SURVEYORS: _ _ _, _ _ _

DISTRESS SURVEY FOR PAVEMENTS WITH ASPHALT CONCRETE SURFACES (CONTINUED)

DISTRESS TYPE	SEVERITY LEVEL LOW	MODERATE	HIGH
SURFACE DEFORMATION			
9. RUTTING - REFER TO SHEET 3 FOR SPS-3 OR Form S1 from Dipstick Manual			
10. SHOVING (Number)			_ _ _
(Square Meters)			_ _ _._
SURFACE DEFECTS			
11. BLEEDING (Square Meters)	_ _ _._	_ _ _._	_ _ _._
12. POLISHED AGGREGATE (Square Meters)			_ _ _._
13. RAVELING (Square Meters)	_ _ _._	_ _ _._	_ _ _._
MISCELLANEOUS DISTRESSES			
14. LANE-TO-SHOULDER DROPOFF - REFER TO SHEET 3			
15. WATER BLEEDING AND PUMPING (Number)			_ _ _
Length of Affected Pavement (Meters)			_ _ _._
16. OTHER (Describe) ______			

Figure 2-2.3. Distress survey form, sheet 2 of 3.(2)

Revised May 29, 1992

SHEET 3

DISTRESS SURVEY

LTPP PROGRAM

STATE ASSIGNED ID __ __ __ __

STATE CODE __ __

SHRP SECTION ID __ __ __ __

DATE OF DISTRESS SURVEY (MONTH/DAY/YEAR) __ __/__ __/__ __

SURVEYORS: __ __ __, __ __ __

DISTRESS SURVEY FOR PAVEMENTS WITH ASPHALT CONCRETE SURFACES
(CONTINUED)

9. RUTTING (FOR SPS-3 SITE SURVEYS)

INNER WHEEL PATH			OUTER WHEEL PATH		
Point No.	Point Distance[1] (Meters)	Rut Depth (mm)	Point No.	Point Distance[1] (Meters)	Rut Depth (mm)
1	0.	__ __ __.	1	0.	__ __ __.
2	15.25	__ __ __.	2	15.25	__ __ __.
3	30.5	__ __ __.	3	30.5	__ __ __.
4	45.75	__ __ __.	4	45.75	__ __ __.
5	61.	__ __ __.	5	61.	__ __ __.
6	76.25	__ __ __.	6	76.25	__ __ __.
7	91.5	__ __ __.	7	91.5	__ __ __.
8	106.75	__ __ __.	8	106.75	__ __ __.
9	122.	__ __ __.	9	122.	__ __ __.
10	137.25	__ __ __.	10	137.25	__ __ __.
11	152.5	__ __ __.	11	152.5	__ __ __.

14. LANE-TO-SHOULDER DROPOFF

Point No.	Point Distance[1] Meters	Lane-to-Shoulder Dropoff (mm)
1	0.	__ __ __.
2	15.25	__ __ __.
3	30.5	__ __ __.
4	45.75	__ __ __.
5	61.	__ __ __.
6	76.25	__ __ __.
7	91.5	__ __ __.
8	106.75	__ __ __.
9	122.	__ __ __.
10	137.25	__ __ __.
11	152.5	__ __ __.

Note 1: "Point Distance" is the distance in meters from the start of the test section to the point where the measurement was made. The values shown are SI equivalents of the 50 ft spacing used in previous surveys.

Figure 2-2.4. Distress survey form, sheet 3 of 3.[2]

Many different types of equipment are currently available for automated collection of distress data. It should be noted, however, that the various types of equipment differ in measurement capabilities and sensitivity, data processing and storage procedures, format of data outputs, and cost. Thus, any agency considering the purchase or lease of automated data collection equipment is strongly encouraged to research the capabilities of each type to ensure that they match those characteristics desired by the agency. Additional information regarding the capabilities of some of the automated data collection equipment can be found in references 9 and 24.

5. ROUGHNESS SURVEY

Roughness surveys are an important part of the pavement evaluation process. They can be conducted in a subjective manner (i.e., windshield survey) or in an objective manner (i.e., with roughness measurement equipment). Regardless of the method of determining roughness, the primary purpose of performing a roughness survey for a given project is to identify areas of severe roughness. Roughness surveys are also useful in determining relative roughness between projects and in gauging the overall benefits of various rehabilitation activities by measuring roughness before and after construction.

Definition of Roughness

Roughness can be defined as the distortion of the road surface which contributes to an undesirable, unsafe, uneconomical, or uncomfortable ride.[(10)] Roughness can be divided into longitudinal variations and transverse variations, with longitudinal roughness having by far the greatest effect on undesirable vehicle forces.[(10)] Roughness can develop from surface irregularities that are built into a pavement during construction, and surface irregularities that develop after construction (due to traffic loading, climatic effects, and other factors).

By this definition, roughness can be the result of any surface irregularity, including distresses such as cracks, potholes, ruts, etc. In fact, when a “Rough Road” warning sign is posted by an agency, it is typically posted in areas with severe distresses. However, it is important to realize that a pavement section can be considered to have roughness even though surface distresses are nonexistent. In this latter case, roughness would be the result of differential elevations (e.g., swells, bumps, depressions) both along the length and across the width of the pavement section.

The type of information desired from a roughness survey should dictate the type of survey to conduct and the type of equipment to use if it is to be measured using roughness measurement equipment. If it is desired, for example, to obtain a qualitative assessment of the roughness of a pavement section, then a windshield survey should be an adequate means of obtaining this information. Conversely, if it is desired to obtain a quantitative measure of roughness, then it may be more advisable to use roughness measurement equipment to obtain the information.

Windshield Survey

A simple windshield survey of the project being evaluated can be an adequate and valid means of 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. Should this be the selected method, the entire project should be driven in each lane in both directions at posted speed limits to get an overall impression for pavement roughness. During these passes, roughness due to surface distress (e.g., raveling, shoving, rutting, potholes) and/or roughness due to differential elevations (swells, depressions, etc.) should be noted separately as the maintenance or rehabilitation activity may be different for different causes of roughness.

Equipment for Measuring Roughness

Many different types of equipment is used to measure roughness. These types of equipment can be broadly categorized as being either inertial road profiling systems (IRPS), which measure actual pavement profiles, or response-type road roughness measuring (RTRRM) systems, which measure vehicle response to pavement roughness.(11) RTRRM systems, which include such devices as the BPR Roughometer, the Mays Meter, and the PCA Roadmeter, accumulate some measure of vertical movement of a rear axle of an automobile or trailer relative to the vehicle frame.(9) Thus, RTRRM systems do not measure profile, but a dynamic effect of roughness that can be attenuated or amplified, depending on the mechanical system of the vehicle.(9)

RTRRM systems have enjoyed widespread use and have the advantage of low initial and operating costs, ease of operation, and high measuring speeds.(12) However, their disadvantages are that the response output is sensitive to the vehicle type, suspension characteristics, tire pressure, speed, and vehicle weight distribution.(12) RTRRM systems also have the disadvantage of requiring frequent calibration to ensure that reasonable and reproducible measurements are obtained.(9)

In recent years, there has been more and more movement to the use of inertial road profiling systems, IRPS, such as the K.J. Law Profilometer and the South Dakota Profiler, provide accurate, scale reproductions of the pavement profile along the path of the vehicle.(11) Most IRPS utilize non-contact sensors (optical or acoustical) that measure the relative displacement between the vehicle frame and the road surface.(11) IRPS yield highly accurate and repeatable profile measurements that can be used to compare projects to one another and can be used for the calibration of RTRRM systems. Disadvantages of IRPS include their relatively high capital and operating costs and the complexity of the system. Additional information on roughness measuring equipment can be found in references 13, 14, and particularly reference 25.

The many different types of roughness equipment in use today has made the comparison of roughness between projects very difficult. To address this problem, work was conducted at the International Road Roughness Experiment in Brazil in 1982 to standardize a summary roughness statistic.(15,16) The result was a standardized roughness measurement called the International Roughness Index (IRI) that provides a common scale of measuring roughness.(17) The IRI is a numeric scale that can be correlated to roughness measurements obtained from RTRRM systems and profilers. The scale ranges from 0 in to 1267 in/mi (0 m/km to 20 m/km), with larger values indicating greater roughness. The approximate break point between “rough” and “smooth” pavements is often considered to be 125 in/mi (2 m/km). The IRI has been adopted as a standard for the FHWA Highway Performance Monitoring System (HPMS) database.

Field Procedures

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 the roughness index for each 0.1 mi (0.16 km) 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 with time.(25)

6. PAVEMENT SURFACE FRICTION SURVEY

Pavement surface friction is the force developed at the tire-pavement interface that resists sliding when braking forces are applied to the vehicle tires.[23] While sufficient surface friction generally exists on dry pavements, the water on wet pavements act as a lubricant to reduce the direct contact between the pavement surface and the tire. If the film of water is sufficiently thick or if vehicle speeds are sufficiently high, the tires may lose contact with the pavement surface, creating a dangerous phenomenon known as hydroplaning.[23]

Surface friction is influenced by three factors: microtexture, macrotexture, and surface drainage. Microtexture is the surface "roughness" of the individual coarse aggregate particles and of the binder, and contributes to friction through adhesion with vehicle tires. Macrotexture refers to the overall texture of the pavement (controlled by coarse aggregate type and size in flexible pavements and by surface finish in rigid pavements), which is intended to serve as escape channels for the surface water at the pavement-tire interface.[23] Adequate surface drainage (i.e., cross slope) influences pavement surface friction by assisting water runoff from the pavement surface.

Surface friction can be measured in a variety of ways. The most common method is the use of a locked-wheel trailer in accordance with ASTM E 274. In this procedure, a truck carrying a water tank wets the surface ahead of a locked-wheel trailer it is towing.[6,9] The friction between the tire and pavement surface is generally measured at a speed of 40 mi/hr (64 km/hr). Another method uses yaw mode trailers (such as the Mu Meter), in which two wheels of the trailer are turned in opposite directions to create transverse forces in order to estimate skid resistance.[6] A summary of the various types of surface friction measuring equipment currently in use is given in table 2-2.2.

Table 2-2.2. Summary of surface friction equipment currently in use.[23]

TYPE OF TESTER	NO. OF TESTERS
K.J. Law Locked-Wheel Skid Trailer	38
Locked-Wheel Skid Trailer (built by agencies using them)	13
Cox & Sons Locked Wheel Skid Trailer	3
Locked Wheel Skid Trailer Meeting AASHTO Specification	2
FMC Locked-Wheel Skid Trailer	2
Soiltest Locked-Wheel Skid Trailer	3
Mu Meter	4
British Pendulum Tester	1
Other	6

Some agencies prefer to use blank (smooth) tires instead of a ribbed tire in their testing program. This is because measurements using the blank tire are reportedly better indicators of the pavement's macrotexture.[23]

Some of the surface conditions that are indicative of potential surface friction problems are:

- Asphalt bleeding, where a film of asphalt collects on the pavement surface.
- Rutting of asphalt pavements, resulting in the collection of water in the wheelpaths.
- Smooth macrotexture that may be the result of polishing of the aggregate (flexible or rigid pavements) and inadequate finishing (rigid pavements).
- Inadequate pavement cross slopes that result in slow runoff of water from the pavement surface.

Poor friction conditions, such as those listed above, may be the primary justification for rehabilitation if the wet weather accident rate is high enough. The friction number should be measured at uniform increments along the project in each traffic lane. 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. There should be a reasonable correlation between friction results with accident rates.

The results obtained from the friction tests should be used in the selection of feasible rehabilitation alternatives. For example, if a pavement has adequate friction properties, then rehabilitation alternatives without overlays or reconstruction may be feasible. If the friction properties are inadequate, then rehabilitation alternatives (overlays, grooving, grinding) should be examined that address this inadequacy.

7. EVALUATION OF SURVEY RESULTS

Serviceability

The concept of serviceability was developed at the AASHO Road Test that was conducted in the late 1950s.[19,20] As part of that study, a panel of raters was driven over the various test sections and asked to rate the pavement using the following scale:

0-1	Very poor
1-2	Poor
2-3	Fair
3-4	Good
4-5	Very good

The average rating of all of the raters, termed the Present Serviceability Rating (PSR), was an indication of how well the pavement was performing from a user's point of view. The PSR was used to track the performance of a pavement over time, and was used to indicate points in time when a pavement became too rough and was in need of rehabilitation. 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 design and for overlay design.

Efforts were conducted at the AASHO Road Test to develop indicators of PSR that would eliminate the need for a large number of raters. The result was the Present Serviceability Index (PSI), which is an estimate of the highway user's subjective assessment (PSR) of the pavement condition. At the AASHO Road Test, the PSI was a computed number obtained from a regression equation containing either roughness alone or roughness and distress. The original PSI equations developed at the AASHO Road Test are shown below:[19,20]

Flexible Pavements $$PSI = 5.03 - 1.91*\log(1 + SV) - 1.38*(RD)^2 - 0.01*(C + P)^{0.5} \qquad (2\text{-}2.1)$$

Rigid Pavements $PSI = 5.41 - 1.80*\log(1 + SV) - 0.09*(C + P)^{0.5}$ (2-2.2)

where:

SV	=	Average slope variance over pavement section, x 10^6
RD	=	Mean rut depth, in
C	=	Class 2 + Class 3 alligator cracking, $ft^2/1000\ ft^2$ (flexible pavements)
	=	Class 2 and sealed cracks, $ft/1000\ ft^2$ (rigid pavements)
P	=	Patching, $ft^2/1000\ ft^2$

These equations were developed from the AASHO Road Test in 1958 and may not be applicable for an individual State today. Each State should develop its own equations.

The slope variance term is the output of the CHLOE profilometer, which was used to measure the profile of the sections at the AASHO Road Test. Many agencies have converted the slope variance term to a roughness index, as measured with another roughness measuring device, so that the roughness measurements can be obtained easily. Other agencies have taken the process a step further and have simply correlated the PSI directly with the roughness measurements. This is because the roughness terms in equation 2-2.1 and equation 2-2.2 dominate and the distress terms do not contribute significantly to the accuracy of the equations. This is seen in figure 2-2.5, which shows a relationship between roughness and panel rating. Other work has been done to relate the serviceability index to IRI< as shown by the equation below:[21]

$$PSI = 5 * e^{(-0.0041*IRI)} \qquad (2\text{-}2.3)$$

where:

PSI	=	Present Serviceability Index
e	=	2.7182 (inverse natural logarithm)
IRI	=	International Roughness Index, in/mi

While PSI only provides an indication of the user's opinion of the pavement condition, it is still a valuable indicator that should be considered when evaluating a pavement. The PSI could be computed at various increments along the project (say, at 0.1 mi [0.16 km] increments) in each traffic lane. Based only on user response, the following values of PSI are generally considered critical levels triggering a need for some sort of rehabilitation:[22]

Traffic Volume	Critical PSI
High (> 10,000 ADT)	3.0 - 3.5
Medium (3,000 - 10,000 ADT)	2.5 - 3.0
Low (< 3,000 ADT)	2.0 - 2.5

The correlation of the rating panel of highway users with roughness can provide an adequate methodology by which to compute the PSI. However, it must be emphasized that the PSI is only an estimator of the user response. It is not necessarily the same rating that a maintenance engineer or pavement engineer would give the pavement.

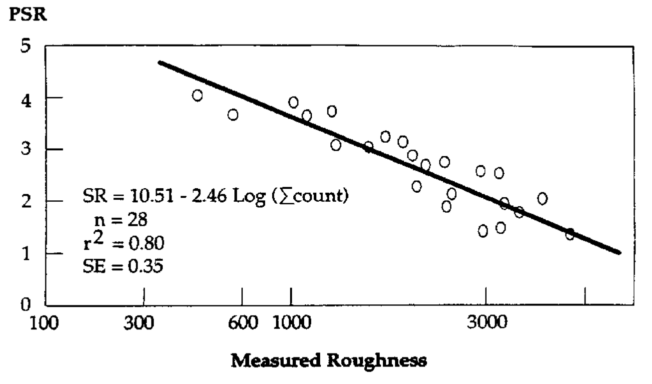


Figure 2-2.5 Example correlation between PSR and PCA Roughometer.

While the PSI concept is useful for determining when rehabilitation is required from a user's viewpoint, it should not serve as the only factor triggering the need for rehabilitation. The entire pavement must be throughly and completely examined in order to identify the causes of deterioration. For example, a pavement may be relatively smooth and still possess certain types of distress that indicate significant deterioration. A complete pavement evaluation, including an evaluation of a pavement's serviceability, is needed to accurately assess its condition.

Presentation of Results

The condition survey data should be summarized so that a clear picture of the existing condition can be obtained by those involved in making design decisions. One of the most useful ways is to prepare a strip chart that shows the various condition deficiencies along the project. These should include the location and severity of surface distress, roughness, and surface friction along the project by lane and shoulder and, because it immediately conveys the time and extent of condition deficiency throughout a project, is highly recommended. Other pavement evaluation information, such as deflections, soil types, etc. can be added to the strip charts to provide a complete picture of the pavement condition. A strip chart will also aid in the selection of sites for further detailed materials and pavement testing.

The information obtained from the condition surveys can be used in many ways. Some of these uses 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 rehabilitation project, it is generally recommended that a follow-up survey be conducted just prior to advertisement 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. For example, a change in subgrade, traffic, or materials may result in a significant change in pavement performance (and hence the occurrence of distress). In addition, the inner lanes of multilane facilities may exhibit significantly less distress or lower severity levels of distress than the outer lane. By recognizing these trends, rehabilitation designs can be varied along the project and/or across lanes to reduce costs.

3. The condition survey data provides permanent documentation of the condition of the existing pavement. This lends itself to several uses, including a comparison of pavement performance before and after rehabilitation, and also the development of performance prediction models and curves.

4. The data provides an excellent source of information with which to plan the structural and subgrade testing of the pavement (module 2-3 and module 2-4). It is also useful in the evaluation of the pavement's drainability (module 2-5).

5. The data provides valuable insight into the mechanisms of pavement deterioration. As a first step, the deficiency can be identified as being either primarily load-associated or primarily climate/materials-associated.

   If the distress is primarily load-associated, rehabilitation work should include a structural improvement. If climatic conditions or paving materials are contributing to the deterioration, appropriate measures should be identified to address those deficiencies or to lessen their impact or effect on pavement performance. If serious climatic- or materials-related problems exist (e.g., severe D-cracking condition, severe raveling, etc.), the best solution may be complete reconstruction.

6. If time-series condition data is 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 rehabilitation actions (e.g., determining if a certain pavement can wait 3 years for an overlay or whether it then will be too deteriorated).

8. SUMMARY

Information on performing project distress surveys, roughness surveys, and surface friction evaluations is presented in this module. Methods for performing each of the surveys are discussed, and the types and capabilities of equipment for each survey are also described. Uses of the evaluation data, and how they can be used in the evaluation of a project, are provided.

The distress survey is the first part of a project evaluation. It can be performed either manually or by automated distress data collection equipment and can be evaluated to provide the following information:

- Estimates of needed repair locations and quantities.
- Significant changes in condition along the project.
- Documentation of the condition of the existing pavement.
- Additional testing needs.
- Causes and mechanisms of pavement deterioration.

The importance of roughness data and its role in a pavement evaluation is also discussed. Major types of roughness measuring equipment are described. Major uses of roughness survey include:

- Determining the need for pavement improvements from the user's perspective.
- Identifying areas where severe roughness exists so that corrective 4R measures can be taken.
- Recording roughness measured before and after construction, so that the benefits and quality of the contractor's work can be measured and specifications can be developed for future projects.

Pavement surface friction is defined, along with the various components that make it up: microtexture, macrotexture, and surface drainage (cross slope). Methods for the determination of pavement surface friction are described and potential uses of the data presented.

The Present Serviceability Index (PSI) is an estimator of the highway user's perception of a pavement on a scale of 0 (impassable) to 5 (perfect). It is best determined by correlating the ratings of a large group of highway users with the measured roughness. A regression equation relating measured roughness can be easily derived by an agency for its local conditions.

9. REFERENCES

1. Smith, R.E., M.I. Darter, and Herrin, S.M., "Highway Pavement Distress Identification Manual," FHWA-RD-79-66, Federal Highway Administration, 1979.

2. "Distress Identification Manual for the Long-Term Pavement Performance Project," SHRP-P-338, Strategic Highway Research Program, Washington, DC, May 1993.

3. Shahin, M.Y. and J.A. Walther, "Pavement Maintenance Management for Roads and Streets Using the PAVER System," USACERL Technical Report M-90/05, July 1990.

4. "Guidelines and procedures for Maintenance of Airport Pavements," Advisory Circular 150/5380-6, Federal Aviation Administration, 1982.

5. Goulias, D.G., H. Castedo, and W.R. Hudson, "Pavement Distress Surveys in the Strategic Highway Research Program's Long-Term Pavement Performance Study," Transportation Research Record 1260, Transportation Research Board, 1990.

6. Hicks, R.G. and J.P. Mahoney, "Collection and Use of Pavement Condition Data," Synthesis of Highway Practice 76, Transportation Research Board, 1981.

7. Sime, J.M. and D.A. Larsen, "Objective Versus Subjective Pavement-Distress Evaluation Systems," FHWA-TS-90-053, Proceedings, Automated Pavement Distress Data Collection Equipment Seminar, Federal Highway Administration, October 1990.

8. Cable, J.K. and V.J. Marks, "Automated Pavement Distress Data Collection Seminar, " FHWA-TS-90-053, Proceedings, Automated Pavement Distress Data Collection Equipment Seminar, Federal Highway Administration, October 1990.

9. Epps, J.A. and C.L. Monismith, "Equipment for Obtaining Pavement Condition and Traffic Loading Data," Synthesis of Highway Practice 126, Transportation Research Board, 1986.

10. Hudson, W.R., "Road Roughness: Its Elements and Measurements," Transportation Research Record 836, Transportation Research Board, 1981.

11. Vorburger, T.F., D.C. Robinson, S.E. Fick, and D.R. Flynn, "Calibration of Road Roughness Measuring Equipment, Volume I—Experimental Investigation," FHWA-RD-89-077, Federal Highway Administration, March 1989.

12. Gulden, W., "Calibration Procedures for Roadmeters," FHWA-TS-86-201, Federal Highway Administration, April 1986.

13. Gillespie, T.D., M.W. Sayers, and L. Segal, "Calibration of Response-Type Road Roughness Measurement Systems," NCHRP Report 228, Transportation Research Board, 1980.

14. Janoff, M.S., “Pavement Roughness and Rideability Field Evaluation,” NCHRP Report 308, Transportation Research Board, 1988.

15. M.W. Sayers, T.D. Gillespie, and C.A.V. Queiroz, “The International Road Roughness Experiment, Establishing Correlation and a Calibration Standard for Measurements,” World Bank Technical Paper 45, The World Bank, 1986.

16. M.W. Sayers, T.D. Gillespie, and W.O. Patterson, “Guidelines for Conducting and Calibrating Road Roughness Measurements,” World Bank Technical Paper 46, The World Bank, 1986.

17. Sayers, M.W., “Profiles of Roughness,” Transportation Research Record 1260, Transportation Research Board, 1990.

18. “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials, 1988.

19. Carey, W.N. and P.E. Irick, “The Pavement Serviceability-Performance Concept,” Highway Research Bulletin No. 250, Highway Research Board, 1960.

20. “The AASHO Road Test, Report 5, Pavement Research,” Special Report 61E, Highway Research Board, 1962.

21. Al-Omari, B. And M.I. Darter, “Relationships Between IRI ad PSR,” UILU-ENG-92-2013, Illinois Department of Transportation, September 1992.

22. “AASHTO Guide for Design of Pavement Structures,” American Association of State Highway and Transportation Officials, 1986.

23. Dahir, S.H.M. and W.L. Gramling, “Wet-Pavement Safety Programs,” Synthesis of Highway Practice 158, Transportation Research Board, 1990.

24. R.E. Smith, T.J. Freeman, and O.J. Pendelton, “Evaluation of Automated Pavement Distress Data Collection Procedures for Local Agency Pavement Management,” Washington State Department of Transportation Research Report, July, 1996, Olympia, WA.

25. M.W. Sayers, S.M. Karamihas, “The Little Book of Profiling,” FHWA NHI Special Course on Road Roughness, available through University of Michigan Transportation Research Institute, Road Roughness Home Page, October 1996.

MODULE 2-3
NONDESTRUCTIVE TEST DATA COLLECTION AND INTERPRETATION

1. INSTRUCTIONAL OBJECTIVES

This module presents the concepts and procedures for conducting nondestructive testing on an existing pavement, primarily through the use of modern deflection testing equipment. It also describes the procedures for processing and interpreting the data, including the so-called backcalculation techniques. The participant should be able to accomplish the following upon completion of this module:

1. Describe the nature of a pavement's response to load.
2. List some of the available pavement deflection measuring devices and their operating characteristics.
3. List the major factors that influence deflections in flexible and rigid pavements.
4. Describe procedures for conducting a nondestructive testing (NDT) program on a typical highway project.
5. Describe the effects of seasonal conditions on NDT deflection results.
6. Describe the basic principles and procedures for characterizing in situ pavement layer material properties (specifically, elastic or Young's modulus) through "backcalculation."

This module provides the primary basis for the discussion of structural evaluation in module 2-7.

2. DEFINITIONS

Nondestructive Testing. In general, NDT, refers to a wide variety of in situ tests that can be executed on any structure in which no physical damage is induced. In this module, NDT will refer to a more specific class of in situ pavement testing that is intended to quantify its ability to support traffic loading.

Deflection. This refers to a measurable response of any pavement as a result of some applied surface load. Specifically, it refers to the vertical displacement of the pavement surface in response to some externally applied simulated wheel load.

3. INTRODUCTION

Nondestructive testing of an existing pavement is an extremely valuable engineering tool in assessing its uniformity and structural adequacy. The modern types of equipment available for measuring the response of the pavement under a simulated wheel loading condition are:

- Useful. The data can be used to 1) identify subsections within a project, 2) identify locations for destructive sampling and testing, 3) estimate fundamental engineering properties of the individual pavement layers, and 4) provide a rational basis for structural capacity assessment.
- Extremely productive. They are capable of collecting on the order of 200 to 400 measurements per day.
- Repeatable. They measure essentially the same value if the same conditions prevail.

Some states operate nondestructive test equipment as part of their network and project level evaluation needs. The emphasis of this module will be on project level application.

4. PAVEMENT DEFLECTIONS

The deflection of a pavement represents an overall "system response" of the paving layers 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 2-3.1.

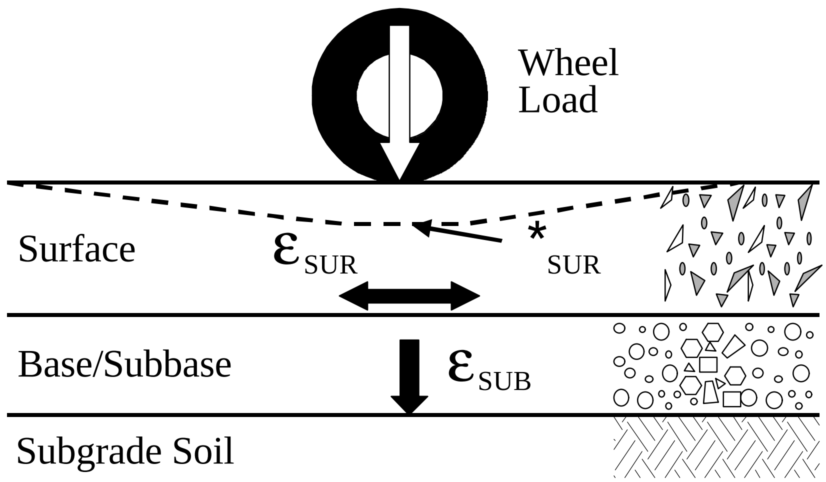


Figure 2-3.1. Illustration of strains and deflections caused by moving wheel loads.

Figure 2-3.2 and figure 2-3.3 show load-induced deflections, strains, and stresses for typical "weak" and "strong" pavement sections. As these figures illustrate, weaker pavements develop higher tensile strains/stresses in the surface layer and higher compressive strains/stresses in the subgrade soil than do stronger pavements. The way a pavement section responds to loading has a significant effect on its performance. The stronger pavement section will support far more heavy traffic loadings than will the weaker pavement section.

5. TYPES OF NDT EQUIPMENT

The four general classes of NDT deflection equipment currently available are: static load deflection equipment, steady-state vibratory load deflection equipment, impulse load deflection equipment, and surface wave propagation equipment.[3]. The key factors that must be considered in the selection of an NDT device are:[4]

- Operational characteristics (data collection and recording ability, traffic delay, calibration requirements, transportability, and crew training requirements).
- Data quality (suitability, repeatability, and accuracy).
- Versatility (number of sensors, sensor configuration and movability, and range of load levels).
- Cost (capital cost and cost per test sequence).

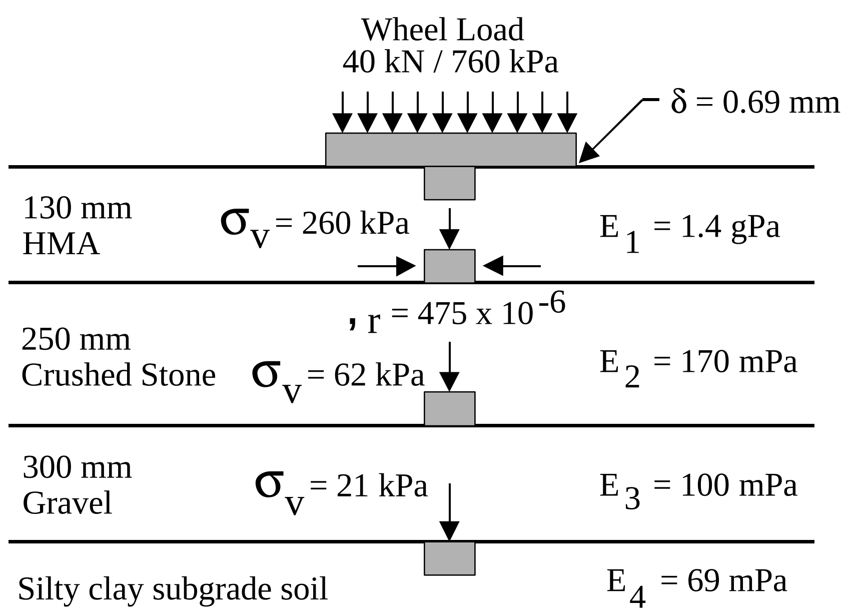


Figure 2-3.2. Typical tensile strain and compressive subgrade soil stress in "strong" pavement section.

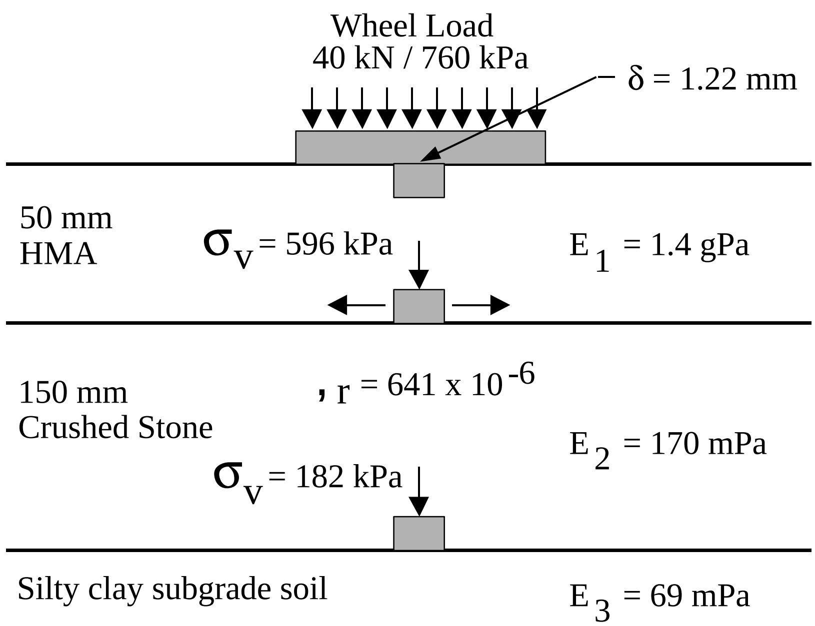
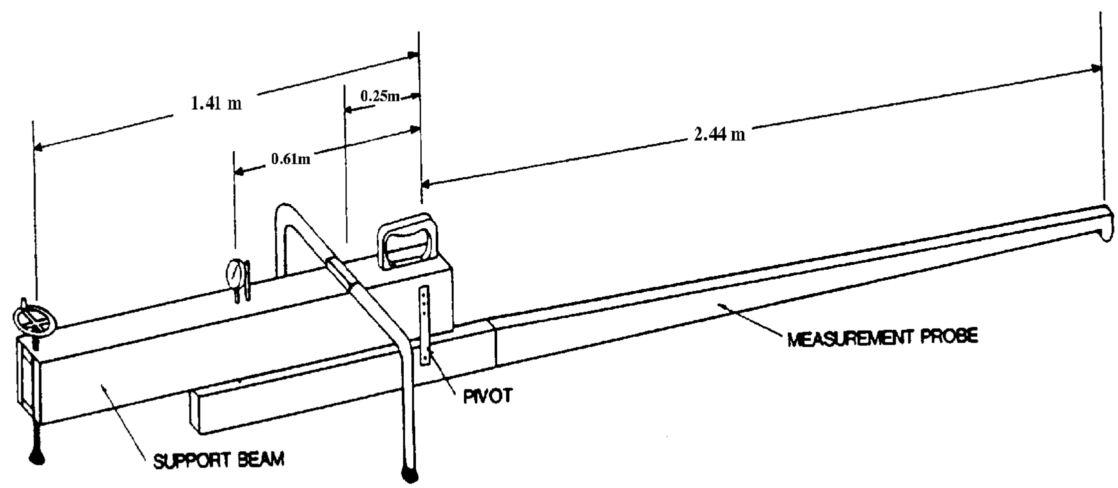


Figure 2-3.3. Typical tensile strain and compressive subgrade soil stress in “weak” pavement section.

Static Load Deflection Equipment

Static load deflection equipment measure the maximum deflection response of a pavement to static or slowly applied loads. The most commonly used static deflection device is the Benkelman Beam. Figure 2-3.4 is a sketch of the basic components of the Benkelman Beam. The deflection measurements are made using either of the standard procedures outlined in reference 5 and reference 6. Other static deflection devices used include the Plate Bearing Test Equipment and the Curvature Meter, which are described in reference 7, reference 8, and reference 9. In some cases, the static load deflection equipment are automated, providing a slowly moving load. Examples of automated static deflection equipment are the La Croix Deflectograph and the California Traveling Deflectometer. Reference 8 and reference 9 provide further descriptions of those devices.

Figure 2-3.4. Sketch of basic components of the Benkelman Beam.



Advantages of the Benkelman Beam include ease of use, low equipment cost, and the existence of a large database from its use over many years. The major technical problems associated with the Benkelman Beam include:

- The difficulty of ensuring that the front supports are not in the deflection basin.
- The difficulty or inability to determine the shape and size of the deflection basin.
- Poor repeatability of measurements obtained by the device.
- The labor intensive and cumbersome nature of the device.

In addition to the problems with the Benkelman Beam noted above, a major technical problem associated with the static load deflection devices is the method of load application. The static or quasi-static loading employed does not accurately represent the effects of a moving wheel load. Also, the devices cannot be easily used to determine load transfer across a joint or crack in rigid or composite pavements.

Steady-State Dynamic 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. This is illustrated in figure 2-3.5. To ensure that the device does not bounce off the pavement surface, the magnitude of the peak-to-peak dynamic force (high to low) must be less than twice the static preload. Consequently, the static preload must be increased as the dynamic peak-to-peak load is increased.

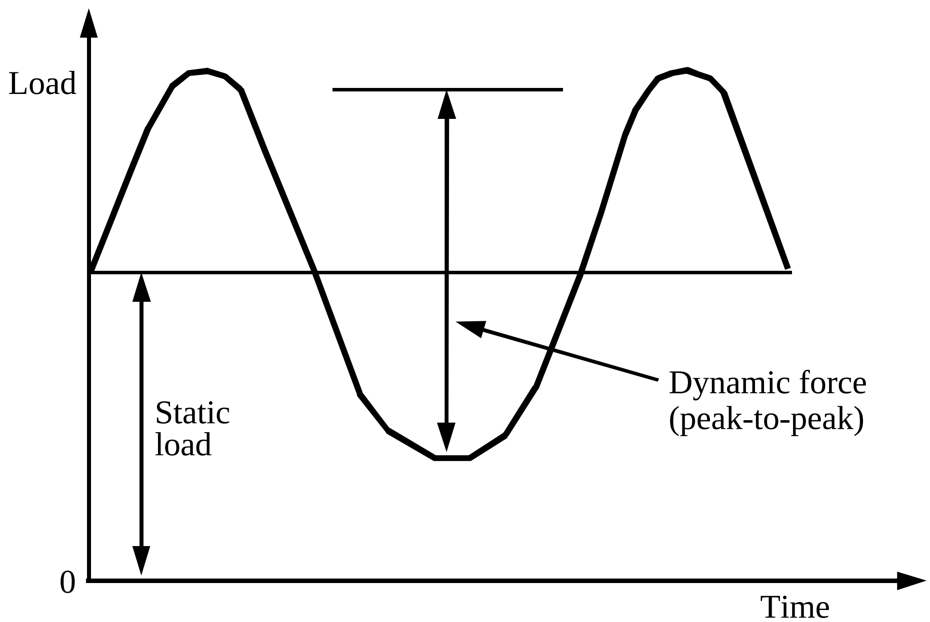


Figure 2-3.5. Typical output of vibrating steady-state force generator.

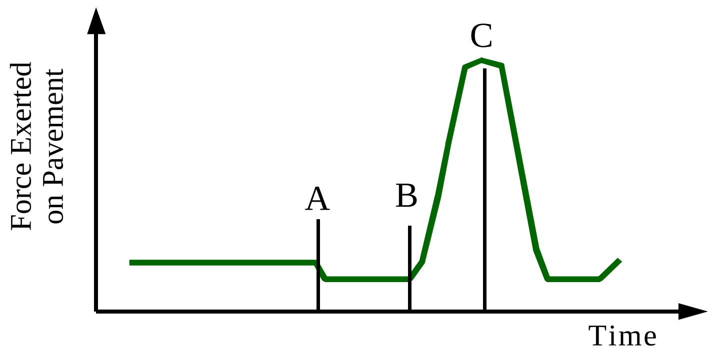
The most commonly-used devices in this category are the Dynaflect and Road Rater. The normal sequence of operation is to move the device to the test point and hydraulically lower the loading wheels and transducers to the pavement surface. A test is run, the data recorded, and the device moved to the next site.

In general, three kinds of devices are rapid, repeatable, and robust. In addition, they record the entire deflection basin, provide for the measurement of load transfer efficiency and, in some cases, likelihood of voids.

While steady-state vibratory load equipment are an improvement over static load deflection equipment (in that a reference point is not needed), the static preload still presents a technical problem. The static preload in most cases is relatively large in comparison to the maximum peak-to-peak loading. Since most paving materials are stress-sensitive (fine-grained soils exhibit stress softening and coarse-grained materials exhibit stress hardening), their stress states and stiffness may be modified by the static preload. In addition, the frequency of loading affects the resultant deflection, and it is difficult to establish a load frequency that matches that of moving vehicles.

Impulse Load Deflection Equipment

Impulse load deflection equipment deliver a transient impulse force to the pavement. A weight is lifted to a given height on a guide system and then dropped onto a buffering plate on the pavement. A transient impulse force, which can be changed by varying the magnitude of the falling weight or by varying the drop height, is generated in the pavement by the impact of the falling weight. This is illustrated in figure 2-3.6. Commercial impulse load deflection devices include the Dynatest, KUAB, JILS, and Phonix falling weight deflectometers (FWDs).



A - Time at which load is released
B - Time at which load makes first contact with load plate
C - Peak load reached

Figure 2-3.6. Typical load pulse produced by falling weight deflectometer.

In the normal sequence of operation, the device is moved to a test point and the loading plate and transducers are hydraulically lowered to the pavement. A test sequence is then completed for the selected weight set and number of drops from selected heights. The loading plate and sensors are then hydraulically lifted and the device is moved to the next test site. There has been some development of rolling wheel deflectometers but these are still at the prototype stage.

A major advantage of the impulse loading devices is their ability to more accurately model a moving wheel load in both magnitude and duration, thus producing a deflection that simulates the pavement deflection caused by a moving vehicle. In addition, as illustrated by figure 2-3.6, a relatively small preload is applied in comparison to the impulse load generated. This preload varies with the equipment used, but is usually in the range of 8 percent to 18 percent of the maximum impulse load, prior to the release of the weights. During the period when the weights are dropping, this preload normally falls to the range of 5 percent to 14 percent of the maximum impulse load.

The advantages that make FWD equipment the structural evaluation equipment of choice include:

- The ability to better simulate a moving axle load.
- The ability to measure deflections at varying loads over time.

- The ability to measure load transfer efficiency and the presence of voids.
- The ability to record a deflection basin.
- The speed, repeatability, and robustness of the equipment.

Surface Wave Propagation

An alternate means of nondestructive testing is through the spectral-analysis-of-surface-waves (SASW) method. This method is based on the theory of stress waves propagating in an elastic media. If a pavement structure is subjected to a vertical impact, surface waves propagate downward and away from the location of the vertical impact. The surface waves are recorded by vertical receivers placed at predetermined distances (based on the wavelength of the test) from the point of impact. Since the velocity of the propagation is a direct indicator of the stiffness of the material, the elastic modulus of the paving layers can be backcalculated by using relationships based on the theory of elasticity.

While SASW methods are not new (procedures for conducting the analysis were introduced in the 1950s), until recently they had seen limited application in the pavements area. However, they have seen widespread use in the geotechnical area. Several recent field demonstrations of equipment and procedures for analyzing highway pavements with SASW are reported in references 10 through 14. An automated device known as the seismic pavement analyzer (SPA) has been developed [36,37] and is being tested in Texas.

5. FACTORS THAT INFLUENCE MEASURED DEFLECTIONS

There are many factors which influence measured pavement deflections. This makes the interpretation of deflection results difficult, especially when the ultimate impact may be on the design overlay thickness. In this section, three primary factors that influence pavement deflection are discussed: load, pavement, and climate.

Load Factors

Ideally, the measured deflection should simulate the deflection that occurs under design load conditions (e.g., 40 kN wheel load). The type and magnitude of the load influences the deflection response of the pavement. As the load increases, the pavement deflection will also increase. However, this will not normally be a linear relationship, since most subgrade soil and granular materials are stress dependent, as discussed in module 2-4. The non-linearity of the load-deflection relationship is shown in figure 2-3.7. In figure 2-3.7, the extrapolated deflection at 40 kN based on a 4.4 kN load is significantly less than the deflection obtained using a 40 kN load. It is strongly recommended that for facilities with considerable truck traffic, the NDT device should produce loads that closely approximate those of heavy trucks. The FWD deflection equipment most closely simulates this deflection. Using this type of equipment eliminates a major problem caused by extrapolating heavy load responses from the application of light loads.

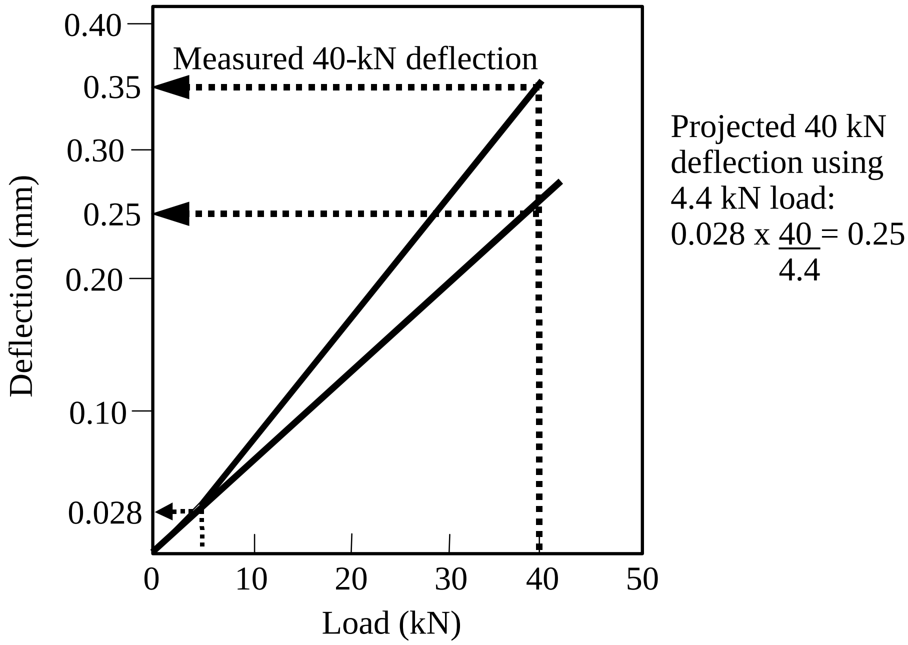


Figure 2-3.7. Pavement deflection as a function of dynamic load.

Several agencies have developed "correlations" between a light load deflection device and a heavy load deflection device (see figure 2-3.8). These are used to convert deflections measured with a light load device into "equivalent" deflections for heavier loads more closely related to design loads. However, great caution should be used in applying such correlations due to the following reasons:

- The data from which these relationships are developed typically contain a large amount of scatter. Thus, the correlation could lead to a large error.
- The correlations made for one type of pavement/subgrade soil structure may not be applicable to a different structure.

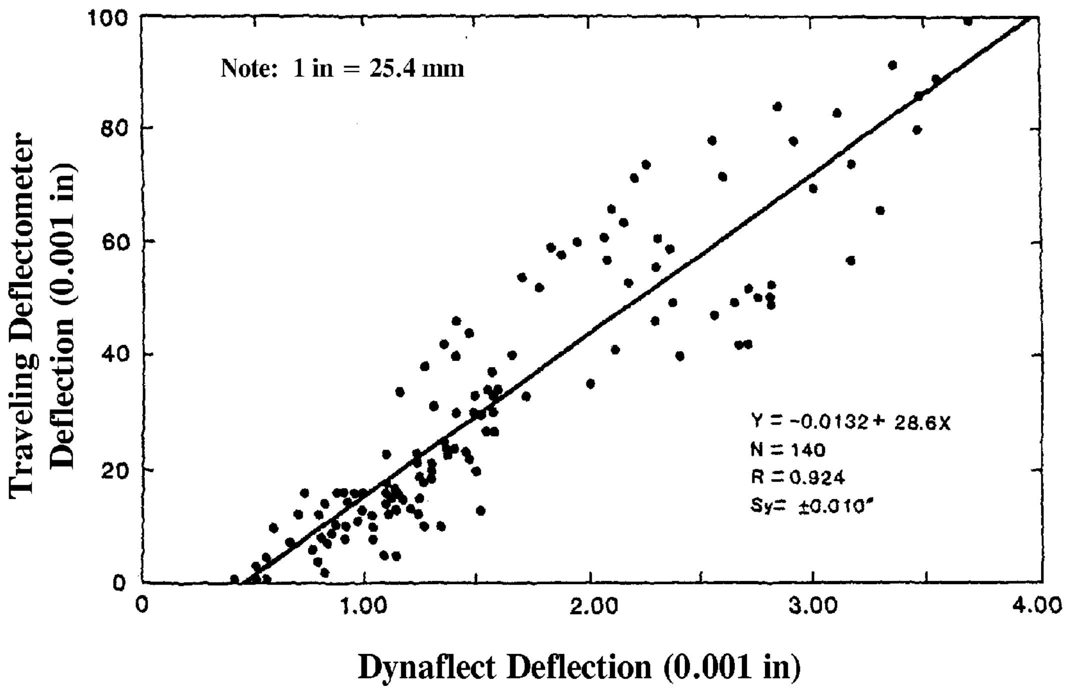


Figure 2-3.8. Correlation between light load (Dynaflect) and heavy load (Traveling Deflectometer) NDT equipment.[44]

Even when the load magnitude applied by several different types of devices is equal, the deflection response may differ, as the inherent differences in load types tends to produce different pavement responses. The duration of loading associated with an NDT device causes the deflections to vary such that the shorter the load pulse (faster the vehicle), the smaller the deflections. For example, static load devices tend to produce deflections significantly larger than those produced by moving wheel loads. The response of steady-state dynamic deflection devices vary with the load frequency, as shown in figure 2-3.9. Impulse load deflection devices produce surface deflections that most closely simulate the deflections produced by a moving wheel load. Reference 8 describes the equipment differences that cause the variation in pavement responses.

Pavement Factors

Variations in pavement deflection may be due to many pavement factors. Examples of situations that cause deflection variation follow:

- Deflection measurements on flexible pavements in or near distressed areas (e.g., alligator cracking, linear cracks), will normally be much higher than measurements in non-distressed areas, as illustrated in figure 2-3.10.

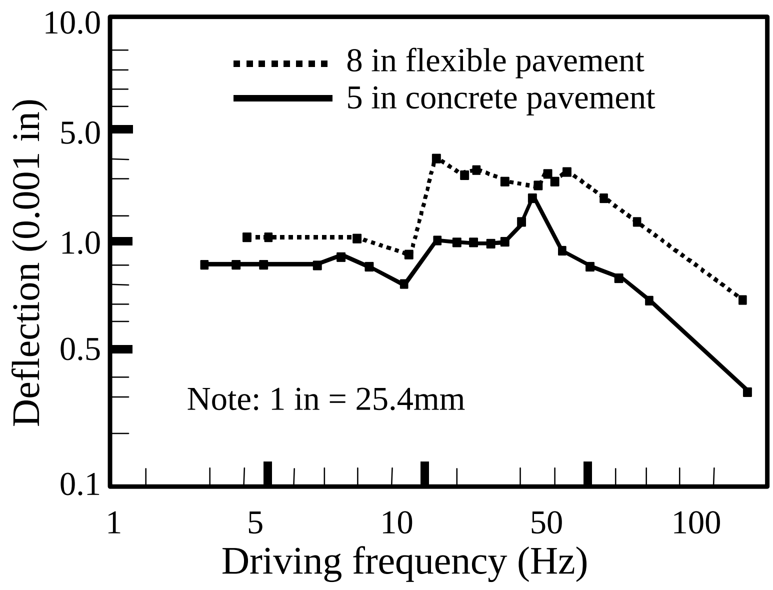


Figure 2-3.9. Variation of deflection with frequency of loading. (15)

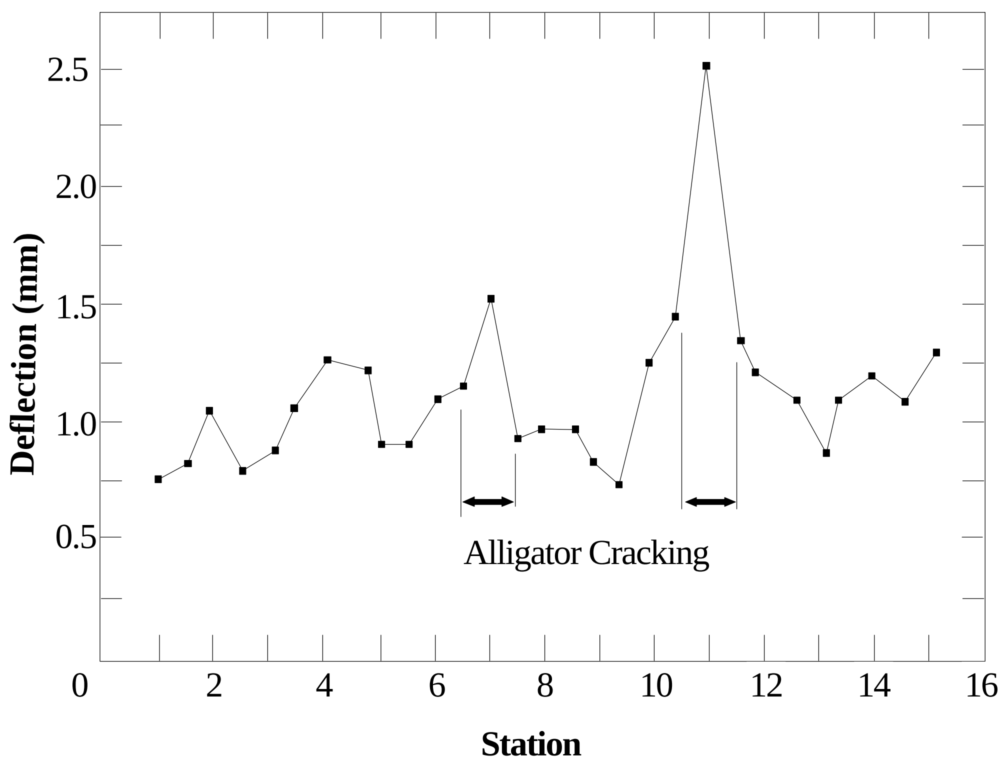


Figure 2-3.10. Effect of alligator cracking on deflections in a flexible pavement (40kN load).

- Deflection measurements in wheelpaths on asphalt pavements are typically higher than measurements between wheelpaths.

- Deflection measurements near longitudinal joints, transverse joints, or corners will be higher than those obtained at mid-slab for rigid pavements.

- Cut sections, sections at grade, and fill sections may show significantly affect deflections (figure 2-3.11), as may the presence of underlying hard (bedrock) layers (especially flexible pavements).

- Voids beneath portland cement concrete (PCC) slabs, edges, and corners will cause increased deflections.

- Random variations in pavement stiffness/strength caused by variation in factors such as compaction, material properties, and water content will result in a high variation in deflection along a typical project, even at close intervals (e.g., 3 m). The coefficient of variation for deflection measurements along a project is typically at least 20 percent to 30 percent.

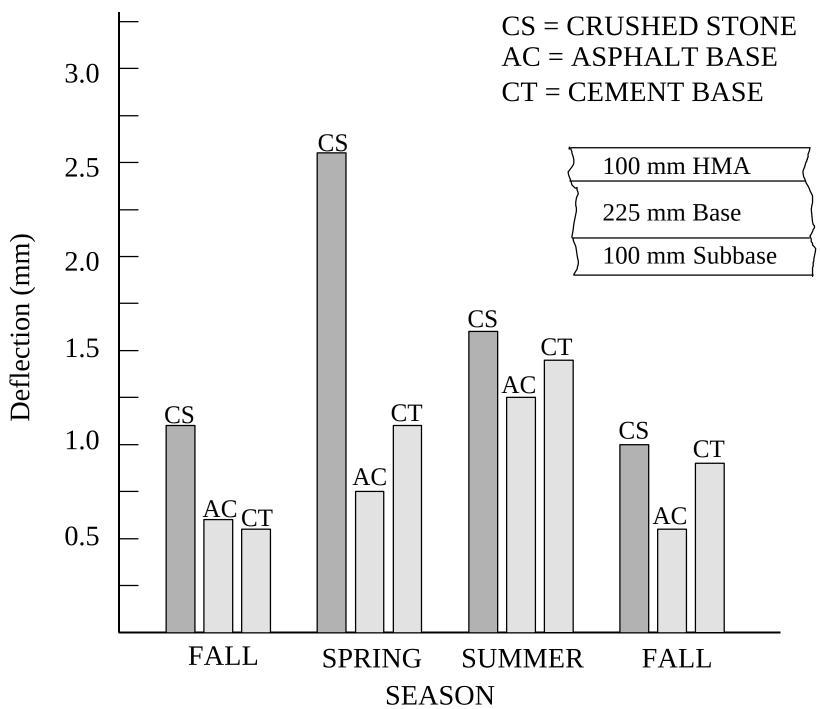


Figure 2-3.11. Pavement deflection as a function of base type and season (AASHO Road Test).[16]

Climatic Factors

There are several climatic factors that may affect deflections on a daily or a seasonal basis, and must be considered in a project deflection survey. For example, temperature greatly affects flexible pavement deflections. As the mean hot-mix asphalt (HMA) layer temperature increases, the deflection of a flexible pavement increases, as illustrated in figure 2-3.12. This is a consequence of asphalt's softening at high temperatures, with a resulting decrease in HMA modulus. Temperature also affects rigid pavement deflections near joints and cracks. In warmer temperatures, the slab expands, causing the joints and cracks to tighten and produce higher joint or crack load transfer efficiency, and lower deflections. Deflection measurements taken during such periods can be misleading.

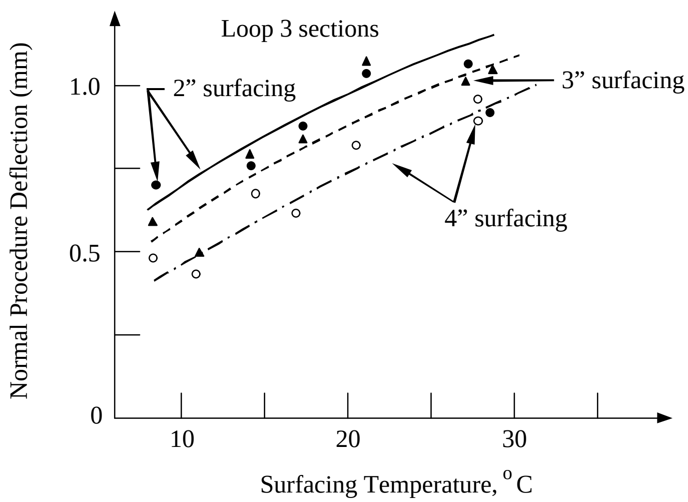


Figure 2-3.12. Influence of temperature on flexible pavement deflection.[19]

Thermal and moisture gradients lead to curling and warping, respectively, and have a significant influence on deflection measurements near joints and cracks of rigid pavements. Measurements taken at night or in the early morning when the top of the slab is typically colder than the bottom, will result in higher corner and edge deflections compared to measurements taken in the afternoon when the top of the slab is warmer than the bottom.

The season has a great effect on flexible pavement deflections in some climatic areas, as illustrated in figure 2-3.13. There are four distinct seasons in cold climatic areas:[17]

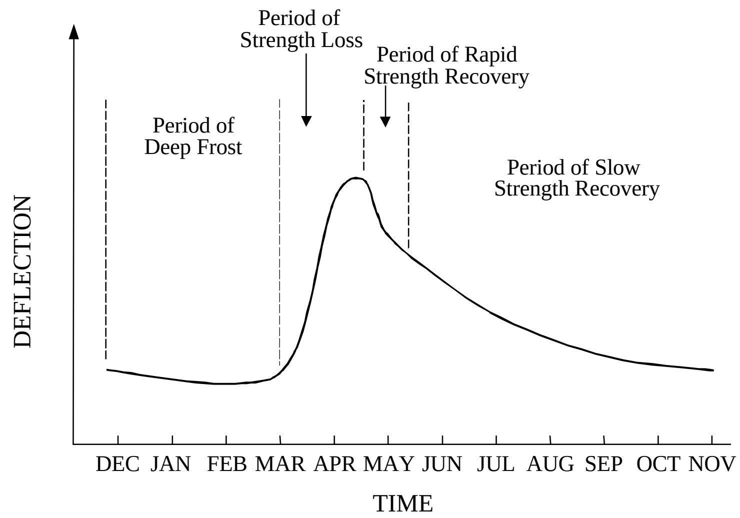


Figure 2-3.13. Influence of season on pavement deflection.[8]

- A period of deep frost when the pavement is the strongest and the measured deflection is the lowest.

- A thaw period during which the frost begins to disappear from the pavement/subgrade soil system and the deflection increases greatly.

- A period during which the excess free water from the melting frost leaves the pavement/subgrade soil structure, the soil begins to recover, and the deflection decreases.

- A period during which the deflection levels off as water content slowly decreases.

Subgrade soil type may also interact with seasonal climatic factors to affect deflections, as shown in figure 2-3.14.

For pavements in areas that do not experience freeze-thaw, deflections follow more of a flat sine curve, with the peak deflection occurring in the wet season where significant free moisture exists. In relatively dry areas, the period of maximum deflection on flexible pavements may occur in the hot summer when the asphalt surface softens due to high temperatures and intense solar radiation.[18]

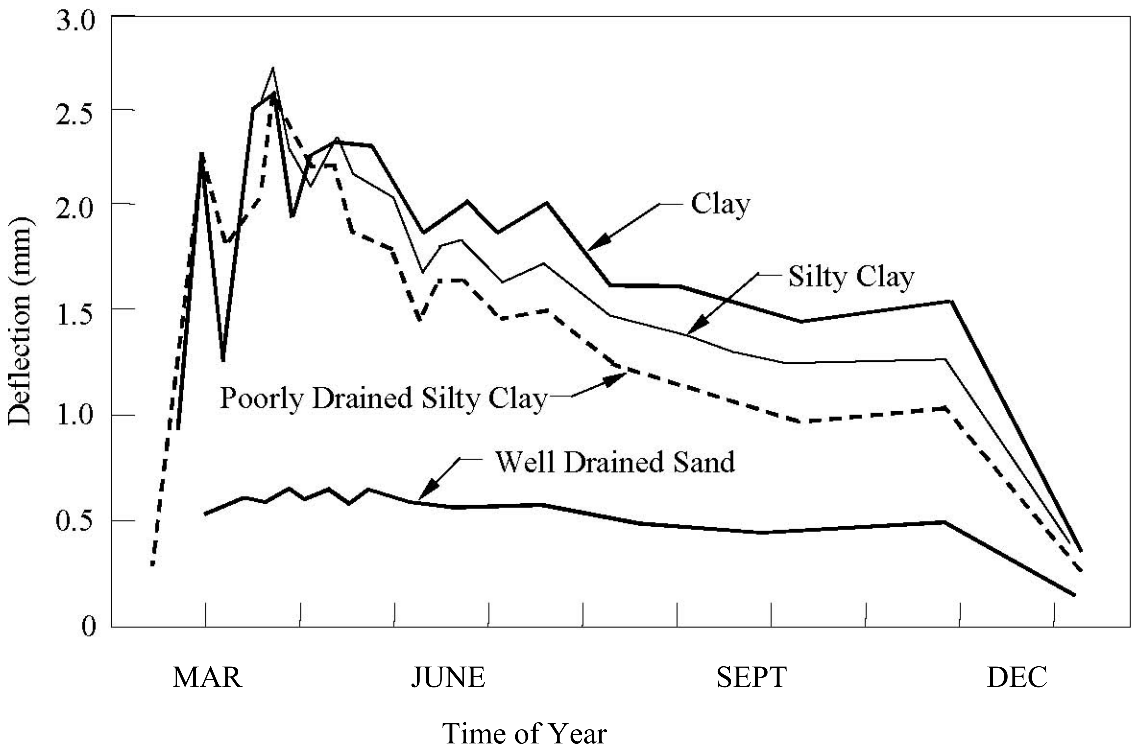


Figure 2-3.14. Influence of subgrade type on seasonal pavement deflection variations.

Thus, it is important to consider the time of day and the season when scheduling an NDT program and when interpreting NDT results. Ideally, deflection measurements are corrected to a standard temperature (typically 21°C) and critical season equivalent deflections based on locally-developed procedures.

7. CONDUCTING NDT FIELD SURVEYS

The NDT survey must be used in conjunction with information from the distress survey. NDT should generally be completed prior to destructive testing to assist in locating areas where sampling and testing will be conducted. When NDT data are used in a deflection-based overlay design procedure, it would be ideal to obtain a series of deflection measurements throughout the year to characterize the seasonal variations in pavement/soil structure strength. In fact, the current AASHTO Guide for Design of Pavement Structures[(2)] makes provision for determining an effective year-round roadbed soil resilient modulus based upon estimates of soil strength for each season. Furthermore, the current FHWA Long-Term Pavement Performance has an on-going study to evaluate seasonal variations as part of its Seasonal Monitoring Program. When this data becomes available, it should help highway agencies better address the effects of seasonal variations. For the time being (because it is impractical to measure deflections throughout the year), the next best practice is to measure pavement deflections at a time that best represents the effective year-round condition. For climates which experience frost penetration during the winter, the best time is probably right after the spring thaw (after the soil has regained some of its strength). Measurements during the spring thaw are not recommended as they may be overly conservative. The deflection survey should never be conducted when the pavement or subgrade soil is frozen, as misleading information will be obtained. Also, it should be noted that accurate layer thicknesses are needed to properly interpret the test data. If not available from original plans and/or specifications, layer thickness information may be obtained from cores or borehole logs.

Temperature Measurements

The pavement temperature must be measured when conducting an NDT deflection survey so that the deflections or moduli obtained can be corrected to a standard temperature. A relationship can be developed by locating a few points on the pavement and repeatedly measuring deflections at these points throughout the day, typically from very early morning to late afternoon. Air and pavement temperatures and deflections should be measured every hour and the results plotted to determine the temperature-deflection relationships. The locations selected should be representative of those at which measurements are to be taken over the entire project, such as wheelpaths, mid-slab, corners, and joints. Using this data, a temperature correction curve can be developed for the project so that the deflections can be adjusted to a standard temperature (such as 21°F) if required by overlay design procedures or for comparing deflections along the project.[6]

Representative pavement temperature information can be determined by drilling holes of varying depths in the HMA or PCC layers to obtain temperature profile information. Temperatures should be obtained at least at the pavement surface, with measurements at the mid-depth of the surface layer also recommended. If drilling is not possible, then pavement surface temperatures should be obtained along with a 5-day average of air-temperatures to estimate the layer temperatures.[41]

Testing Locations and Frequency

Deflections should be measured at 30- to 150-m intervals for all pavement types. 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 in one direction, they should be placed between those measurements in the opposite direction. The suggested deflection measurement locations for the various conventional pavement types are presented below.

- Flexible pavements (not including HMA overlays of jointed concrete pavements) should be tested in the outer wheelpath in each lane at the selected interval.

- Jointed PCC pavements (including those with HMA overlays) should be tested at the selected interval at mid-slab (center), at least 1.5 m away from any crack or joint, and at the joint corner. The corner deflection should be taken with the load plate placed as close a possible to the corner of the slab to measure load transfer across the joint, as well as the corner deflection. It may be desirable to measure load transfer across transverse cracks as well. Tests should be conducted at the approach and leave corners for void detection (loss of support).

- Continuously reinforced concrete pavements (CRCP) should be tested in the outer wheelpath with the center of the load approximately 0.6 m form the edge of the slab. The load should be placed between cracks and not on top of a crack. 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.

The adjusted deflection data can be plotted as shown in figure 2-3.15 and figure 2-3.16 to graphically illustrate 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. Note that if the specific overlay design procedure utilized by the agency requires a different pattern of deflection measurement, it should be followed in lieu of these recommendations.

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. Examples of areas that may require intensive testing are described below:

- PCC joints and corner cracks are likely areas where intensive deflections and coring tests would provide valuable information. This information is used to determine load transfer, loss of support, and the overall condition of the PCC surface and the base course.
- Areas in flexible pavements where deflections are inexplicably high could be studied in coordination with a coring program to determined the causes.
- Composite pavements may be tested at given locations to try to determine the amount of slab deterioration under the asphalt. Such NDT testing is often accompanied by coring or even removal of the HMA surface layer for direct observation of the PCC.
- Areas of non-uniform support in rigid pavements that would require more extensive rehabilitation

.

8. INTERPRETATION OF NONDESTRUCTIVE TEST DATA

The NDT deflection data should be used in conjunction with the distress, drainage, materials, and subgrade soil test results to determine the pavement structural condition. Several ways that deflection data are interpreted are discussed in this section. More detailed information is available in the NHI class on deflection analysis.

Uniformity of Project

The deflection profile along the project may be examined to determined if significant changes exist in the pavement's structural response, as illustrated in figure 2-3.15 and figure 2-3.16. The deflection profile should be compared with the results from the distress, drainage, materials, and subgrade soil surveys to determine the causes of areas of high or low deflections. This can be accomplished by plotting and comparing deflection profile, distress, and subgrade soil properties/types on strip maps. It may be advantageous to conduct the NDT evaluation after the distress survey, but before subgrade soil and materials testing, to assist in locating areas where the more intensive tests will be required. Based on all of this information, the project may be divided into two or more "design sections." Each design section may be treated separately for rehabilitation design purposes. For example, the overlay design or subdrainage design may be different for each of the distinct design sections.

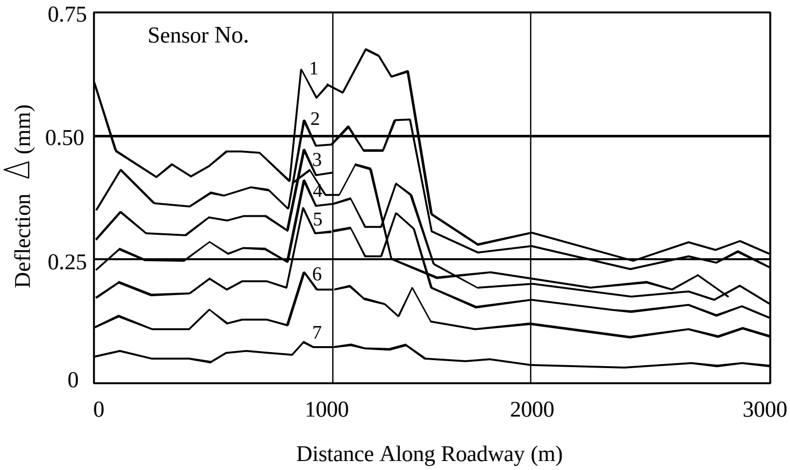


Figure 2-3.15. Illustration of deflection variation along a project.

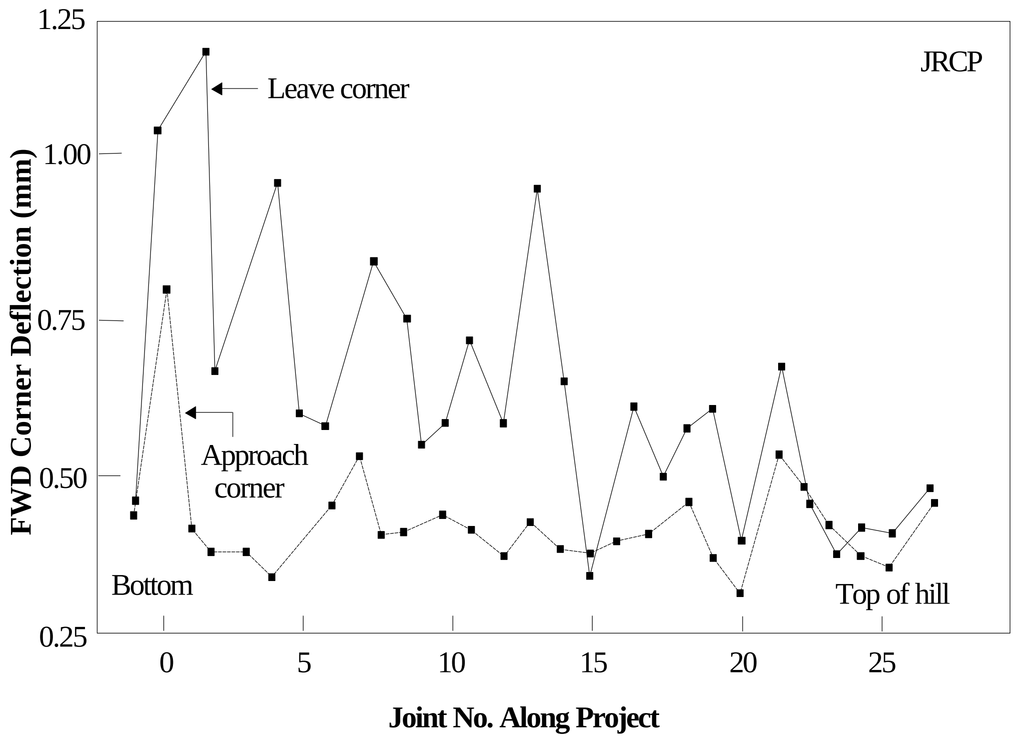


Figure 2-3.16. Illustration of joint-deflection profile for JRCP. [20]

Determining Pavement Layer Moduli

Backcalculation Process

"Backcalculation" is the accepted term used to identify a process whereby the fundamental engineering properties (i.e., effective elastic moduli) of the pavement structure and underlying subgrade soil are estimated based upon measured surface deflections. The process can be described using the simpler case of a simply supported beam (see figure 2-3.17). Given the dimensions of the beam (i.e., L=length, h=height, and b=width), the magnitude (P) and position (midspace) of the load and the measured deflection at midspan (δ), it is possible to "backcalculate" the elastic modulus (E) of the beam.

From fundamental engineering mechanics, the equation for calculating the midspan deflection is:

$$\delta = \frac{PL^3}{48EI} \tag{2.4.1}$$

here:

I = moment of inertia for a rectangular beam (i.e., $I = bh^3/12$).

By substituting the known values of δ, P, L, b, and h, the elastic modulus of the beam (E) can be backcalculated.

The primary problem associated with backcalculating elastic modulus values for a layered pavement structure is that the equation(s) for calculating pavement surface deflection (or any other pavement response) are not closed form. Consequently, a rigorous iterative process involving some decision and convergence criteria is typically required.

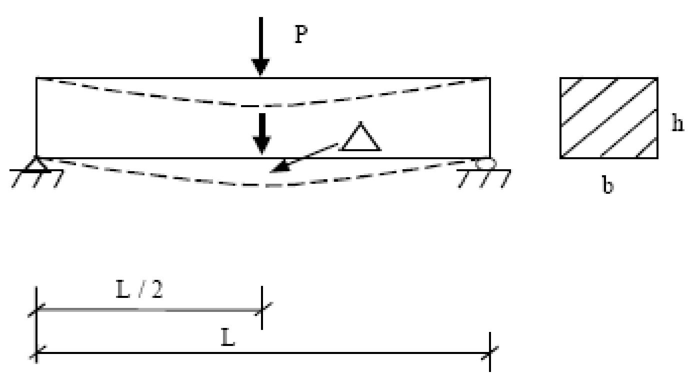


Figure 2-3.17. Illustration of simply supported beam, of length (L), width (b), height (h) with concentrated load (P) at mid span.

Figure 2-3.18 illustrates the traditional use of deflection basin results to backcalculate in situ layer moduli. The pavement is viewed as a multi-layered elastic system, and in order to solve for the layer moduli, the thickness, h, and Poisson's ratio, μ, of each layer must be know. In practice, a Poisson's ratio is assumed for analysis, based on the materials in each layer. Typical values use for various paving materials are shown in table 2-3.2.

For specific dynamic NDT equipment and test conditions, two of the following three inputs must be known: the dynamic load, P, plate radius, a , and plate pressure, P_c. Deflections are measured at radial offset values, r , from the plate load center to define the deflection basin. The underlying assumption is that a set of layer moduli (E , E , ...E) exists that, under the dynamic load-pavement combination, yields specific deflection values equal to the measured deflections of the basin. Note, however, that this solution may not be unique. Multi-layer elastic theory is used in a "backcalculation" technique to arrive at a set of layer moduli which produces a theoretical deflection basin that matches the measured deflection basin. A limitation of this approach is that the mathematical complexities require a computerized solution. However, several such solutions are now commonly available for use on all types of computer systems, examples of which are presented in the subsequent section. Another limitation is the inherent assumption that the pavement materials conform to an idealized condition of being linearly elastic, uniform, and continuous. This assumption ignores discontinuities (cracks), stress dependency, material variability, and the effects of temperature gradients. Thus, engineering judgment is a key element of the backcalculation process.

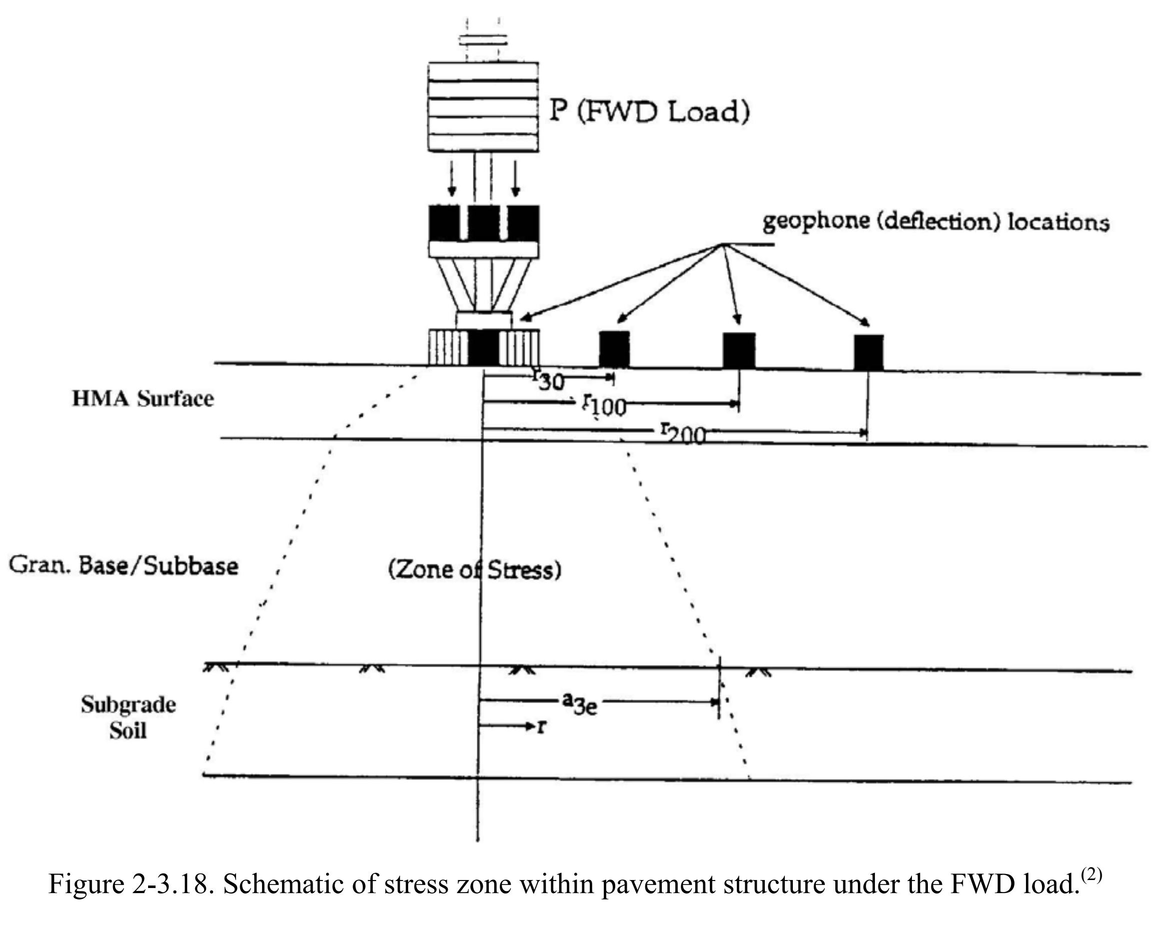


Figure 2-3.18. Schematic of stress zone within pavement structure under the FWD load.[2]

Table 2-3.1. Typical Poisson ratio values.[2]

Material	Range General	Remarks	Value Typical
Portland Cement Concrete	0.10 - 0.20		0.15
HMA/Asphalt Treated Base	0.15 - 0.45	Highly dependent upon temperature; use low value (0.15) for cold temperatures (less than 30°C and high value (0.45) for warm pavement 50°C	0.35
Cement Stabilized Base	0.15 - 0.30	Degree of cracking in stabilized layer tends to increase value towards 0.30 From sound (crack free) value of 0.15	0.20
Granular Base/Subbase	0.30 - 0.40	Use lower value for crushed material and high value for unprocessed rounded gravels/sands	0.35
Subgrade Soils	0.30 - 0.50	Value dependent upon type of subgrade soil. For cohesionless soils, use value near 0.30. A value of 0.50 is approached for very plastic clays (cohesive soils).	0.40

Layer moduli backcalculated from NDT measurements should be compared to the agency's prior experience with similar materials and with the general material type (as shown in table 2-3.3) to verify that the results are reasonable. The use of limited destructive testing/sampling is encouraged to provide spot verification of NDT derived moduli values. The final output of the NDT deflection basin backcalculation approach is the individual layer moduli values, as well as the subgrade soil modulus.

Table 2-3.2. Typical layer modulus values.

Material	General Range (mPa)	Typical Value (mPa)
Hot-Mix Asphalt[1]	1,500 - 3,500	3,000
Portland Cement Concrete	20,000 - 55,000	30,000
Asphalt-Treated Base[1]	500 - 3,000	1,000
Cement-Treated Base	3,500 - 7,000	5,000
Lean Concrete	7,000 - 20,000	10,000
Granular Base	100 - 350	200
Granular Subgrade Soil	50 - 150	100
Fine-Grained Subgrade Soil	20 - 50	30

The properties of HMA layers are highly temperature dependent. These modulus values are based on pavement temperatures in the range of 20°C to 30°C.

Flexible pavement systems are most commonly modeled using multi-layer elastic theory; however, stress-dependent procedures are also used. Rigid pavement systems are most commonly analyzed using plate theory methods that are able to consider the discontinuities that exist in rigid pavements (i.e., joints and cracks), although interim infinite slab solutions are also used.[21,22] Any backcalculated modulus values should be considered as “effective” layer modulus values over a given horizontal and vertical area, due to the stress dependency of most subsurface paving materials.

Backcalculation Programs

The deflection results from all types of dynamic deflection equipment (steady-state vibratory and impulse) may be used to determine pavement layer moduli. Most backcalculation programs involve the application of the multi-layer elastic theory to calculate a set of theoretical deflections for a pavement system with assumed initial modulus values. Through a series of iterations, the layer moduli are changed, and the calculated deflections are then compared to the measured deflections until a match is obtained within tolerance limits. The backcalculated moduli represent a theoretical pavement system that responds to the load based on surface deflections, in a manner similar to the actual in situ pavement system. Table 2-3.4 provides an example of an iterative backcalculation solution involving a three-layer flexible pavement and NDT measurements from and FWD.

Some “rules of thumb” that have evolved as a result of the development and application of the backcalculation technology include:

- Deflections greater than about one meter from the center of the load are almost totally dependent upon the modulus of the roadbed soil. In fact, if there is one layer for which a closed-form solution (between deflection and modulus) might exist, it is the roadbed soil. Theoretically, thicknesses and modulus values of the pavement layers do not have much affect on deflections significantly away from the load.

- Underlying rigid layers (ranging from bedrock to water tables) do have an impact on the measured surface deflections and should be considered in the backcalculation process. In fact, ignoring these rigid layers when they are present will frequently results in a poor “match” and an unconservatively high subgrade soil modulus.

- Deflections from pavements with multiple-bound surface layers tend to have multiple solutions. By increasing the modulus of the surface HMA layer and simultaneously reducing the modulus of an asphalt stabilized base layer, for example, an alternate (potentially accurate) solution can be found. In these cases, it is useful to have information from laboratory testing to help eliminate the duplicity.

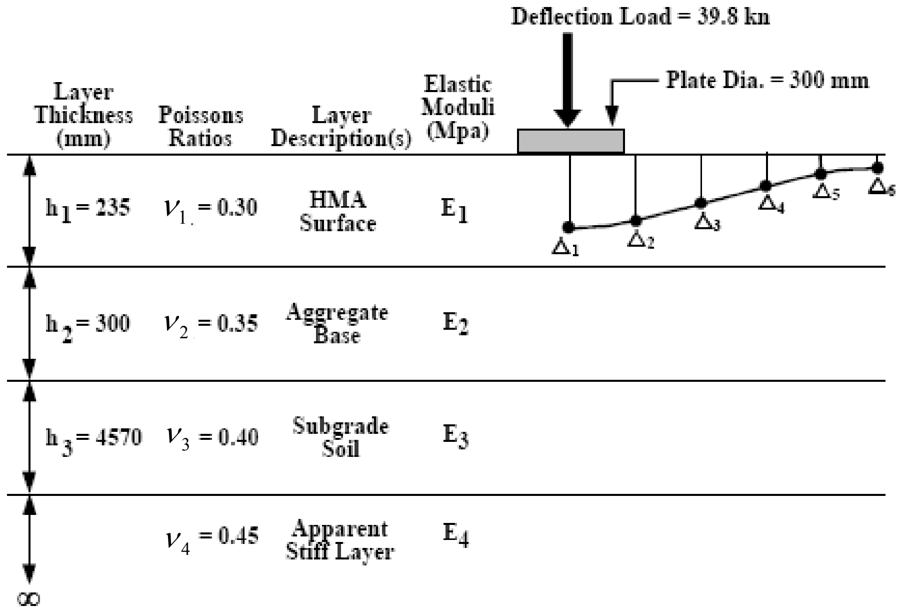
- It is difficult to backcalculate the modulus of thin (less than 75 mm) layers, especially those at the surface. Variations in the moduli of thin layers, theoretically, do not produce significant variations in surface deflection (at least by measurement standards).

The iterative procedures used to match the measured and calculated deflection have given rise to a wide array of backcalculation programs available to pavement engineers. The backcalculation programs currently used most by engineers are those based on an iterative, inverse application of the elastic layered theory. Examples of these programs include BISDEF,[23] ELSDEF,[4] and CHEVDEF,[15] based on the elastic layered programs BISAR,[24] ELSYM5,[25] and CHEVRON,[26] respectively. Although these programs can be used to evaluate both flexible and rigid pavements, they are particularly suitable for flexible pavements.

Table 2-3.3. Example of an iterative backcalculation solution for a three-layer flexible pavement.

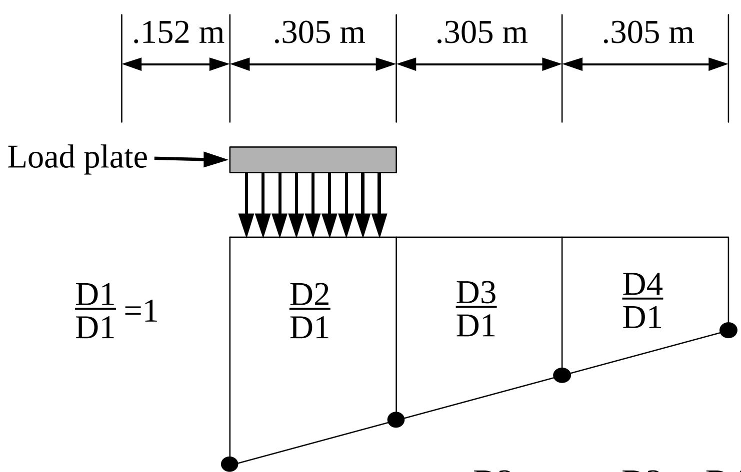
Iteration	Trial Moduli (MPa)				Predicted Deflections (mm)							Average Percent Difference
	E_1	E_2	E_3	E_4	Δ_1	Δ_2	Δ_3	Δ_4	Δ_5	Δ_6	Δ_7	
1	1724	276	138	690	.276	.201	.166	.132	.108	.075	.040	20.5
2	1724	276	207	345	.238	.167	.136	.105	.083	.055	.031	36.4
3	1724	207	103	276	.335	.257	.218	.177	.147	.104	.058	5.9
4	1793	224	107	297	.320	.245	.208	.169	.141	.100	.056	1.3
5	1862	224	107	297	.316	.243	.207	.169	.141	.100	.056	0.9
	Measured Deflections (mm):				.309	.243	.208	.171	.140	.099	.054	

**Note: Average percent difference represents the calculated average between measured and predicted deflections for all sensors.



In a variation of the traditional backcalculation programs, there are programs in which the measured deflections are directly compared to sets of deflection basins stored in a database. Typically, elastic layer programs are used to generate a set of deflection basins based on pavement system parameters, including layer thicknesses and acceptable ranges of modulus values. The deflection basins collected by the NDT device are then compared to the calculated basins in the database. Programs that use this process are very efficient, as basins have to be calculated only once for a given set of parameters. Examples of programs that use this approach are MODULUS[27] and COMDEF,[28] both of which utilize databases generated with the CHEVRON[26] program. MODULUS, which uses the WESLEA program to generate its database of deflection basins, is applicable to flexible pavements while COMDEF was developed specifically for composite pavements consisting of an HMA overlay over a PCC layer.

A backcalculation program currently available for two-layer rigid pavements is the ILLI-BACK program.[29] Based on closed-form procedures, this program utilizes a unique relationship between the parameter "AREA" of the deflection basin and the radius of relative stiffness of the pavement system. The term AREA refers to the general shape of the deflection basin, as shown in figure 2-3.19, rather than the absolute cross-sectional area of the deflection basin developed under a load.[30] Typical AREA values range from 730- to 840-mm. The AREA and maximum deflection (D1) are used in Westergaard's deflection equation to backcalculate the slab E and modulus of subgrade reaction, *k*. Large AREA values indicate a greater E/*k* ratio, whereas small AREA values indicate a low E/*k* ratio. A small AREA value does not necessarily mean that the slab is disintegrating. Figure 2-3.20 provides a graph illustrating how the AREA and D1 parameters are used to backcalculate the PCC slab modulus and subgrade soil *k*-value.



$$\text{Area (m)} = .152\left(.025 + .051\frac{D2}{D1} + .051\frac{D3}{D1} + \frac{D4}{D1}\right)$$

Figure 2-3.19. Calculation of deflection basin "AREA."[30]

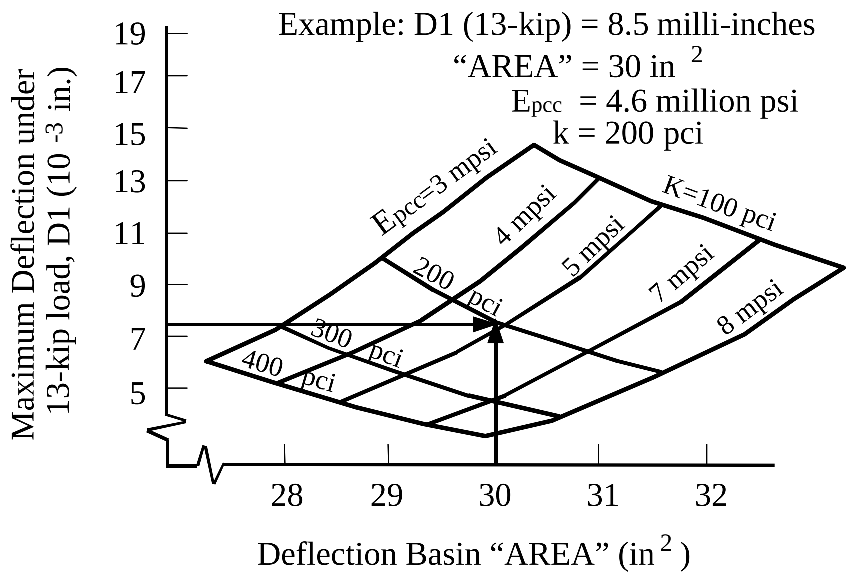


Figure 2-3.20. Illustration of concept for backcalculating moduli for a two-layer rigid pavement structure. [Chart is based on equations developed by Hall(35)].

A list of various backcalculation program currently available for use by pavement engineers is given in table 2-3.5. This list, which shows the number of layers each program can handle, their basis, and the pavement type for which they are intended, is by no means exhaustive.

Location of Loss of Support (Voids)

One of the major causes of deterioration of all types of rigid pavements in pumping and the subsequent loss of support beneath the slab. Data from both the general deflection profile and localized testing can be used to estimate the general extent of loss of support and, thus obtain estimates of the extent of 4R work required to provide improved support. A rational procedure for determining loss of support beneath slab corners using FWD testing is outlined in references 20, 41, and 43.

Deflections taken on both sides of a joint can be used to determine the general extent of loss of support. Field studies have shown that materials pump counter to the flow of traffic. Where pumping exists, the deflection on the leave slab should be higher than that of the approach slab. The greater the difference between deflections across the joint or crack, the greater the extent of pumping. However, if deflections on both sides of the joint are abnormally high, the loss of support may extend under both sides of the joint.

Table 2-3.4. Pavement layer backcalculation programs.

Backcalculation Program	Number of Layers	Basis	Pavement Type	Non-linear Stress Effects	Reference
BISDEF	4	BISAR	Flexible	No	23,24
ELSDEF	4	ELSYM5	Flexible	No	4,25
CHEVDEF	4	CHEVRON	Flexible	No	15,26
MODULUS	4	WESLEA	Flexible	No	27
COMDEF	3	CHEVRON	Composite	No	28
ILLI-BACK	2	Westergaard	Rigid	No	29
MDOCOMP2	8	ELSYM5	Flexible	Yes	31
BOUSDEF	4	Equivalent Thickness	Flexible	No	
ELMOD	4	Equivalent Thickness	Flexible	Yes	32
EVERCALC	5	WESLEA	Flexible	Yes	33
WESDEF	5	WESLEA	Flexible	No	34

In multi-lane facilities, loss of support generally develops under the corners of the outer lane, they rarely develop under the corners of the inside or passing lane although that can happen, especially where no tiebars exist across the longitudinal joint. However, deflections could be measured on the inside corner and outside corner. A significantly larger outside corner deflection in comparison to the inside corner may be indicative of a loss of support. An example of a corner joint deflection profile for jointed reinforced concrete pavements (JRCP) is shown in figure 2-3.16 and indicates that the leave corner is exhibiting greater loss of support than the approach corner. It is important to note that the void may be very small, e.g., 0.25 mm, and yet still be enough to cause very significant loss of support and high stresses in the slab.

Joint and Crack Load Transfer

The load transfer across a longitudinal or transverse joint or crack can be determined by one of two basic methods. With the steady-state dynamic load and impulse load deflection devices, the loading plate can be positioned adjacent to the joint or crack with a deflection sensor located across the joint or crack. The load transfer efficiency in terms of the measured deflection can then be calculated using the following formula:

$$LTE = \left(\frac{\Delta_u}{\Delta_l}\right) * 100 \qquad (2\text{-}3.2)$$

where: LTE = load transfer efficiency, percent

Δ_u = deflection on unloaded side of joint or crack

Δ_l = deflection on loaded side of joint or crack

Figure 2-3.21 illustrates the concept of deflection load transfer. It is important to note that deflection load transfer is not the same as stress load transfer. It should also be noted that different load transfer salvage may be obtained depending upon which side of the joint is loaded. Thus, both sides of the joint should be tested and the lowest value used.

The presence of subsurface deterioration can be determined by coring through the joint and comparing the results to NDT and distress survey results. Then an estimate of joint and crack repair can be made. The
deflection load transfer of the joint/crack can be judged approximately by the following scale:

Good: greater than 75 percent
Fair: 50 percent to 75 percent
Poor: less than 50 percent

Any joint having less than about 50 percent deflection transfer is likely to show pumping of the underlying material and rapid reflection cracking through asphalt overlays due to large differential deflections.

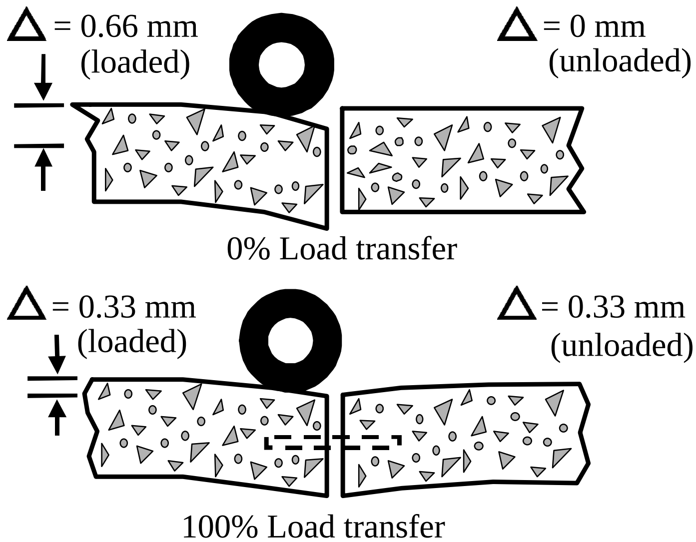


Figure 2-3.21. Concept of load transfer.

9. SUMMARY

This emphasis of this module was on the application, collection, processing, and interpretation of NDT data for project level pavement evaluation. It serves as one of the basic criteria for assessing the structural adequacy of an existing pavement (which is covered in module 2-7).

The major factors that influence the measured deflection on flexible and rigid pavements are:

- Type of loading (static, vibratory, impulse), frequency, and magnitude.
- Pavement structure, subgrade soil, existing distress, cracks, joints, voids beneath slabs, and random variations in material properties.
- Temperature, thermal gradient, moisture gradient, and season of the year.

The Falling Weight Deflectometer is one of the most widely used forms of non-destructive equipment. Considerations in an NDT program include:

- Obtaining deflections at regular intervals along the project length.
- Correcting the deflections for temperature and seasonal influences.
- Identifying the need for a more intensive, localized deflection study where necessary.

NDT surveys should be made during the period that best represents the year-round condition. For climates with frost penetration, this should be after the spring thaw. NDT should not be run during periods of deep frost. Where it is unavoidable, testing at times other than the critical season is acceptable, provided the necessary adjustment factors are applied to the results obtained.

The results of nondestructive testing can be used in a variety of ways, including:

- Determining the structural uniformity of a project by identifying areas of significant weakness.
- Assessing the structural capacity of a pavement (remaining life).
- Designing overlays.
- Locating loss of support beneath slabs and determining the need for preoverlay repair.
- Evaluating joint and crack deterioration and load transfer efficiency.
- Determining the stiffness properties of the various pavement layers and subgrade soil.

10. REFERENCES

1. Darter, M.I. and K.T. Hall, “Structural Overlay Strategies for Jointed Concrete Pavements, Volume IV—Guidelines for the Selection of Rehabilitation Alternatives,” FHWA-RD-89-145, Federal Highway Administration, June 1990.

2. “AASHTO Guide for Design of Pavement Structures,” American Association of State Highway and Transportation Officials, 1993.

3. Hudson, W.R., G.E. Elkins, W. Uddin and K.T. Reilley, “Evaluation of Pavement Deflection Measuring Equipment,” FHWA-TS-87-208, Federal Highway Administration, 1987.

4. Lytton, R.L., F.P. Germann, Y.J. Chou, and S.M. Stoffels, “Determining Asphaltic Concrete Pavement Structural Properties by Nondestructive Testing,” NCHRP Report 327, Transportation Research Board, June 1990.

5. “Standard Recommended Practice for Pavement Deflection Measurements,” T 256-77, Standard Specifications for Transportation Materials and Methods of Sampling and Testing, American Association of State Highway and Transportation Officials, Part II, 1990.

6. “Asphalt Overlays for Highway and Street Rehabilitation,” The Asphalt Institute Manual Series No. 17, 1987.

7. “Soils Manual for the Design of Asphalt Pavement Structures,” The Asphalt Institute Manual Series No. 10, Second Edition, 1988.

8. Lytton, R.L., W.M. Moore, and J.P. Mahoney, “Pavement Evaluation Equipment,” Federal Highway Administration, 1975.

9. “Methods of Test to Determine Overlay Requirements by Pavement Deflection Measurements,” California Test 356, Department of Transportation, Division of Construction, Office of Transportation Laboratory, 1978.

10. Nazarian, S. and K.H. Stokoe II, “Use of Surface Waves in Pavement Evaluation,” Transportation Research Record 1070, Transportation Research Board, 1986.

11. Sanchez-Salinero, I., J.M. Roesset, K.Y. Shao, K.H. Stokoe II, and G.J. Rix, “Analytical Evaluation of Variables Affecting Surface Wave Testing of Pavements,” Transportation Research Record 1136, Transportation Research Board, 1987.

12. Sheu, J.C., K.H. Stokoe II, and J.M. Roesset, “Effect of Reflected Waves in SASW Testing of Pavements,” Transportation Research Record 1196, Transportation Research Board, 1988.

13. Nazarian, S., K.H. Stokoe II, R.C. Briggs, and R. Rogers, “Determination of Pavement Layer Thicknesses and Moduli by SASW Method,” Transportation Research Record 1196, Transportation Research Board, 1988.

14. Rix, G.J., J.A. Bay, and K.H. Stokoe II, “Assessing In Situ Stiffness of Curing Portland Cement Concrete with Seismic Tests,” Transportation Research Record 1284, Transportation Research Board, 1990.

15. Moore, M.R., C.R. Haile, D.I. Hanson, and J.W. Hall, “An Introduction to Nondestructive Structural Evaluation of Pavements” Transportation Research Circular 189, Transportation Research Board, 1978.

16. “The AASHO Road Test, Report 5—Pavement Research,” Special Report 61E, Highway Research Board, 1962.

17. Scrivner, F.H., R. Poehl, W.M. Moore, and M.B. Phillips, “Detecting Seasonal Changes in Load-Carrying Capabilities of Flexible Pavements,” NCHRP Report 76, Highway Research Board, 1969.

18. Poehl, R., “Seasonal Variations of Pavement Deflections in Texas,” Research Report 136-1, Texas Transportation Institute, 1971.

19. Bushey, R.W., et al., “Structural Overlays for Pavement Rehabilitation,” CA -DOT-TL-3128-3-74-12, California Department of Transportation, 1974.

20. Crovetti, J.A. and M.I. Darter, “Void Detection for Jointed Concrete Pavements,” Transportation Research Record 1041, Transportation Research Board, 1985.

21. Foxworthy, P.T. and M.I. Darter, “Preliminary Concepts for FWD Testing and Evaluation of Rigid Airfield Pavements,” Transportation Research Record 1070, Transportation Research Board, 1986.

22. Foxworthy, P.T. and M.I. Darter, “A Comprehensive System for Nondestructive Testing and Evaluation of Rigid Airfield Pavements,” Transportation Research Record 1070, Transportation Research Board, 1986.

23. Bush, A.J., “Computer Program BISDEF,” United States Army, Corps of Engineers, Waterways Experiment Station, November 1985.

24. Peutz, M.G.F., Van Kempen, H.P.M., and A. Jones, “Layered Systems Under Normal Surface Loads,” Highway Research Record 228, Highway Research Board, 1968.

25. Ahlborn, G., “ELSYM5 Computer Program for Determining Stresses and Deformation in Five Layer Elastic System,” University of California, 1972.

26. Warren, T. and W.L. Diekmann, “Numerical Computation of Stresses and Strains in Multiple-Layer Asphalt Pavement System,” Internal Report, Chevron Research Company, 1963.

27. Uzan J., Scullion, T., Michalek, C.H., Paredes, M., and R.L. Lytton, “A Microcomputer-Based Procedure for Backcalculating Layer Moduli From FWD Data,” Research Report 1123-1, Texas Transportation Institute, July 1988.

28. Anderson, M., “A Database Method for Backcalculation of Composite Pavement Layer Moduli,” Nondestructive Testing of Pavements and Backcalculation of Moduli, ASTM STP 1026, A.J. Busch III and G.Y. Baladi, Eds., American Society for Testing and Materials, Philadelphia, 1989.

29. Ioannides, A.M., Barenberg, E.J., and J.A. Lay, “Interpretation of Falling Weight Deflectometer Results Using Principles of Dimensional Analysis,” Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, 1989.

30. Hoffman, M.S. and Thompson, M.R., “Mechanistic Interpretation of Nondestructive Pavement Testing Deflections,” Transportation Engineering Series No. 32, Illinois Cooperative Highway and Transportation Research Program Series No. 190, University of Illinois, 1981.

31. Irwin, L.H., “Users Guide to MODCOMP2,” Report No. 83-8, Cornell University Local Roads Program, Cornell University, November 1983.

32. Ullidtz, P., “Overlay and Stage by Stage Design,” Proceedings, Fourth International Conference on the Structural Design of Asphalt Pavements, Volume I, University of Michigan, 1977.

33. “Pavement Deflection Analysis,” National Highway Institute Course No. 13127, FHWA-HI-94-021, Federal Highway Administration, February 1994.

34. Van Cauwelaert, F.J., D.R. Alexander, T.D. White, and W.R. Barker, “Multilayer Elastic Program for Backcalculating Layer Moduli in Pavement Evaluation,” Nondestructive Testing of Pavements and Backcalculation of Moduli, ASTM STP 1026, A.J. Bush III and G.Y. Baladi, Eds., American Society for Testing and Materials, Philadelphia, 1989.

35. Hall, K.T., “Performance, Evaluation, and Rehabilitation of Asphalt-Overlaid Concrete Pavements,” Ph.D. Dissertation, University of Illinois at Urbana-Champaign, 1991.

36. Nazarian S., Baker M. and Crain K. (1993) “Developing and Testing of a Seismic Pavement Analyzer,” Research Report SHRP-H-375, Strategic Highway Research Program, December.

37. Nazarian S., Yuan D. and Baker M.R. (1995) “Rapid Determination of Pavement Moduli with Spectral-Analysis-of-Surface-Waves Method,” Research Report 1243-1F, Center for Geotechnical and Highway Materials Research, UTEP, November, 1995.

38. Lukanen, E.O., R. Stubstad, R., and R. Briggs, “Temperature Predictions and Adjustment Factors,” Draft FHWA Report, July 1996, FHWA McLean, VA.

39. Darter, M.I., K.T. Hall, and Chen-Ming Kuo, “NCHRP Report 372 Support under Portland Cement Concrete Pavements,” Transportation Research Board, National Academy Press, Washington, DC, 1995.

40. Hall, K.T., M.I. Darter, T.E. Hoerner, and L. Khazanovich, “LTPP Data Analysis Phase 1: Validation of Guidelines for K-Value Selection and Concrete Pavement Prediction,” FHWA Report No. FHWA-RD096-198, January 1997, FHWA McLean, VA.

41. Roberts, D.V., G.W. Mann, and C.A. Curtis. 1977, “Evaluation of the Cox Deflection Devices,” Report number FHWA-CA-TL-3150-77-14, California Department of Transportation, Sacramento, CA.

MODULE 2-4

LABORATORY MATERIALS CHARACTERIZATION

1. INSTRUCTIONAL OBJECTIVES

In considering various pavement rehabilitation alternatives, it is often advantageous to perform selected laboratory tests on samples taken from the candidate pavement section. The results of these laboratory tests may be used to compare with results from nondestructive testing (NDT) tests such as falling weight deflectometer (FWD) deflection testing or simply to obtain accurate engineering properties of the existing pavement layers. The emergence of new mechanistic pavement design procedures has also increased the need for accurate measurement of the material properties of the existing pavement structure. Often, the only alternative for obtaining accurate data on these material properties is to collect samples in the field and test them in the laboratory.

This module presents factors that influence strength, permanent deformation, and repeated load behavior of granular and cohesive soils, as well as procedures for subgrade evaluation. Information is also provided on procedures for the evaluation of materials in the bound pavement layers (asphalt and portland cement). Special emphasis is placed on the relatively new techniques for obtaining engineering properties of existing pavement layers for the purpose of mechanistic pavement design procedures. At the conclusion of this module, the participant should be able to accomplish the following:

1. Describe the basic stress states of the in-service pavement layers and subgrade. Identify the major engineering properties and test procedures for soils and paving materials.
2. Restate the basic terminology related to material characterization.
3. Describe the concept behind resilient modulus testing and its importance to mechanistic pavement design.
4. Indicate how moisture content, density, and freeze-thaw influence the repeated load behavior of a fine-grained soil.
5. Identify the major strength and material tests for portland cement concrete (PCC) and hot-mix asphalt (HMA) and describe their use in rehabilitation design.
6. Correlate different subgrade soil tests and relate them to resilient modulus.
7. Identify information sources for use in characterizing subgrade support and in evaluating problems such as frost heave and expansive soils.

2. DEFINITIONS

Resilient Modulus (M_r). Engineering property of the base, subbase, subgrade layers often used in mechanistic pavement design. Equivalent to the ratio of the amplitude of the repeated axial stress to the amplitude of the resultant recoverable axial strain.

Loading Waveform. The specific loading form used in testing for resilient modulus. The sinusoidal and haversine waveforms are commonly used.

Deviator Stress. Axial stress in an unconfined compression test and the excess axial stress in a triaxial compression stress.

Total Strain. Total deformation under the load.

Resilient (Elastic) Strain. Deformation recovered when the load is removed.

Permanent (Plastic) Strain. Deformation not recovered when the load is removed.

Indirect Tensile Test. A test commonly used to determine the resilient modulus of a bituminous specimen whereby tensile stresses are measured across the horizontal diameter of the specimen while a compressive force is applied in the vertical direction.

R-Value. Test property resulting from the Hveem Stabilometer.

California Bearing Ratio (CBR). Test property determined by measuring the penetration resistance of a soil.

3. INTRODUCTION OF STATES OF STRESS

A moving wheel imparts a dynamic load pulse to the pavement structural layers and the subgrade soil. This dynamic load results in a complex set of stresses and strains on the pavement structure, as illustrated in figure 2-4.1. (Not shown in this picture are the additional shear stresses that act on each face of the exploded element shown). Although each of these elemental stresses must be accounted for in mechanistic design, there are primary stresses that often control the design. These primary stresses are the vertical compressive stress (σ_v), the tangential stress (σ_t), and the radial stresses (σ_r). Each of these stresses is shown in figure 2-4.1. The magnitude of the stresses and strains are a function of tire pressure, point location within the pavement structure, the number of pavement layers, and the thickness and material properties of those layers.

While it is beyond the scope of this course to delve into the complexities of the material mechanics that make it possible to calculate those stresses and strains, it is important to review and understand the basis stress states in a typical flexible pavement structure. Most mechanistic pavement design procedures address one of two primary stresses in the pavement structure. First, is the tensile stress (σ_t) at the bottom of the HMA surface layer. A greatly simplified version of the tensile stresses in a typical flexible pavement directly under a wheel load is shown in figure 2-4.2. The magnitude of this tensile stress, relative to the elastic modulus of the bituminous material that makes up the layer, will dictate the fatigue life of the pavement. High tensile stresses in the bottom of the surface layer result in shorter fatigue life of the HMA. High tensile stresses in the bottom of the surface layer are also indicative of high stresses in the top of the base layer. Since often the base layer is not stabilized, it will have little or no strength to resist these tensile stresses that can lead to decompaction.

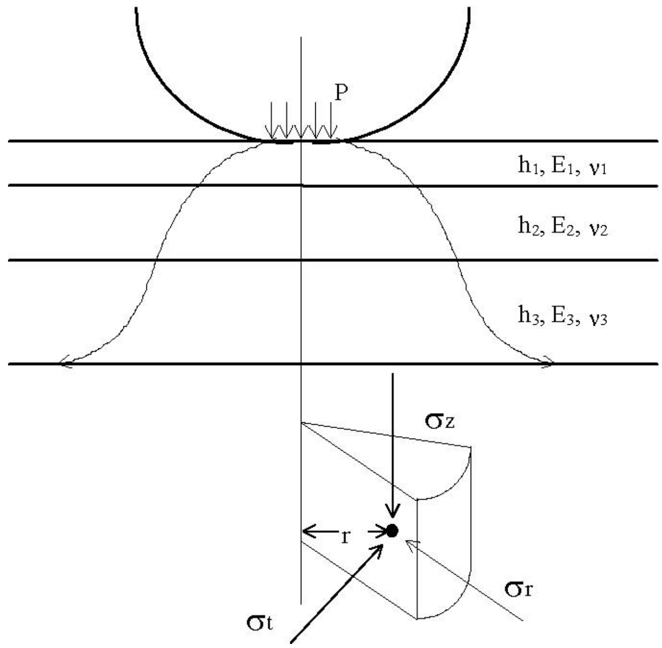


Figure 2-4.1. Three dimensional stress states in a typical pavement structure.

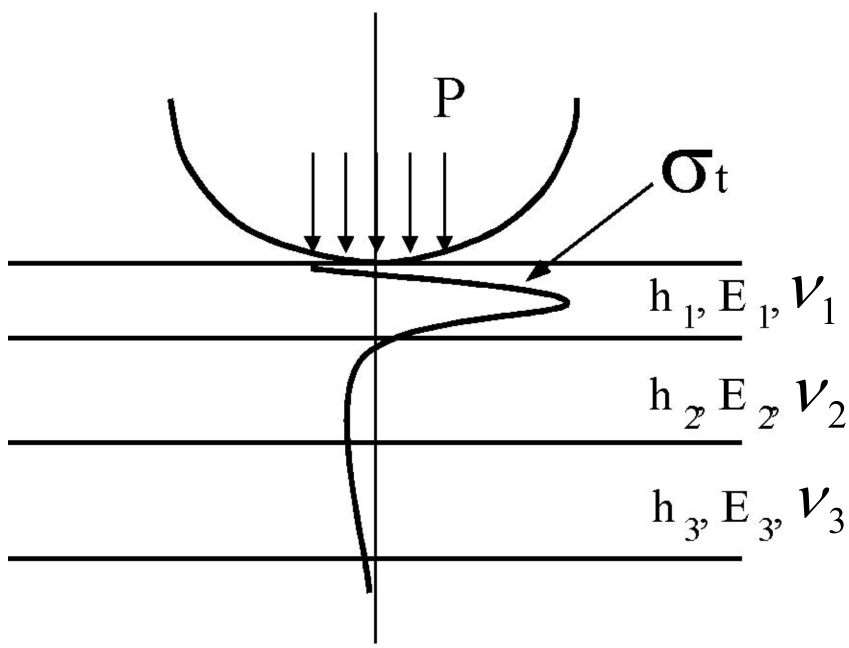


Figure 2-4.2. Typical distribution of tensile stresses in a flexible pavement structure.

A second and equally important stress often considered in mechanistic pavement design is the vertical compressive stress (σ_v) at the top of the subgrade soil. A simplified version of vertical compressive stress is shown in figure 2-4.3. In response to the application of a wheel load, each pavement layer deflects under stress. If the stress exceeds the bearing capacity of the subgrade soil, failure in the pavement structure will occur. Also, if the vertical compressive strength is high relative to the resilient modulus of the subgrade soil layer, permanent deformation will be excessive in the subgrade soil that can also lead to failure of the pavement structure.

The cumulative deformation in the base and subbase layers, due to vertical compressive stress, is also considered in most mechanistic rutting models. Therefore, it is important to quantify the resilient modulus of these layers by performing laboratory tests on field samples.

4. IMPORTANCE OF LABORATORY TESTING

In recent years the importance of adequately characterizing the structural characteristics of the existing pavement, prior to evaluating various rehabilitation alternatives, have been widely-recognized. Although economics is often the driving force behind these efforts, other factors such as the increased emphasis on recycling and the increased emphasis on rehabilitating existing pavement networks are also credited.

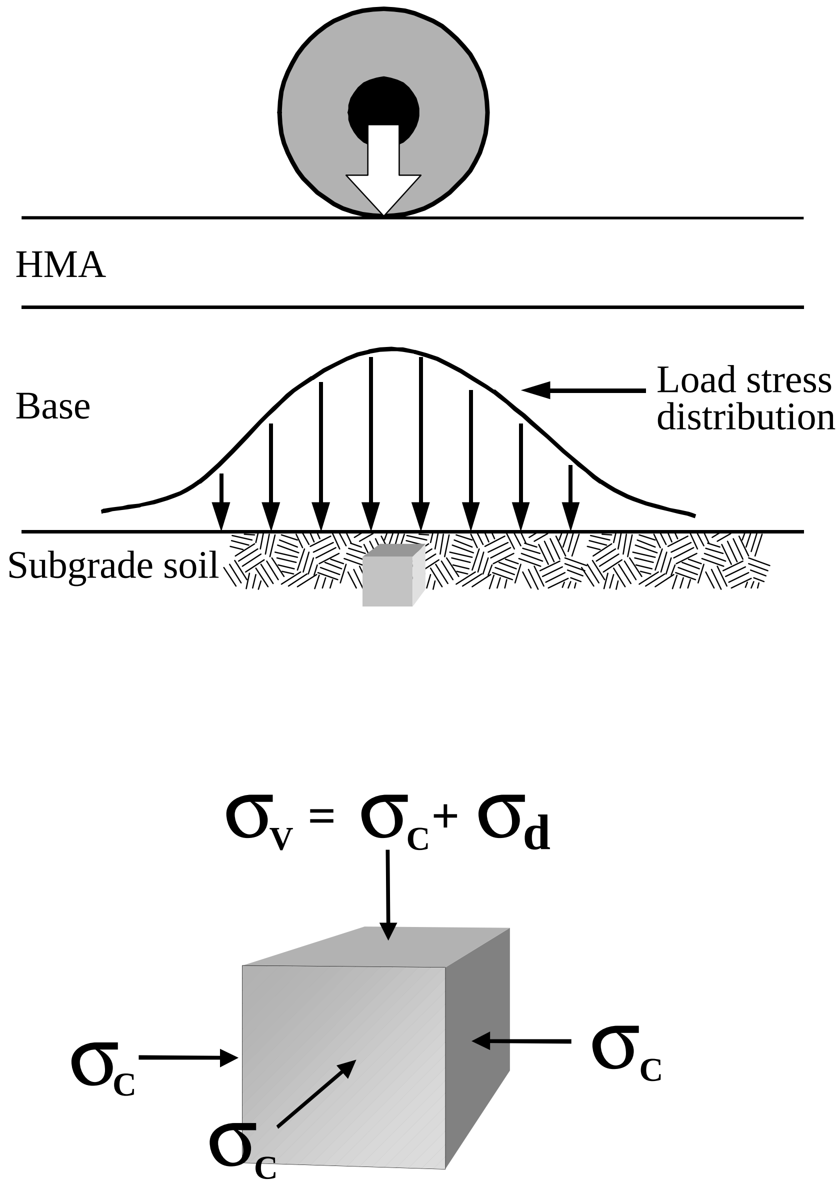
The emphasis on accurately characterizing the existing pavement structure has led to a significant increased in the number of NDT techniques for project level pavement evaluation. The most common of these NDT techniques is FWD deflection testing and ground penetrating radar (GPR). However, many others are presently in the development stage. These NDT techniques, which are discussed in more detail in the next section, are powerful tools for identifying weaknesses and defects in the existing pavement structure. By properly interpreting this NDT information, more efficient decisions can be made regarding the most feasible rehabilitation alternatives to consider.

Despite the significant advancements in these NDT technologies, it is often necessary to verify and/or calibrate the data generated with these devices. This is best accomplished by taking cores of the existing pavement structure at predetermined locations to compare with the results generated with NDT. By properly interpreting the NDT data, the most appropriate locations for taking cores can be determined in advance. Coordinated in this fashion, these two approaches are very complimentary and can lead to extremely accurate characterization of the entire pavement section being considered for rehabilitation.

Determining When Laboratory Testing is Required

Determining when and where to take field samples for laboratory testing are not always easy questions to answer. Research, presently underway at the Texas Transportation Institute, is addressing this very issue. Presently, there are only general guidelines that can be used for establishing a laboratory testing program. The following is a summary of these guidelines:

- NDT Data. The availability of project level FWD deflection or GPR data can greatly improve the efficiency of laboratory evaluation of existing pavement materials. NDT data can be evaluated along the entire project to determine the degree of variability along the section. The NDT data will readily provide data regarding layer thicknesses, moisture content, and density. By properly interpreting this data, the quantity and location of core samples can readily be identified. As a general rule, the higher the variability of the NDT results, the more coring is necessary to verify those results via laboratory testing.



σ_c = Confining stress

σ_d = Deviator stress $= \sigma_v - \sigma_c$

σ_v = Vertical stress $= \sigma_c - \sigma_d$

Figure 2-4.3. Stress state in a subgrade soil element under an approaching wheel load.

- Absence of NDT Data. In the absence of any project level NDT data, it is important to develop a thorough laboratory testing program. However, in developing the testing program, the engineer must rely solely on condition survey data and historical construction records (if available). If the distresses in the pavement are of a similar nature throughout the section, the number of field samples collected can be evenly distributed throughout the project. Alternatively, if it is apparent that different distress mechanisms are evident at different locations within the candidate pavement sections, the field data collection plans must be tailored to adequately address these differences. Again, the engineer must use judgment in determining the number and location of field cores.

- Distress Type. The type of distresses evident in the section will also dictate the need for field samples. The following is a brief summary of some of the situations in which field samples should be collected for laboratory characterization. The actual number of these samples may be dictated by the previously discussed items, presence of NDT data, and uniformity of distress throughout the section.

Rutting is a common distress that often requires laboratory testing of field samples to verify the appropriate cause. There are numerous factors that contribute to rutting and, therefore, it is important for the engineer to have as much data regarding the causes of rutting prior to selecting a specific rehabilitation strategy. Rutting is generally the result of unacceptably high levels of permanent deformation in one or more of the pavement layers or underlying subgrade. It is important for the design engineers to know as much as possible about which layers are contributing to the rutting. In this instance, it is important to collect field samples at locations where excessive rutting has been observed. Cores should be taken both inside and outside of the wheelpath. The samples can then be tested in the laboratory to determine the elastic modulus of the HMA layer and the resilient modulus of the subsurface layers. If there are sections of the pavement that have not shown excessive rutting, it is recommended that cores be taken in these sections as well, so that the thickness and material properties of each layer can be compared. This is a helpful technique for identifying the weak layer or at least quanitfying the relative strength of the different layers.

Fatigue cracking is another distress that can have many underlying causes. Although it is usually expected in the later stages of a flexible pavement's life, if fatigue cracking occurs early on, accurate characterization of existing pavement layers is warranted prior to selecting a rehabilitation alternative. The premature cracking may simply be the result of an under-designed pavement structure. This can be verified with the basic strength properties of the layers combined with data on local truck traffic axle weight. However, it may also be the result of a prematurely age-hardened bituminous layer. This too can be verified or refuted by characterizing the viscosity characteristics of the asphalt in the existing pavement layers.

Other distresses (and their associated causes) such as thermal cracking, raveling, stripping, bleeding, and shoving can also be quanitified by accurately evaluating field samples in the laboratory. Combining the information obtained from laboratory tests with distress survey data, NDT, traffic data, and historical construction records (if available), the engineer can make better decisions regarding the most appropriate rehabilitation alternative.

5. TYPICAL LABORATORY TEST METHODS

The paving materials used in the surface, base, subbase, and subgrade soil layers must each be evaluated. The function and purpose of each of these layers is very different. The naturally occurring subgrade soil is often a given. With the exception of several techniques for stabilization, the design engineer has little control over the base properties of this material. In many parts of the United States, engineers are left to deal with poor subgrade materials.

On the other hand, the design engineer often has much greater control over the quality of the subbase, base, and surface layers. These layers serve to protect the subgrade soil and other underlying layers from being overstressed. When the integrity of the surface, base, or subbase is diminished, the subgrade soil and other layers will be subjected to stresses that are higher than those developing under intact material and distress will develop at an accelerated rate.

The primary functions of the pavement layers above the subgrade are as follows:

- <u>Surface</u>. Provide a smooth-riding, textured surface for the projected traffic loadings.

- <u>Base</u>. Provide protection for the subbase/subgrade soil, provide a strong support layer for surface placement, and provide for drainage of moisture out of the pavement.

- <u>Subbase</u>. Provide protection for subgrade soil and provide a working platform for subsequent construction activities.

Many of the tests that are used in initial mixture design and QC/QA during construction can also be used to evaluate materials that have been in service. However, there are many new tests that may be of more use to the designer in evaluating the existing pavement and determining the most appropriate rehabilitation strategy. The following sections briefly summarize these test procedures.

Subgrade Soil

The subgrade soil has a tremendous impact on pavement performance. Unstable subgrade soils present problems in placing and compacting base and subbase materials and in providing adequate support for subsequent HMA paving operations. The latter is important for final paving and construction operations. Without an adequate “working platform,” critical pavement construction details may not be accomplished within acceptable tolerances. Frequently, such construction deficiencies are undetected because they are “hidden” in the finished pavement. However, they may cause deterioration in the pavement after exposure to traffic and the environment.

Pavement structural responses (stresses, strains, deflections) are also highly dependent on the support provided by the subgrade soil. A large percentage of the surface deflection of a pavement is a direct result of the support provided by the subgrade soil. With surface deflection a design criterion in many overlay design methods, the need for accurate subgrade soil characterization is obvious to ensure adequate rehabilitation design.

Desirable properties of a subgrade soil include adequate shear strength, adequate permeability, ease and permanency of compaction, volume stability, and durability. Unlike some engineering materials (e.g., steel), it is difficult to assign one strength or stiffness value to the subgrade soil. Along a project, roadbed soil properties can be highly variable. In addition, soil properties change with changes in moisture, density, and confining pressure. Due to this variability, it is desirable in the analysis and design stages to conduct an extensive survey to determine the subgrade soil properties as they change throughout the year (although this step is rarely taken). It is then up to the designer and individual agencies to address the variability that is encountered in the design process.

Traditional pavement design procedures have considered the effect of the subgrade through empirical strength indicators, such as the California Bearing Ratio (CBR) (AASHTO T193-92) or the Penetration

Test (AASHTO T206-87). These are empirical tests and are only useful when combined with practical experience with local material and environmental conditions. The lower these values, the more pavement thickness is required to protect the subgrade soil from being overstressed. Current design procedures, however, have recognized that these traditional strength indicators are not inherent properties of the subgrade soil, nor are they directly related to the performance of a pavement structure once it is constructed. Rather, it has been observed that the stress-strain characteristics of the subgrade soil are more closely related to performance and the development of distress. The stress-strain characteristics of a subgrade soil can be evaluated using the resilient modulus test (AASHTO T294-92). This test has become widely-accepted as the most appropriate way of characterizing subgrade soil materials for both new and rehabilitation design.

In 4R projects, subgrade soil evaluation starts with an estimate of in situ properties, goes through an analysis of design assumption, and proceeds to the selection of subgrade soil design values for the 4R project. This process determines if the subgrade soil has provided the support that was assumed during design. If the subgrade soil has performed as expected, the design assumptions were correct and can probably be used again. If, however, the subgrade soil has not performed as assumed, then the design assumptions may have been incorrect. The subgrade soil may have been the cause of the deterioration that has necessitated the 4R project and new subgrade soil design parameters may need to be determined.

In a 4R project, the location of the project is fixed and the subgrade soil is already in place. This implies that the subgrade soil evaluation efforts could be done more effectively than in new construction. The identification of subgrade soil properties for 4R design should account for the variability of the subgrade soil along the length of the project, its variability by depth, and its projected variability over the 4R design period.

Resilient Modulus

Recent years have seen a movement to the use of the resilient modulus value to characterize subgrade soils and unbound base materials. The resilient modulus test provides a material property that more closely simulates the behavior of the soil under a moving wheel. A moving wheel imparts a dynamic load pulse to all pavement layers and the subgrade soil as previously illustrated in figure 2-4.3. In response to the applied compressive stress, each pavement layer deflects. The stress pulse builds from a low value to a peak value over a brief period that is related to the speed of the vehicle.

The moving loads produce deformation, as shown in figure 2-4.4, that can be used to calculate the following deformation properties of the soil:

- Total Strain. Total deformation under the load, ε_t.
- Resilient Strain. Deformation recovered when the load is removed, ε_r.
- Permanent (Plastic). Strain deformation not recovered when the load is removed, ε_p.

The resilient modulus test is conducted by placing a compacted soil specimen in the triaxial cell, as shown in figure 2-4.5. The specimen is subjected to an all around confining pressure, σ_c, and a repeatedaxial stress (deviator stress), σ_d, is applied to the sample. The number of times the axial load is applied

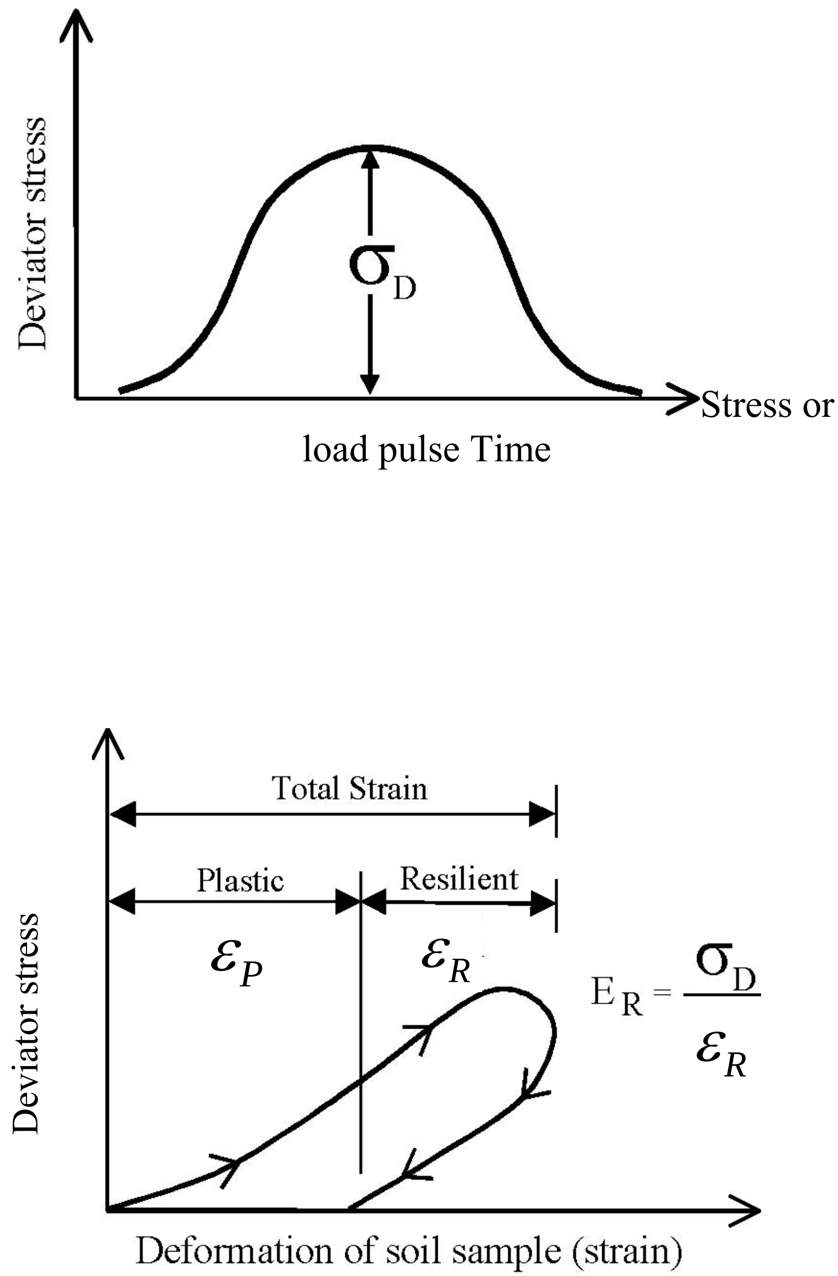


Figure 2-4.4. Typical repeated load response.

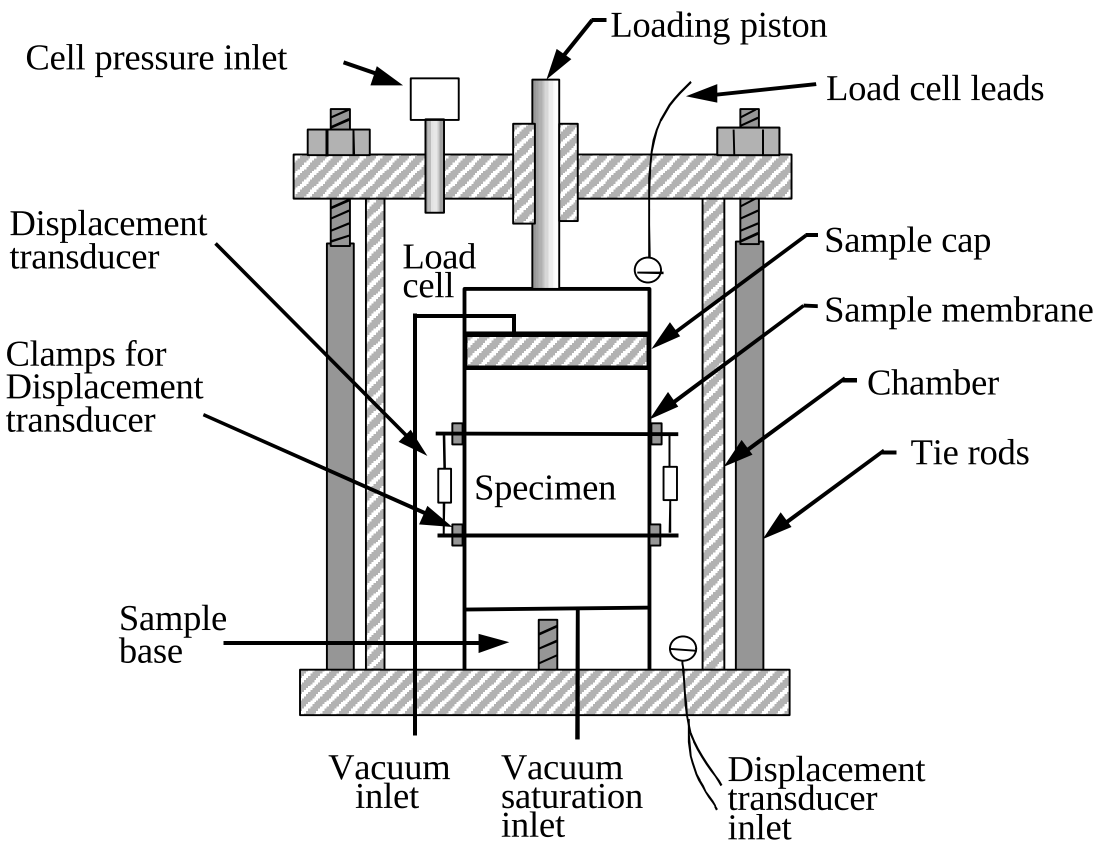


Figure 2-4.5. Subgrade resilient modulus test apparatus.

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 resilient modulus is determined using equation 2-4.1 for a number of deviator stress and confining pressures:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{2-4.1}$$

where:

M_r = Resilient modulus

σ_d = Deviator stress (σ_v - σ_c)

σ_v = Total vertical stress

ε_r = Recoverable strain

Typical curves resulting from the resilient modulus testing of a cohesive soil sample are shown in figure 2-4.6.[7] This figure illustrates how increased deviator stresses serve to decrease the resilient modulus, a phenomenon known as "stress softening." This phenomenon is common of most fine-grained, cohesive materials.

Figure 2-4.6 also shows the effect of confining pressure on the resilient modulus. As the confining pressure increases, the resilient modulus value also increases due to the increased lateral support to the sample. The resilient modulus value appears to be less sensitive to confining pressure at higher levels of deviator stress.

There are other factors that affect the resilient modulus value in addition to the confining pressure and the deviator stress. Perhaps primary among these factors is the degree of saturation of the material; a more saturated material will have a lower resilient modulus.

The many factors that affect the resilient modulus value make it difficult to select a single resilient modulus value for use in rehabilitation design. It is generally recommended that resilient modulus values be selected from the test conditions most expected to be encountered in the field. For example, work done by the University of Illinois concluded that the resilient modulus stress dependency for Illinois soils could be adequately characterized by two intersecting straight lines whose slope did not vary significantly as seen in figure 2-4.7.[8] The point of intersection (E) always occurred around 41 kPa.[8]

The confining pressure and moisture contents used in the resilient modulus testing should be representative of actual field conditions. Typically, this may mean confining pressures of 21 kPa and saturated conditions (wet of optimum).[7] However, some advocate the use of unconfined testing (0 kPa), because the effect of confining pressure is low on saturated samples, because it provides a conservative estimate of the resilient modulus, and is a quicker test to perform.[8]

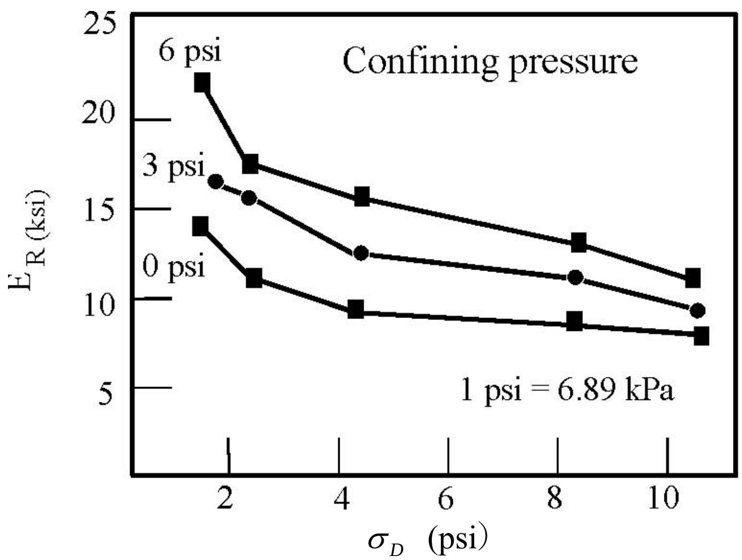


Figure 2-4.6. Resilient modulus curves for various confining pressures.[7]

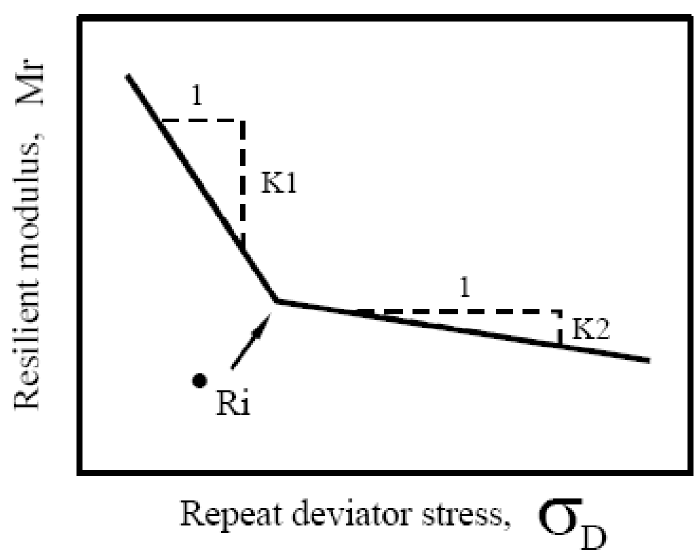


Figure 2-4.7. Idealized resilient modulus curve for a fine-grained, cohesive soil.[8]

The resilient modulus set up can also be used to develop data about the rutting potential of the materials by recording the permanent deformation produced by the repeated loadings. Typical permanent strain-to-number of stress repetitions responses are shown in figure 2-4.8. Permanent strain accumulates very rapidly when the repeated stress is large in relation to the strength of the soil.

Resilient modulus testing can also be conducted on granular materials; however, the resulting relationships are much different from fine-grained, cohesive soils. While granular materials are also stress-sensitive, they exhibit “stress hardening” characteristics, in which the resilient modulus increases with increasing stress states. This is due to increased interlock between the individual aggregate particles. The resilient modulus as a function of the applied stress state in granular materials is given by the following equation:

$$M_r = k\theta^n \tag{2-4.2}$$

where:

M_r = Resilient modulus
k, n = Experimentally-derived factors
θ = Bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$
σ_1 = Major principal stress
σ_2, σ_3 = Intermediate and minor principal stresses

Figure 2-4.9 illustrates the resilient modulus for a sandy gravel as a function of the bulk stress.

The standard procedure for resilient modulus testing was revised recently. At the time of this writing, two test methods were available: AASHTO T292-91 I (SHRP Protocol P46) and AASHTO T 294-92 I.

Other Tests of Subsurface Layers

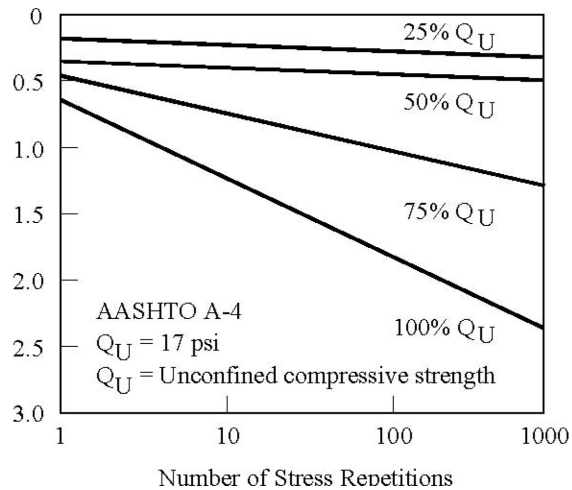
This section presents some of the tests commonly used in evaluating the subsurface layers (subgrade soil, subbase, and base). The tests can be broken into material constituency tests, strength tests, and performance-based tests used in pavement design procedures. Each series of tests has its own area of applicability and it is important that each test be conducted on representative material samples.

Sampling of soils to be tested can be accomplished according to the ASTM D1596-84 procedure for split-spoon soil sampling. The split-spoon sampler has an outside diameter of 50 mm, inside diameter of 35 mm, and is effective in obtaining representative samples with a minimum of disturbance. The split-spoon sampling is most suited for the sampling of fine-grained soils. Shelby tubes may also be used for soil sampling. Granular materials may be sampled using augers, trenches, and cores.

Material Constituency Tests

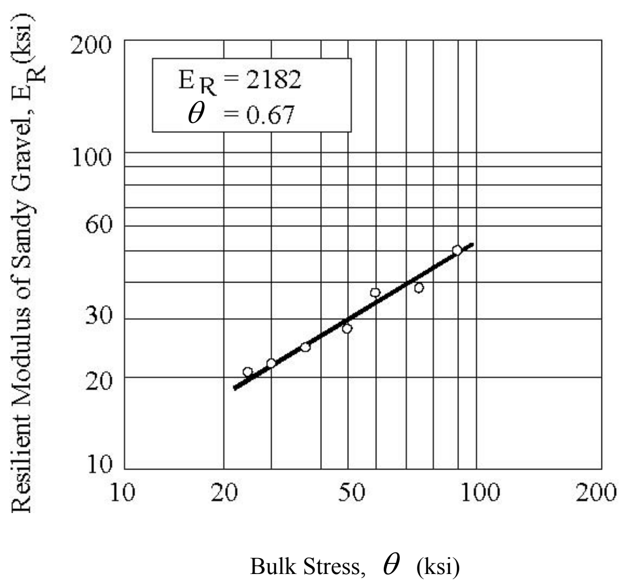
Material constituency tests measure intrinsic properties of the materials that may affect performance indirectly. The tests that are normally conducted in this category include:

Moisture content.
Density.
Gradation.
Plasticity characteristics



1 psi = 6.89 kPa

Figure 2-4.8. Effect of load magnitude and repetitions on permanent strain (AASHTO A-4[9] soil).



1 psi = 6.89 kPa

Figure 2-4.9. Resilient modulus vs. bulk stress for a sandy gravel (AASHTO A-1-b[0]).

These tests are run to show whether the properties of the materials have changed since construction. Construction records containing original test results may be compared with the present condition of each material. Any significant changes in the properties of the subgrade may be indicative of a problem in the subgrade.

Strength Tests

There are various tests that can be run to measure material strength or its ability to resist deformation or bending. These tests include:

- California Bearing Ration (CBR).
- Hveem Resistance Value (R-value).
- Plate load test (*k*-value).
- Triaxial testing.
- Dynamic cone penetrometer (DCP).
- Resilient modulus.
- Compressive strength (stabilized materials or cohesive soil).
- Indirect tensile strength (stabilized materials).

California Bearing Ratio (CBR)

The CBR test measures the resistance of soil to penetration by a piston with an end area of 1935 mm^2 being pressed into the soil at a standard rate of 1.3 mm per min. A schematic of the test and typical data are shown in figure 2-4.10. 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 penetration), multiplied by 100, is the CBR of the soil. CBR values will range from 2 to 3 for a silty-clay material to 70 or higher for a high quality granular material (see figure 2-1.10).

Fine-grained cohesive soils are generally compacted at the optimum moisture content for testing. Granular materials are generally compacted at several moisture contents above and below optimum. The samples are soaked for 96 hours prior to testing to simulate saturated conditions that may develop under the pavement. Weights may be added to the surface of the sample to simulate overburden pressures of a pavement structure.

The CBR test is an empirical test that has been used extensively in pavement design. It is important that the test be conducted in strict accordance with the AASHTO T193 procedure if results for different soils are to be comparable. The major advantages of this test are the simple equipment requirements and the large amount of data available for correlating results with field performance. A major drawback is that this test procedure cannot be conducted on materials prepared to conditions approximating field conditions to develop any comparison with existing data. The CBR test can be performed in the field in accordance with ASTM D4429.

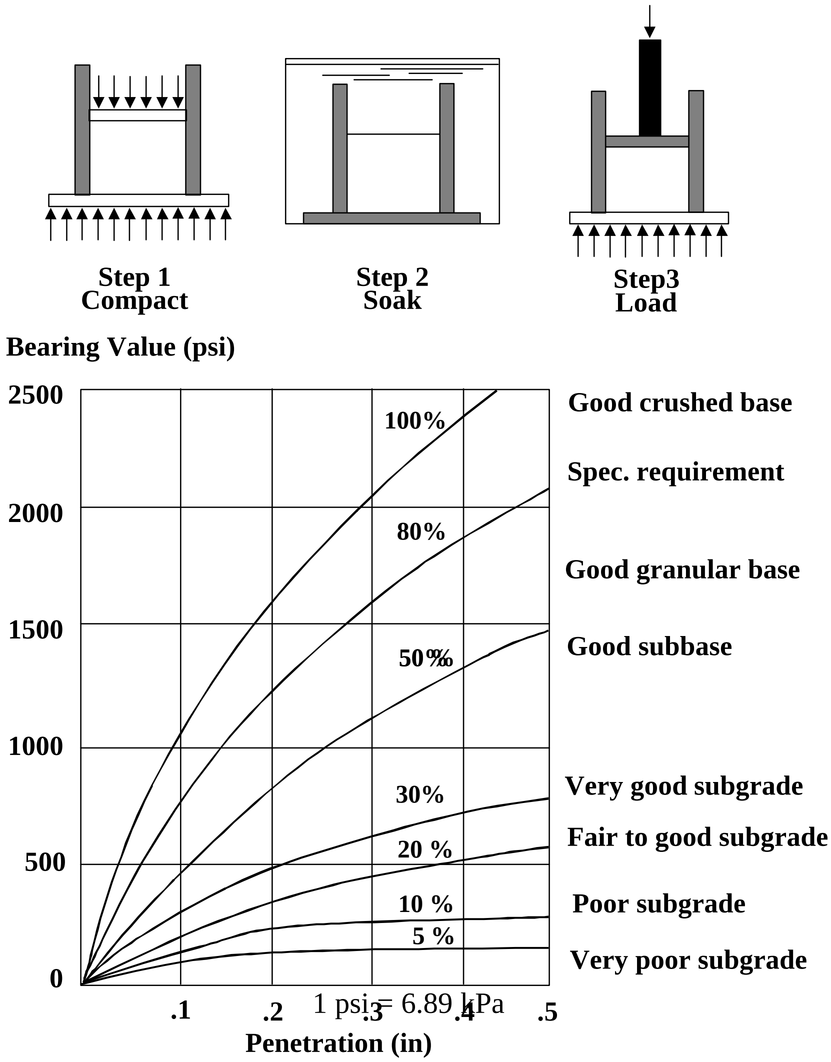


Figure 2-4.10. CBR testing procedures and load penetration curves for typical soils.[3]

Hveem Resistance Value

The R-value (Resistance Value) is obtained from the test conducted in accordance with ASTM D 1560. A cylindrical sample 100 mm in diameter and 6 mm tall is enclosed in a membrane and loaded vertically over the full face of the sample to a given pressure. The resulting horizontal pressure is measured and used to calculate the R-value. The R-value gives an idea of the stiffness of the material. The R-value method has been used most frequently in the Western States. The test result is empirical and does not represent a fundamental soil property. The results cannot be used with any analytical evaluation of the structural adequacy of the material in the pavement.

Plate Load Test

The plate load test is performed to obtain the modulus of subgrade reaction, often termed the *k*-value. The *k*-value is the primary subgrade design value for rigid pavements. In this test, a stack of steel plates, with the largest measuring 760 mm in diameter, is placed on the soil in a test pit and loaded. The resulting deflection of the plate is measured. The process is repeated for several load levels and the results are plotted on a graph of load versus deflection. The *k*-value is calculated as the pressure on the plate divided by the deflection of the plate (psi per inch or pci). There are two procedures to calculate the *k*-value that can yield different results. One procedure uses the pressure at a deflection of 1.3 mm, while the other uses the deflection at a pressure of 69 kPa. Typical *k*-values may range from 27 kPa per mm to 54 kPa per mm for granular materials and up to 136 kPa per mm for stabilized materials.

This test is conducted in the field directly on the subgrade or on the existing base course materials. For rehabilitation work, it requires removal of the pavement layers to gain access to the base or subgrade. However, the plate load test takes a long time to run, requires expensive equipment, and reflects only the strength of the soil at the in-place moisture content and density. Thus, it is more common for the *k*-value to be estimated from other material test results and layer analysis procedures, or for it to be obtained from the backcalculation of FWD data. See references 20 and 21 for the most recent and comprehensive work in this area.

Triaxial Testing

The triaxial test is a compression test in which a 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 in the materials when they are in the pavement. 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. Texas and Kansas are two States that have design procedures based on this test.

Dynamic Cone Penetrometer (DCP)

The DCP is a device for measuring the in situ strength of paving materials and subgrade soils. Although the DCP was introduced in the 1960s, highway agencies are just now becoming familiar with it or are starting to use it, although it is extensively used by the United States Air Force. Currently, DCP testing procedures are being developed in an ASTM subcommittee. The DCP penetration rate (PR) can be used to identify pavement layer boundaries and subgrade strata, and to estimate the CBR values of those layers. Figure 2-4.11 shows the DCP apparatus.[4]

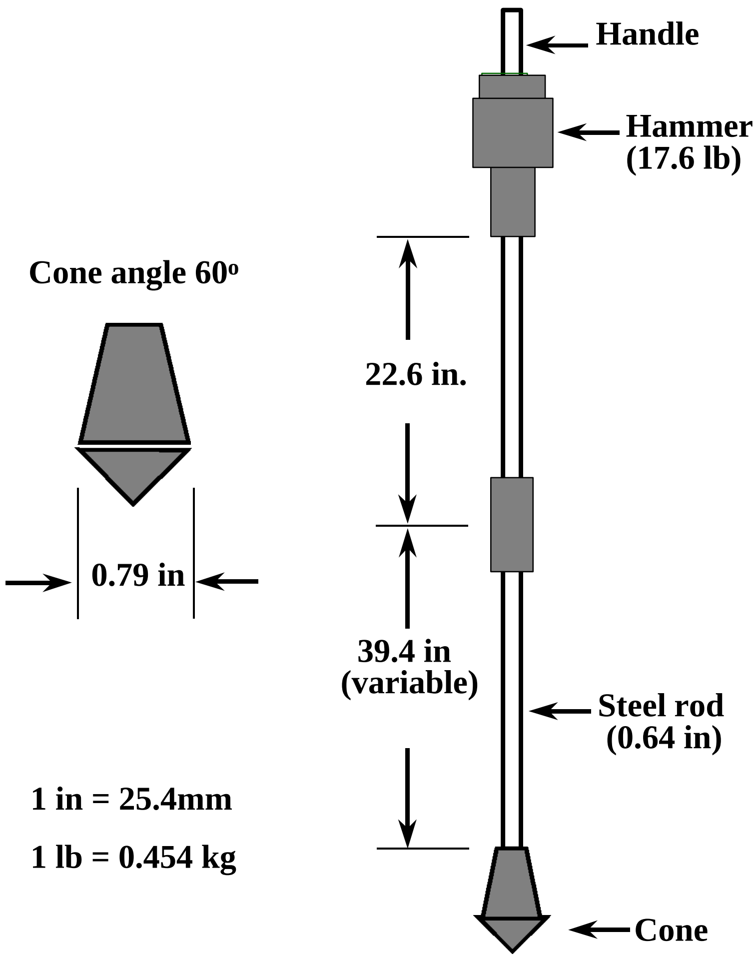


Figure 2-4.11. Dynamic cone penetrometer.[4]

The DCP test is performed by driving the cone into the pavement/subgrade by raising and dropping the hammer. The cone penetration is recorded for each drop and termed the penetration rate. After the test is done, the soil CBR values can be determined using the following equations:[5,6]

$$CBR = \frac{405.3}{PR^{1.259}} \quad for \quad 60^{\circ}\, cone \tag{2-4.3}$$

$$Log\ CBR = 2.20 - 0.71\,(\log DCP)^{1.5} \quad (for\ \ 30^{\circ}\, cone) \tag{2-4.4}$$

where:

PR = Penetration rate measured in mm per blow
DCP = Penetration rate measured in inches per blow

Surface Layer Tests

Pavement surface layers are either HMA or PCC. The tests for these materials must determine the ability of the material to withstand load stresses. For example, PCC is generally characterized by either a compression or a flexural strength, but other properties, such as durability, should also be determined. HMA materials are characterized by fatigue, resilient modulus, creep properties, and also by more mix-specific properties (such as gradation and certain asphalt qualities).

HMA Resilient Modulus

Resilient modulus testing can also be conducted on HMA and asphalt-stabilized materials. However, the test for asphalt concrete is performed in a different manner from that of the subgrade soil or granular material. The test procedure normally used for determining the HMA resilient modulus is the indirect tensile test mode with a pulsed load (ASTM D 4123). The HMA resilient modulus is calculated using the deformation across the horizontal diameter of the core, using equation 2-4.5. This test can be run at varying temperatures. The test speed of loading must also be considered when using this result in a structural evaluation.

$$M_r = \frac{P(0.2734 + \mu)}{t\Delta h} \tag{2-4.5}$$

where:

μ = Poisson's ratio, typically assumed to be 0.35 for asphalt concrete
Δh = Deformation across the horizontal diameter, in
t = Thickness of the core, in
P = Maximum load on the sample, lb

1 lb = 0.454 kg 1 in = 25.4 mm

The resilient modulus values for the subgrade, granular base, and asphalt concrete represent the elastic stiffness values typically required for some structural evaluation methods. The stiffness values of the materials directly affect the deformation, stresses, and strains that develop in the pavement layers under a load.

Marshall Stability and Flow Test

The test is performed in accordance with ASTM D 1559. It involves coring a 100 mm diameter sample from the existing HMA surface and trimming it to 65 mm high. The two principal features of the Marshall test are stability-flow and density-voids properties.

The stability value is the maximum load resistance in pounds that a specimen will develop at 60° C. The flow value is the total movement or strain, in units of 0.25 mm, occurring in the specimen between no load and maximum load during the stability tests. After the completion of the stability and flow tests, mix density and air voids are determined. The maximum theoretical specific gravity must also be determined.

Performing the Marshall stability and flow test can aid in the evaluation of an HMA pavement. Air voids have been found to be critical to rutting development, and the information is needed if recycling is considered as a rehabilitation option. Furthermore, it is possible that correlations may be identified between certain mix properties and the development of pavement distress.

Extraction Test

This test is performed in accordance with ASTM D 2171. The bitumen is separated from the aggregate so that the bitumen content and aggregate gradation can be determined. Properties of the bitumen (viscosity, penetration, and ductility) can also be identified by testing the recovered asphalt. These properties are especially needed when recycling is considered as a rehabilitation strategy.

Unconfined Compression Test

The unconfined compression test can be performed on all stabilized materials used in pavement construction. A cylinder of material with a height-to-diameter ratio of 2 to 1 is placed in the test apparatus. The sample is loaded at a constant rate of deformation, typically 1.3 mm per min. The load on the sample and the deformation are recorded. The data are normally plotted as stress (load per cross section area) versus strain (deformation per original length). This test result is not used directly in design, but can be correlated with the flexural strength and the elastic modulus.

Indirect Tension Test

The indirect tension test, also called the splitting tensile test, involves applying a vertical load on the diameter of a sample, as shown in figure 2-4.12. The load is applied at a constant rate of deformation, 1.3 mm per min. The sample will fail in tension along the vertical diameter of the sample and the indirect tensile strength is calculated from the following equation:

The DCP test is performed by driving the cone into the pavement/subgrade by raising and dropping the hammer. The cone penetration is recorded for each drop and termed the penetration rate. After the test is done, the soil CBR values can be determined using the following equations:[5,6]

$$CBR = \frac{405.3}{PR^{1.259}} \; for\ 60^{o}\ cone \qquad (2\text{-}4.3)$$

$$Log\ CBR = 2.20 - 0.71\,(\log DCP)^{1.5} (for\ 30^{o}\ cone) \qquad (2\text{-}4.4)$$

where:

PR = Penetration rate measured in mm per blow
DCP = Penetration rate measured in inches per blow

Surface Layer Tests

Pavement surface layers are either HMA or PCC. The tests for these materials must determine the ability of the material to withstand load stresses. For example, PCC is generally characterized by either a compression or a flexural strength, but other properties, such as durability, should also be determined. HMA materials are characterized by fatigue, resilient modulus, creep properties, and also by more mix-specific properties (such as gradation and certain asphalt qualities).

HMA Resilient Modulus

Resilient modulus testing can also be conducted on HMA and asphalt-stabilized materials. However, the test for asphalt concrete is performed in a different manner from that of the subgrade soil or granular material. The test procedure normally used for determining the HMA resilient modulus is the indirect tensile test mode with a pulsed load (ASTM D 4123). The HMA resilient modulus is calculated using the deformation across the horizontal diameter of the core, using equation 2-4.5. This test can be run at varying temperatures. The test speed of loading must also be considered when using this result in a structural evaluation.

$$M_r = \frac{P(0.2734 + \mu)}{t \cdot \Delta h}$$

where:

μ = Poisson's ratio, typically assumed to be 0.35 for asphalt concrete
Δh = Deformation across the horizontal diameter, in
t = Thickness of the core, in
P = Maximum load on the sample, lb

1 lb = 0.454kg 1 in = 25.4 mm

$$\sigma = \frac{2P}{\pi LD} \quad (2\text{-}4.6)$$

where:

σ = Indirect tensile strength, kPa
P = Vertical compressive force, N
L = Length of sample, mm
D = Diameter of sample, mm

This test is conducted on stabilized materials, HMA, and PCC. It is a particularly valuable test for rehabilitation testing, as it is performed on cores taken from the pavement as well as on samples prepared in the laboratory. Relationships between the flexural strength and the indirect tensile strength have been developed for PCC and are available from a number of sources. One typical estimate is that the indirect tensile strength is about 70 percent to 80 percent of the flexural strength.

Using a structural model, it is possible to measure surface deflections from various types of loading devices (Benkelman Beam, vibratory testing, and FWD) and backcalculate the subgrade M_r or the *k*-value. The values selected are those that match the observed surface deflection to the calculated surface deflection from the computer model.

It is important to recognize that the various surface deflection measurement approaches and structural models do not calculate or use the same values in the same manner. The modulus values for the granular and surface layers can be obtained in much the same manner, although the accuracy of their determination from deflection basins is more questionable at present. Measured or assumed pavement material properties and layer thicknesses are required inputs for the structural models. If layer thicknesses are known, subgrade and pavement material moduli values can be estimated from surface deflection measurements. Furthermore, the backcalculated M value of the subgrade soil can be three tosix times higher than the laboratory value.

The use of NDT is particularly advantageous, since the technique is nondestructive, and many field tests can be conducted quickly and easily to provide an extensive database. It is often helpful to correlate the backcalculated support values with values measured from field sites and laboratory work.

6. MATERIAL PROPERTY RELATIONSHIPS

The material property tests discussed in the previous section provide an indication of the present ability of the pavement materials to sustain both traffic and environmental loadings. Not all of the tests can or should be conducted on any one rehabilitation project. An agency should run only those tests that provide the most useful data for their rehabilitation evaluation.

Over time, materials undergo changes in their properties. Intrusion of subgrade soil fines into a granular base course can lower its shear strength, CBR value, and resilient modulus. They also can have an adverse effect on the layer's permeability. Loss of density in the subgrade soil or base due to climatic influences greatly alters the support potential as indicated by the M_r and possibly the CBR. Changes in the moisture condition of all paving materials are perhaps the most detrimental changes that can occur as moisture has a tremendous impact on the structural behavior of most subgrade soils.

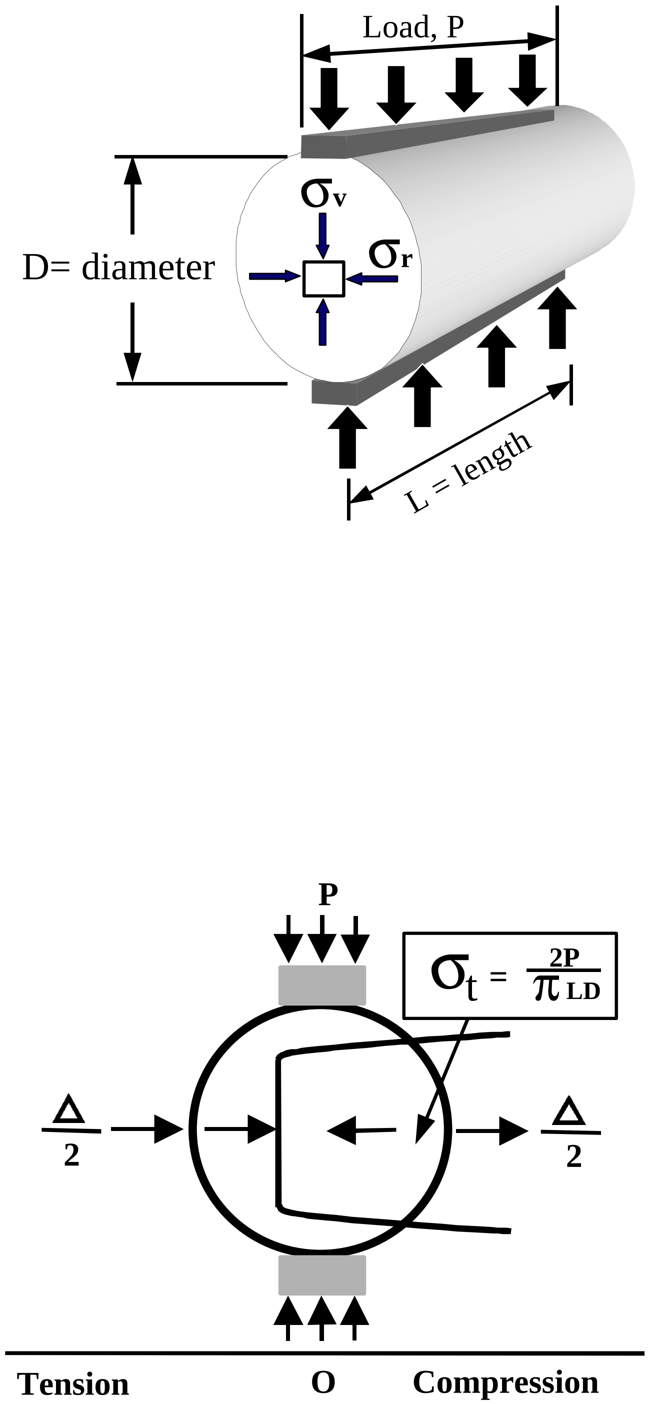


Figure 2-4.12. Indirect tension test.

Properties such as gradation, moisture, and density are easily determined and provide the only direct indication of how the material has changed from the time it was constructed. These tests are described in the next sections and should be used in conjunction with other material tests (e.g., resilient modulus) in order to fully characterize the properties of a material.

Soil Classification

The AASHTO method of soil classification uses the grain size distribution and the plasticity characteristics of the soil to differentiate among soils based on their potential to perform as a subgrade soil under a pavement structure. Other soil classifications in use include the Unified Soil Classification system, the Federal Aviation Administration (FAA) classification system, pedologic classifications, and the United States Department of Agriculture classification system.

Different soil classifications have been related to the traditional strength parameters. Figure 2-4.13 presents one set of correlations among soil types and several measures of strength. It is important to realize that the relations between soil types and soil strengths are not precise and a wide range of strengths are possible for a given soil type. For instance, a soil with a CL designation in the unified system could have a CBR of 5 to 15.

Moisture and Density

The moisture content and density relationship for a soil is a critical factor affecting the strength and deformation properties of any prepared soil. It must be remembered that each soil has an individual moisture-density relationship that must be determined in the laboratory. Figure 2-4.14 illustrates some typical compaction curves for a variety of soils.[3] During construction, the subgrade soils and pavement materials were compacted to a certain density at a specified moisture content. This combination provided the strength values assumed in the original pavement design. However, both the moisture content and density can change after construction.

Some agencies have developed moisture-density curves for all of their known soils. This allows them to identify the moisture-density relationship for a soil from a catalog after only determining one moisture-density point. Such one-point moisture-density curves should be used very carefully and only when developed specifically for soils in the local area.

Moisture-density relationship is a very important property of subgrade soils. Figure 2-4.15 illustrates the effect of an increase in moisture content on the permanent strain in a fine-grained soil. Figure 2-4.16 shows the dramatic effect of an increase in density on the permanent deformation of a well-graded granular material.

Cyclic freeze-thaw (without moisture change and the resultant heaving) causes drastic changes in the strength and repeated load behavior of fine-grained soils. Figure 2-4.17 shows that the resilient modulus value decreases with exposure to freeze-thaw cycles. The most dramatic decrease occurs after only one freeze-thaw cycle, whereas subsequent freeze-thaw cycles create much smaller decreases in the resilient modulus. A single freeze-thaw cycle freezes any moisture in the material, which causes expansion. Then, upon thawing, the soil particles are no longer tightly compacted and a reduction in resilient modulus occurs. Subsequent freeze-thaw cycles have a substantially smaller effect. However, a soil can regain strength lost due to freezing and thawing.

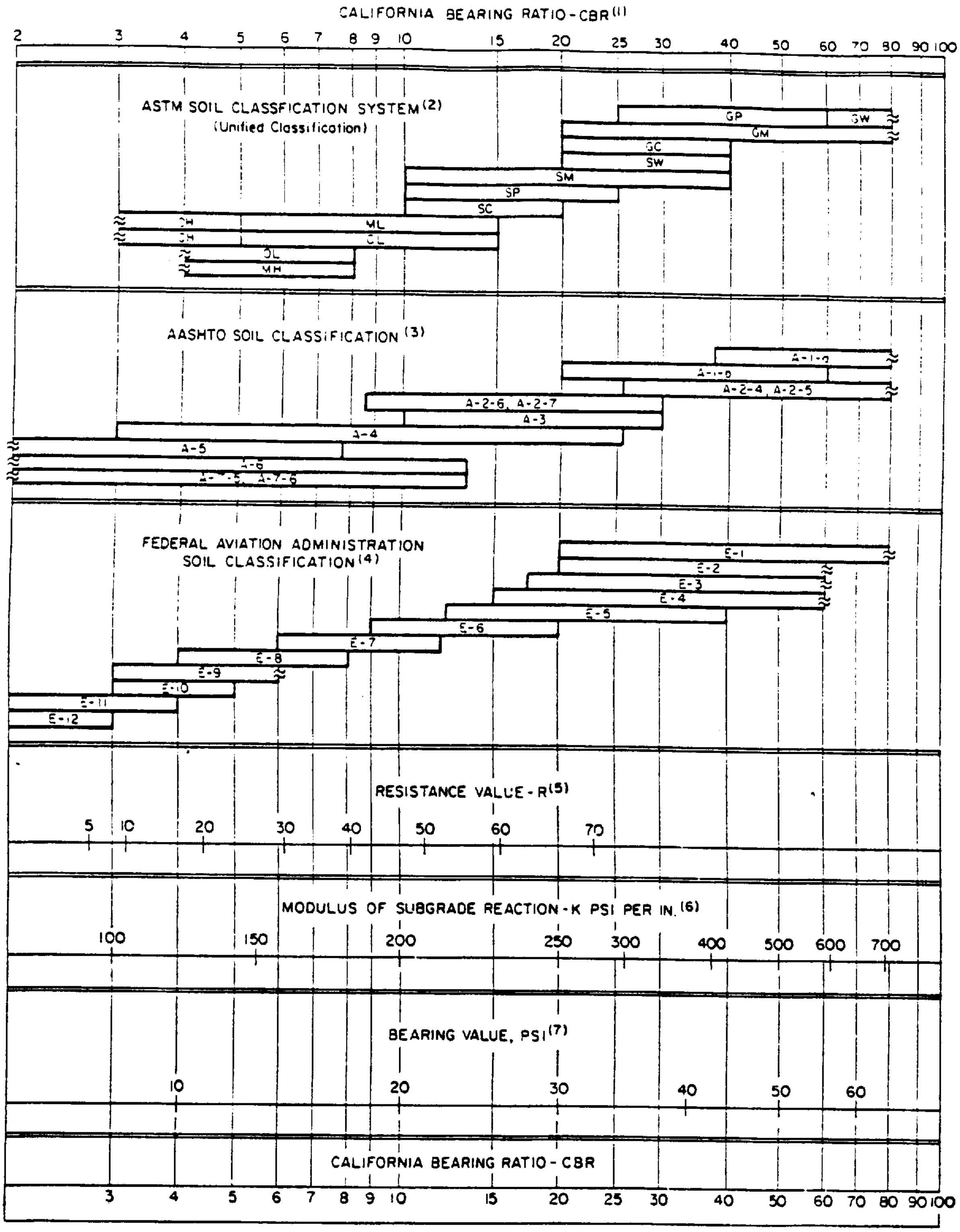
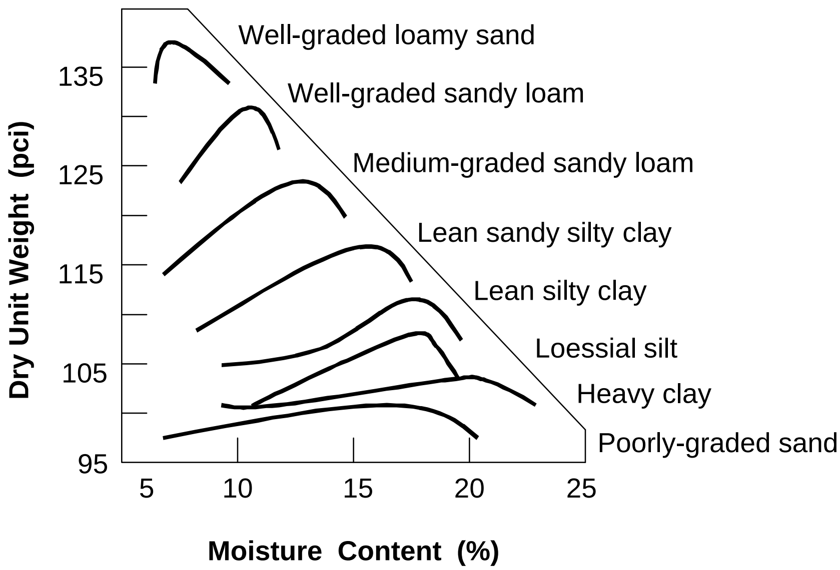
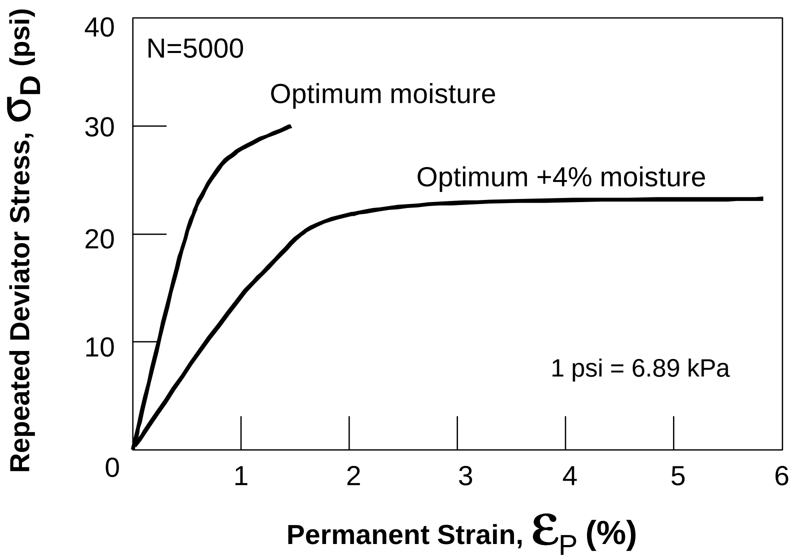


Figure 2-4.13. Soil classification related to strength parameters.[10]



1 pci = 27682.9 kg/m^3

Figure 2-4.14. Typical compaction curves for different soil types.[3]



1psi = 6.89 kPa

Figure 2-4.15. Influence of moisture content on permanent strain response of a loess-derived soil (AASHTO A-7-6[23]).

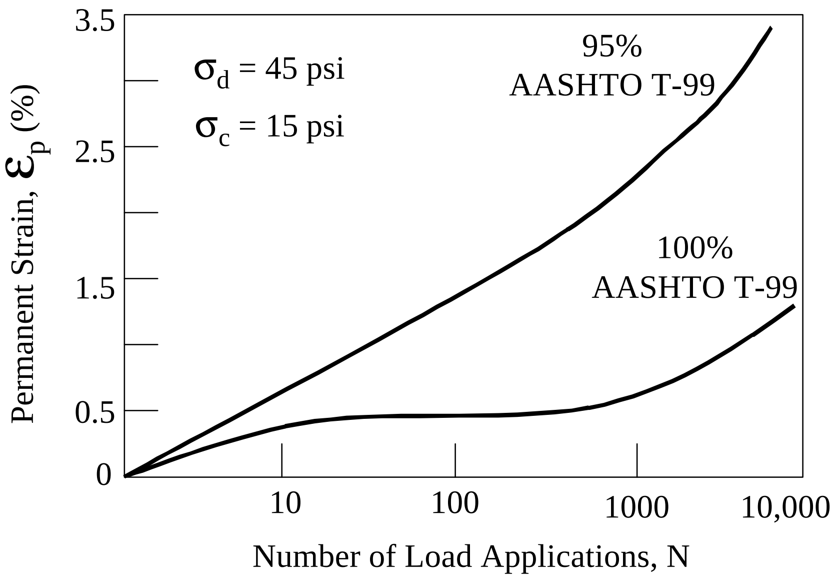


Figure 2-4.16. Permanent deformation as a function of load application for two compaction efforts for a granular material.

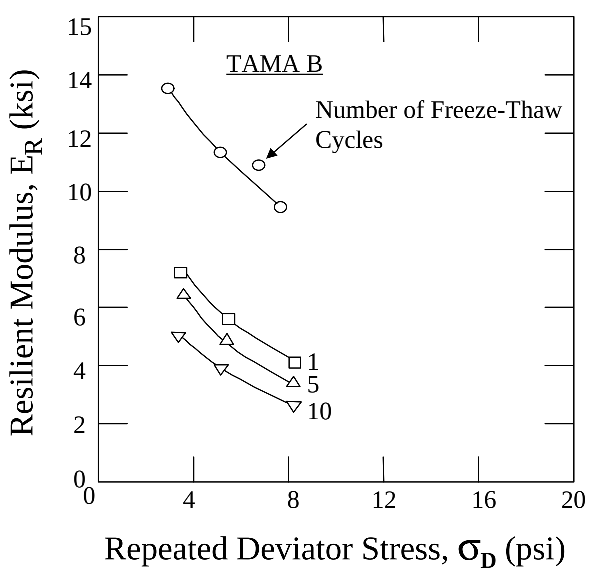


Figure 2-4.17. Influence of cyclic freeze-thaw on the resilient behavior of a fine-grained soil (AASHTO A-7-6[27]).

7. CORRELATIONS

Since not all agencies are familiar with the different material tests, and since most agencies are gradually adopting the use of the resilient modulus to characterize their subgrade soils and unbound materials, it is useful to consider correlations between some of the various material strength indicators. Figure 2-4.18 presents some approximate correlations between other materials strength indicators and the resilient modulus value.[1] The design engineer should be aware that these correlations are approximate and based on limited data; therefore, they should be applied with great caution.

An approximate correlation for the resilient modulus of subgrade materials based on the CBR is

$$M_r = B \times CBR \qquad (2\text{-}5.6)$$

where:

M_r = Resilient Modulus, kPa

CBR = California Bearing Ratio

B = Coefficient

= 750 - 3000 (1500 for CBR < 10)

An approximate correlation for the resilient modulus of the subgrade materials based on the resistance value (R-value) is:[3]

$$M_R = A + B(R) \qquad (2\text{-}5.7)$$

where:

M_R = Resilient modulus, kPa

R = Resistance value

A = Constant

= 772 - 1155 (1000 for R < 20)

B = Constant

= 369 - 555 (555 for R < 20)

Correlations are also available between the resilient modulus and soil properties such as the AASHTO classification, plasticity index, and moisture content (especially for cohesive soils). In all cases, any relationships should be applied with extreme caution.

8. SELECTION OF ANALYSIS UNITS

Figure 2-4.19 illustrates the idealized analysis unit delineation procedure discussed in the AASHTO Design Guide.[1] This procedure considers pavement type, construction history, cross section, subgrade soil, and future traffic levels. Of these factors, the subgrade soil is the most difficult to quantify and is the most variable along the highway project.

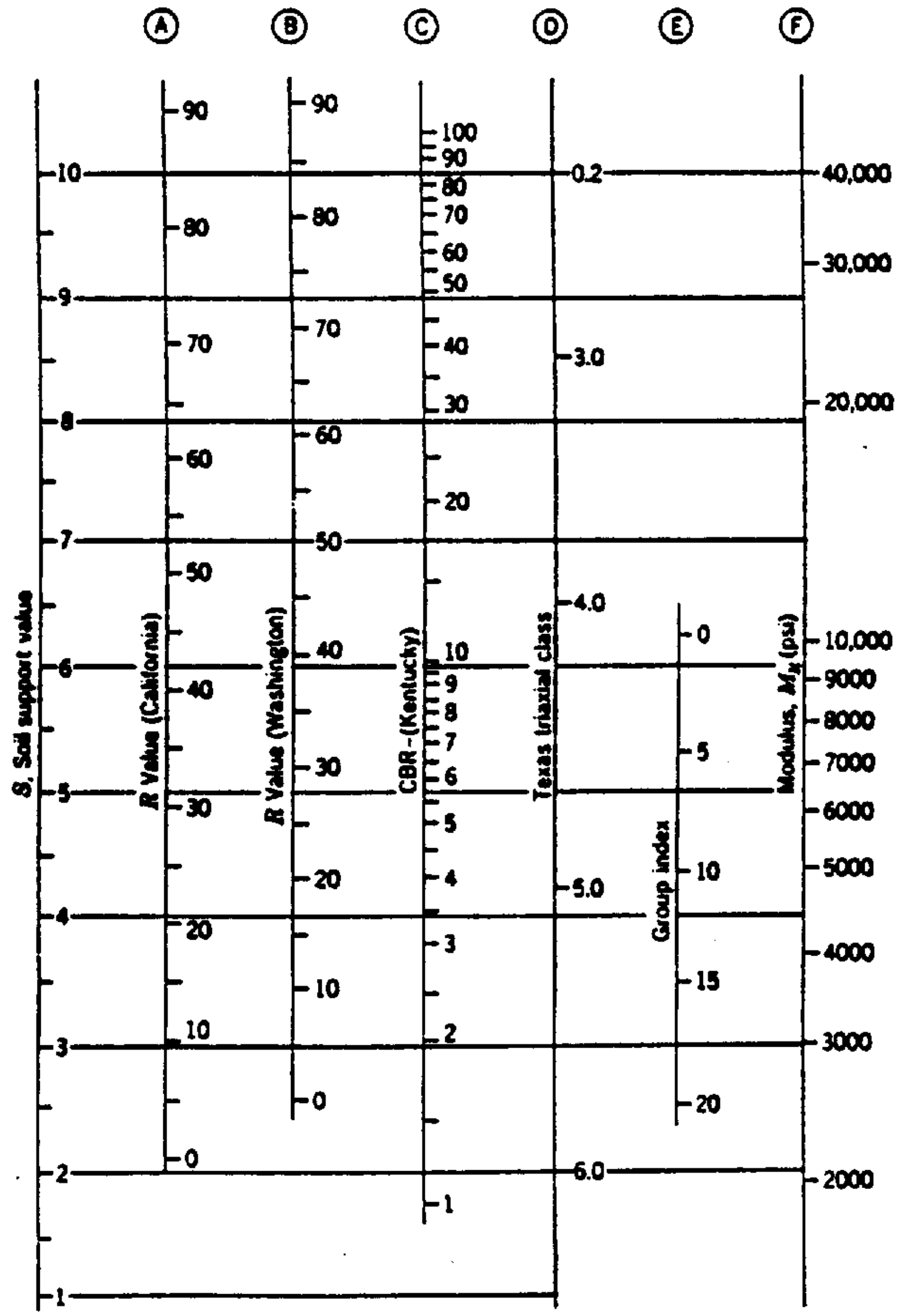
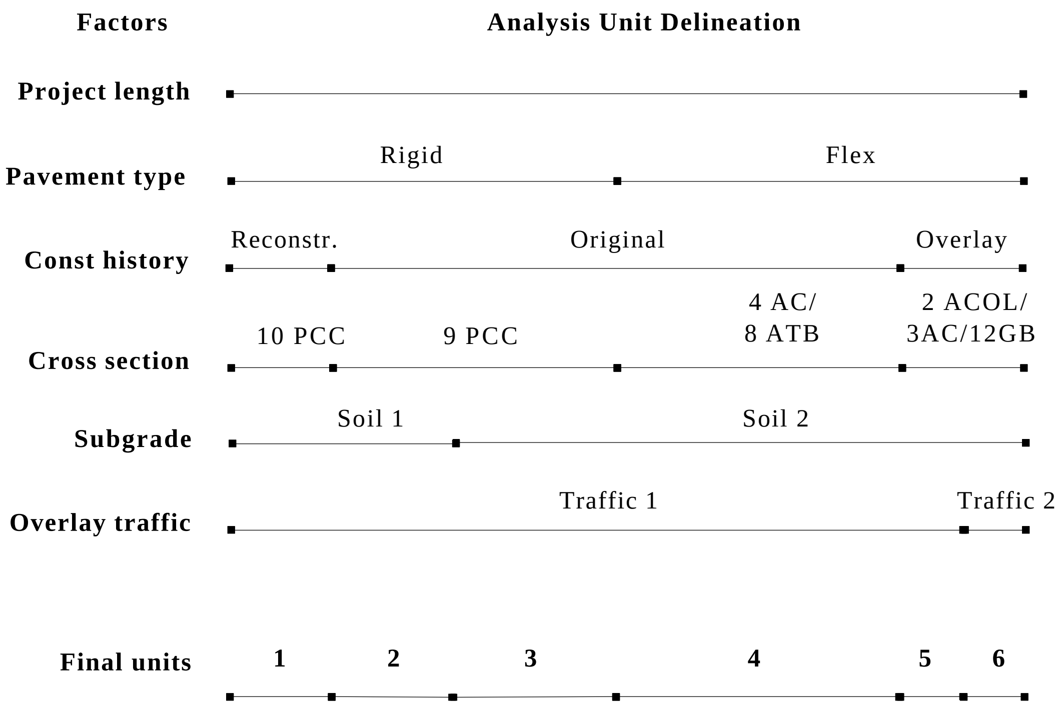


Figure 2-4.18. Correlations with resilient modulus.[1]



Unit attributes	1	2	3	4	5	6
Pavement type	R	R	R	F	F	F
Construct hist	REC-5	OR-15	OR-15	OR-15	OL-20	OL-20
Cross section	C1	C2	C2	C3	C4	C4
Subgrade	S1	S1	S2	S2	S2	S2
Overlay traffic	T1	T1	T1	T1	T2	T2

Figure 2-4.19. Idealized method for analysis unit delineation.[1]

The first step in the evaluation of the subgrade is to determine what problems are occurring in the structure. A distress survey helps to delineate analysis units and provides clues as to whether the pavement distress is due to structural inadequacy, materials deterioration, or poor subdrainage. If structural or moisture-related distress is developing, the subgrade soil needs to be thoroughly evaluated to determine if it is failing to give the required support or if it is contributing to volume-change problems. It is helpful to plot the type and amount of distress on a strip map of the project to determine if there are localized problem areas or if the whole project is performing in generally the same manner.

Many sources of information are available to assist the pavement engineer in evaluating the different subgrade soil properties that may need to be considered in the design of the rehabilitation strategy selected without an intensive field testing program. These include:

- Previous engineering "soil reports" for the project.

- As-built plans.

- Materials testing information.

- Pedologic soil data.
  - United States Department of Agriculture County soil reports.
  - State soil manuals based on "pedologic soil type."
  - Soil association maps.
  - Unpublished surficial soils data available from Soil Conservation Service Office and land grant universities.

- Climatic information.

- Geological data (Federal and State geological surveys are excellent contacts).

- Climatic information.
  - National Oceanic and Atmospheric Administration (NOAA).
  - Agricultural experiment stations.
  - State water surveys.
  - Federal geological survey.
  - Weather atlases.
  - Moisture-accelerated distress zones.[2]

Since the subgrade is already in place, a soil study may have been completed before the pavement was originally constructed. That information should be obtained and used as the starting point for 4R subgrade soil evaluations. Soil parameters, material types, and thickness profiles should be plotted along the project strip map for comparison with the strip chart of the distress. This side-by-side comparison will provide insight as to whether the problems are subgrade/soil related.

The most concise and complete collection of subgrade soil data at the project level will be contained in the as-built plans and material testing records from the original construction if they are available. If these sources of information are not available, then an alternative source of subgrade soil information is the soil surveys produced by the Soil Conservation Service of the United States Department of Agriculture. They include complete information on engineering properties of the soils, such as:

- Soil classification (AASHTO, Unified).
- Grain size distribution.
- Atterberg limits.
- Shrink-swell potential.
- Water table location.
- Frost action potential.
- Average daily maximum and minimum temperature by month, and average monthly precipitation for each major soil type in the survey area.
- Permeability, drainage characteristics, and infiltration capacities.

The project location is clearly shown on these maps and the different soil types present can be determined with enough precision to satisfy a preliminary survey and to establish limits on the site. The soil types can be compared with the distress survey and previous soil studies to determine potential subgrade soil problem areas. The deflections obtained from FWD testing should also be used to help in determining potential problem areas. The different problem areas provide an indication of where sampling should be concentrated to develop the most data for the least expenditure of money and time.

Detailed Survey Per Testing

The information developed in the preliminary survey must be analyzed to show where problems may exist that require detailed material property descriptions to be developed. In general, sampling of the subgrade soil is not as extensive as is needed for new pavement design. Detailed sampling of the base and surface may be needed if there is a large amount of variability in distress types along the project or if different materials are known to have been used. Specific recommendations concerning sampling frequency cannot be made without having the distress strip map of the existing project, the original soils data, and the original construction records for the pavement materials.

The detailed survey is conducted to furnish specific engineering properties of the different materials in the pavement over the length of the project. These engineering properties are used as inputs in the structural design of the selected rehabilitation strategy. These tests must also indicate the existing condition of the pavement and highlight any degradation that has taken place during the life of the pavement. The use of NDT equipment is a cost-effective means of determining the stiffness of the subgrade soil and its variability along the project.

9. OTHER CONSIDERATIONS

Volume Stability

Volume stability of the subgrade soil is essential for long-term performance. Swelling soils or soils susceptible to frost heave can have a significant impact on pavement performance. Collapsing soils are another, more localized problem.

Frost heaving and swelling soil potential can be easily identified in 4R work by the associated pavement distress. These distress types appear as crescent-shaped cracking in rigid pavements and as irregular swells and depressions in rigid and flexible pavements. If possible, corrective actions should be programmed to remedy or alleviate frost heaving and swelling soils problems. Localized areas of repeated frost heave or swelling soils can be removed and replaced. Large areas are more difficult to treat.

Several references provide some guidance for frost action and swelling soils corrective actions. (See references 11, 12, 13, and 14.) In practice, many swelling and frost heaving soils are not treated because of the high cost. For high-volume highways, several expensive and sometimes exotic methods have been used with mixed results. (See references 11, 12, 13, and 14.)

Stripping

Stripping in asphalt concrete is the physical separation of the asphalt cement film from the aggregate. The strength and integrity of an asphalt concrete material is the direct result of the bonding and waterproofing provided by the asphalt cement. When the cement is no longer bonded to the aggregate, there is little or no resistance to deformation under load, since the integrity of the material is lost. The potential for this to develop should be examined in the mix design phase and, for rehabilitation, should be investigated by visual observation of material taken from the pavement. If material is removed from the pavement by coring, the cores should be split immediately and examined internally for evidence of stripping, since the coring action can camouflage the evidence of stripping on the outer surface. A material exhibiting stripping is usually removed from the pavement. An excellent synthesis has been prepared on the development of moisture damage in asphalt concrete.[15]

There are several testing procedures that may be used to assess the stripping potential of HMA mixes (although these are primarily used in assessing the stripping potential for new mixes and not for rehabilitation). These are:

- NCHRP 246 or "Lottman" Test.[16,17] This test calls for testing Marshall specimens, both dry and moisture conditioned, at 12.8 °C and a loading rate of 1.65 mm per min. The data are functionally expressed as the ratio of wet to dry tensile strength and represented as the tensile strength ratio (TSR). The Lottman moisture condition procedure includes vacuum saturation and a freezing cycle of 15 hours.[17]
- NCHRP 274 or "Tunnicliff and Root" method.[18] This method is similar to the Lottman Procedure, with the following modifications:

  Load rate (50 mm per min)
  Test temperature (25 °C)
  Presaturation of 55 percent to 80 percent
  Preparation of mixture to void content of 7 ± 1 percent
  No-freeze cycle

- Immersion Compression (I/C) Test.[17] This is the oldest standard method by which quantitative stripping predictions on compacted bituminous mixtures have been made. This procedure tests 100- by 100-mm specimens prepared by double plunger method (ASTM D 1074). The I/C specimens are divided into two groups, forming dry and wet sets. The wet set is usually conditioned in a water bath at 49 °C for 4 days. The two sets are tested at 25 °C for compressive strength. The results are expressed as a retained compressive strength ratio representing the quotient between wet and dry strength. A threshold value of the ratio for nonstripping mixtures is 75 percent retained strength.[17]

- ASTM D 3625-83. This is primarily a field test and involves immersing 0.23 kg of bituminous aggregate in boiling water for 1 minute. Then, the percentage of total visible area of the aggregate that retains its original coating is visually estimated as above or below 95 percent. Some agencies have modified this test to include 10 minutes boiling instead of 1 minute.

Concrete Durability

D-Cracking

D-cracking in PCC pavements occurs when water freezes in the pores of susceptible aggregates, expands, and cracks the aggregate. The distress appears as hairline cracks typically following joints and cracks, where the presence of moisture is the highest. These cracks indicate that the cement paste is being broken down and the integrity of the concrete is being lost.

If D-cracking is suspected in an area, special sampling may be conducted. Cores taken from the joint or crack area will show whether the slab is intact over its full-depth, or if rubble is present. If rubble is found in a core at a joint, D-cracking can be assumed and the structure of the slab is suspect, then rehabilitation should be planned accordingly.

A petrographic evaluation may be done on the concrete cores to detect microcracking of the coarse aggregate and the cement matrix. The test is performed in accordance with ASTM C 856.

Alkali Reactivity

Certain types of aggregates contain materials that react with the alkalies (sodium and potassium) in portland cement. The two aggregate components that have been found to promote alkali-aggregate reactions are silica found predominantly in aggregates in the west and southwest and carbonate found predominantly in the Midwest and Canada. Alkali Silicate Reaction is called ASR, while Alkali-Carbonate Reaction is termed ACR. The reaction produces a gel that absorbs water and swells, thus fracturing the cement matrix and causing map cracking in the concrete. Depending on the amount of alkali in the cement and the degree of reactivity of the aggregate, the expansion may also cause significant compressive stress buildup in the concrete, resulting in closing, spalling, or shattering of transverse joints.

Unlike D-cracking, which starts at the bottom of the slab at joints and cracks, reactive aggregate problems develop throughout the slab. It shows up as fine cracks in a map pattern similar to alligator cracking in HMA pavements.

Tests for identifying potentially reactive aggregates are described in ASTM C 289, C 227, and C 342. Petrographic examination can also be useful in identifying potentially reactive aggregate. Neither laboratory testing nor petrographic analysis, however, can predict with certainty the rate of future distress development in a concrete pavement containing reactive aggregate. Recycling, which is discussed in detail in a later module, may be considered under some circumstances for a reactive aggregate concrete pavement that is deteriorating rapidly.

In addition to ASR and ACR, Delayed Ettringite Formation (DEF) has also been shown to cause early pavement distress.[22]

Freeze-Thaw Damage

In freeze-thaw climates, concrete pavements require the establishment of an adequate air system to allow for the internal expansion of the concrete. This is accomplished through the addition of an air-entraining agent in the concrete mix, which creates microscopic bubbles in the mix that allow for the internal expansion. Typically, 4 percent to 6 percent air is specified.

While not that common, more cases have been documented recently where an inadequate air void system on a void system filled with ettringite (DEF) has led to premature deterioration of concrete pavements.[22] When present, extensive spalling and scaling can occur at the joints and throughout the slab. A petrographic analysis on thin section analysis should be conducted to verify an inadequate air system if it is suspected.

Seasonal Variations

Seasonal variation in material properties must be recognized and accounted for in the pavement evaluation. This is particularly important for the subgrade soil. The support for the pavement structure provided by the subgrade soil varies from season to season. All materials show seasonal variation, and if any in-place material property determinations are conducted, they must be corrected to account for seasonal variability produced by moisture or temperature variation in the different materials.

10. SUMMARY

Many factors (soil type, density, water content, and freeze-thaw action) influence subgrade soil properties. All of these factors show considerable variation (e.g., depth in grade, length along project, soil type variants, and climatic changes) that makes the determination of exact subgrade soil properties difficult. It is particularly important for rehabilitation evaluation that the design values selected for a project reflect the expected in-place condition of the soil and be consistent with the design procedure being used. Since 4R projects deal with in-place material, the soil properties can be evaluated very quickly from published records. These records furnish very detailed information that can be used for several steps in the evaluation process.

To develop an adequate understanding of material structural adequacy, laboratory testing will be required in the future to develop a database of resilient modulus values that reflect expected field conditions. These tests should include changes due to moisture and density variation, as well as freeze-thaw cycles to which the subgrade soil is subjected.

The resilient modulus is the material parameter of choice for future pavement design and rehabilitation. Correlations between the resilient modulus and traditional material strength values or soil index properties may be used, since not all agencies have conducted extensive testing to develop historical data on the resilient modulus. The limitations in these procedures must be recognized, however, since subgrade soils are affected by minor changes in soil properties that may not be reflected in cross correlations.

Visual examination of cores and material samples provides valuable information about environmentally-induced deterioration in pavement layers, including concrete, asphalt, and other stabilized layers. No evaluation should be considered complete unless visual observations of materials from the project have been completed by an experienced engineer. A number of laboratory tests are also available to characterize in-place paving materials.

11. REFERENCES

1. "Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials, 1986.

2. Carpenter, S.H., M.I. Darter, and B.J. Dempsey, "Evaluation of Pavement systems for Moisture-Accelerated Distress," Transportation Research Record 705, Transportation Research Board, 1979.

3. Oglesby, C.H. and G.L. Hicks, "Highway Engineering," John Wiley & Sons, 1982.

4. "Instruction Manual, dynamic Cone Penetrometer,' United States Army Engineer Waterways Experimental Station, August 1989.

5. Harrison, J.A., "Correlation Between California Bearing Ratio and Dynamic Cone Penetrometer Strength Measurement of Soils," Proceedings, Institution of Civil Engineers, Part 2, Vol. 83, Technical Note No. 463, 1987.

6. Livneh, M., "The Use of Dynamic Cone Penetrometer in Determining the Strength of Existing Pavements and Subgrades," Proceedings, 9th Southeast Asian Geotechnical Conference, Bangkok, 1987.

7. Elliott, R.P., S.I. Thornton, K.Y. Foo, K.W. Siew, and R. Woodbridge, "Resilient Properties of Arkansas Subgrades," FHWA/AR-89/004, Arkansas State Highway and Transportation Department, November 1988.

8. Thompson, M.R. and Q.L. Robnett, "Final Report—Resilient Properties of Subgrade Soils," Civil Engineering Studies, Transportation Engineering Series No. 14, Illinois Cooperative Highway and Transportation Series No. 160, June 1976.

9. Vinson, Ted S., "Fundamental of Resilient Modulus Testing," presented at the workshop on Resilient Modulus Testing, Oregon State University, Corvallis, OR, March 28-30, 1989.

10. "Thickness Design for Concrete Highway and Street Pavements," EB-109.01B, Portland Cement Association, 1984.

11. "Roadway Design in Seasonal Frost Areas," NCHRP Synthesis of Highway Practice 26, Transportation Research Board, 1974.

12. Dempsey, B.J., "Climatic Effects on Airport Pavement Systems; State-of-the Art," Contract Report S-76-12, United States Army Waterways Experiment Station, June 1976.

13. Snethen, D.R., et al., "A Review of Engineering Experiences with Expansive Soils in Highway Subgrades," FHWA-RD-75-48, Federal Highway Administration, June 1975.

14. Snethen, D.R., "Technical Guidelines for Expansive Soils in Highway Subgrades," FHWA-RD-79-51, Federal Highway Administration, June 1979.

15. Lottman, R.P., "Predicting Moisture-Induced Damage to Asphaltic Concrete—Field Evaluation," NCHRP Report 246, Transportation Research Board, 1982.

16. Kiggundu, B.M. and F.L. Roberts, "The Success/Failure of Methods Used to Predict Stripping Propensity in the Performance of Bituminous Pavement Mixtures," preprint, Transportation Research Board, 68th Annual Meeting, 1989.

17. Tunnicliff, D.G., and R.E. Rott, "Use of Antistripping Additives in Asphaltic Concrete Mixtures—Laboratories Phase," NCHRP 274, Transportation Research Board, December 1984.

18. Hicks, R.G., "Moisture Damage in Asphalt Concrete," NCHRP Synthesis of Highway Practice 175, Transportation Research Board, October 1991.

19. Johnson, K.D., E.H. Rmeili, and M.I. Darter, "Development of Maintenance and Rehabilitation Strategies—Northern Illinois Toll Highway, East-West Tollway Extension," Phase II Final Report, Illinois State Toll Highway Authority, December 1990.

20. Darter, M.I., K.T. Hall, and Chen-Ming Kuo, "NCHRP Report 372 Support Under Portland Cement Concrete Pavements," Transportation Research Board, National Academy Press, Washington, DC, 1995.

21. Hall, K.T., M.I. Darter, T.E. Hoerner, and L. Khazanovich, 'LTPP Data Analysis Phase 1: Validation of Guidelines for K-Value Selection and Concrete Pavement Prediction," FHWA Report No. FHWA-RD096-198, January 1997, FHWA McLean, VA.

22. Gress, D., "Early Distress in Concrete Pavements," FHWA Report No.FHWA-SA-97-045, January 1997, FHWA Washington, DC.

MODULE 2-5

DRAINAGE SURVEY AND EVALUATION

1. INSTRUCTIONAL OBJECTIVES

This module presents the procedures necessary to examine the drainage characteristics of pavements in order to assess the need for subdrainage improvements as part of the pavement rehabilitation process. After completion of this module, the participant will be able to:

1. List four distress types that are indicative of moisture damage in a pavement and discuss the importance of their severity levels.
2. Define external and internal drainage factors and list their major components.
3. Describe the principle behind drainage time for a flooded base course and discuss how this time influences pavement performance.
4. List the properties that influence the drainability of a subgrade.
5. Combine base and subgrade drainage criteria to develop the American Association of State Highway and Transportation Officials (AASHTO) drainage coefficient.
6. Make recommendations regarding the drainage capabilities of specific materials.

2. INTRODUCTION

It has long been recognized that a major cause of pavement distress is water in the pavement system. The recognition of the amount, severity, and cause of moisture damage plays an important role in the selection of the 4R scheme used on the pavement. Unless moisture-related problems are recognized and corrected where possible, the effectiveness of any 4R work will be reduced. If the subgrade has moisture problems, it will be of little benefit to overlay the pavement, recycle it, or rework and stabilize the base, without also addressing any problems inherent in the subgrade. Likewise, if the base or subbase has moisture problems, it will be wasteful to rehabilitate, restore, or overlay the pavement without addressing the moisture problems.

3. DEFINITIONS

Moisture. In this context, moisture refers to the liquid form of water which can have a deleterious affect on the properties and overall performance of the pavement. The sources of moisture can be through:

- Infiltration from above.
- Lateral seepage through natural soils.
- Capillary action from underlying water tables.

Drainage. This refers to any process that provides for the removal of moisture from the pavement or helps to prevent its entry into the pavement structure.

4. MOISTURE-RELATED DISTRESS SURVEY

Nondestructive deflection testing (NDT) provides information about a pavement's overall structural capacity, and it will pinpoint areas of weakness. NDT alone cannot, however, completely identify which pavement component is responsible for the weakness, or whether moisture-related problems are responsible for the deterioration. A drainage evaluation must be performed in conjunction with the NDT analysis to identify existing moisture-related problems and to identify the potential for moisture problems to develop within a pavement structure. A drainage evaluation includes a distress survey and an examination of the external and internal drainage factors that influence the moisture condition in a pavement.

Visual Evaluation

The first step in performing a drainage evaluation involves a visual inspection of the pavement. Each distress type that develops on a particular pavement will be due to loads, materials, environmental factors, or a combination of the three. Moisture will accelerate any deterioration, regardless of its source. Table 2-5.1 and table 2-5.2 contain a breakdown of moisture-related distress types for asphalt concrete (AC) and portland cement concrete (PCC) pavements, respectively.[1]

During a visual inspection of a project, the following drainage-related items should be noted, and any unsatisfactory conditions should be identified on a strip map:

- Are the ditchlines clear of standing water?
- Are the ditchlines and pavement edge free from weed growth?
- After a rain, is there moisture standing in the joints or cracks? Is there evidence of pumping? Is there water standing at the outer edge of the shoulder, or is there evidence that the water may pond on the shoulder?
- If subdrainage is present, are the outlets locatable, clear of debris, and set at the proper elevation above the ditchline?
- Are inlets clear and set at proper elevations, with adequate cross slope on the pavement surface to get water to the pavement edge?
- Are joint sealants or crack sealants in good condition, and do they prevent water from entering the pavement?

If drains are present, their effectiveness should be evaluated. This can be done with a water truck releasing water over pavement discontinuities, or by observing the pavement after a rain. The flow from each outlet should be examined and any outlets that are flowing at a much lower rate than the others should be noted on a strip map. Where drains are not functioning, they can be dug up and checked, or a video camera can be used to inspect the drains for damage.[15]

5. EXTERNAL AND INTERNAL DRAINAGE FACTORS

By examining the results of the distress survey, moisture-related problems can be identified. Moisture-related distresses develop due to external and internal drainage factors. External drainage factors are the climatic conditions in an area that regulate the supply of moisture to the pavement. Internal drainage factors are those pavement material properties whose interaction with moisture influences pavement performance.

External Factors

The external factors that influence the amount of moisture in a pavement are primarily climatic. A large number of climatic variables have been studied and catalogued for nearly every region in the United States. It should be noted that identifying climatic variables alone does not provide any detail concerning moisture in the pavement and the damage the moisture may cause.

Table 2-5.1. Moisture-related distresses in AC pavements.[1]

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associated	Structural Defect Begins in		
						Asphalt	Base	Subgrade
Surface Defects	Abrasion	No	No	Aggregate	No	Yes	No	No
	Bleeding	No	Accentuated by High Temp	Bitumen	No	Yes	No	No
	Raveling	No	No	Aggregate	Slightly	Yes	No	No
	Weathering	No	Humidity & Light-Dried Bitumen	Bitumen	No	Yes	No	No
Surface Deformation	Bump or Distortion	Excess Moisture	Frost Heave	Strength-Moisture	Yes	No	Yes	Yes
	Corrugation or Rippling	Slight	Climatic & Suction Relations	Unstable Mix	Yes	Yes	Yes	Yes
	Shoving	No		Unstable Mix Loss of Bond	Yes	Yes	No	No
	Rutting	Excess in any layer	Suction & Materials	Compaction Properties	Yes	Yes	Yes	Yes
	Waves	Excess	Suction & Materials	Exp. Clay Frost. Susc.	No	Not Initially	No	Yes
	Depression	Excess	Suction & Materials	Settlement, Fill Material	Yes	No	No	Yes
	Potholes	Excess	Frost Heave	Strength-Moisture	Yes	No	Yes	Yes
Cracking	Longitudinal	Yes	Spring-Thaw Strength Loss		Yes	Faulty Construction	Yes	Yes
	Alligator	Yes Drainage		Possible Mix Problems	Yes	Yes, Mix	Yes	Yes
	Transverse	Yes	Low-Temp, F-T Cycles	Thermal Properties	No	Yes, Temp Susceptible	Yes	Yes
	Shrinkage	Yes	Suction, Moisture Loss	Moisture Sensitive	No	Yes, Hardening	Yes	Yes
	Slippage	Yes	No	Loss of Bond	Yes	Yes, Bond	No	No

Table 2-5.2. Moisture-related distresses in PCC pavements.[1]

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associated	Structural Defect Begins in		
						Surface	Base	Subgrade
Surface Defects	Spalling	Possible	No		No	Yes	No	No
	Scaling	Yes	F-T Cycling	Chloride Influence	No	Yes-Finishing	No	No
	D-Cracking	Yes	F-T Cycling	Aggregate	No	Yes	No	No
	Reactive Agg.	Yes	No	Agg-Cement	No	Yes	No	No
	Crazing	No	No	Rich Mortar	No	Yes-Weak Surface	No	No
Surface Deformation	Blow-Up	No	Temperature	Thermal Properties	No	Yes	No	No
	Pumping	Yes	Moisture	Fines in Base Moisture Sensitive	Yes	No	Yes	Yes
	Faulting	Yes	Moisture Suction	Settlement Deformation	Yes	No	Yes	Yes
	Curling	Possible	Moisture and Temperature		No	Yes	No	No
Cracking	Corner	Yes	Yes	Follows Pumping	Yes	No	Yes	Yes
	Diagonal Transverse Longitudinal	Yes	Possible	Cracking Follows Moisture Buildup	Yes	No	Yes	Yes
	Punch Out	Yes	Yes	Deformation Following Cracking	Yes	No	Yes	Yes
	Joint	Produces Damage Later	Possible	Proper Filler and Clean Joints	No	Joint	No	No

Regional Climatic Indicators

The climatic regions of the United States were established under an FHWA project on pavement subdrainage.[1] These regions represent areas subjected to similar climatic influences in terms of precipitation and temperature. Reference 1 contains a detailed explanation of the selection process for these climatic zones, which are shown in figure 2-5.1. Simplified zones for climatic characterization are used in the AASHTO Design Guide, as defined in figure 2-5.2.[2]

The moisture regions on these maps were developed using the Thornthwaite Moisture Index.[3] This index is a numerical rating of the amount of moisture available to the subgrade soil during a year and combines rainfall and evapotranspiration data to give a broad indication of moisture availability. A positive number represents an excess of moisture, whereas a negative number represents a moisture deficiency.

Local Climatic Information

Local climatic information must be evaluated to determine whether an area is accurately represented by the average values presented in figure 2-5.1 and figure 2-5.2. The United States Weather Bureau is the main source of local climatic data. The United States Environmental Data Service publishes "Local Climatological Data" for many cities across the country. This information can be used to identify typical rainfall intensities, total monthly precipitation, and average daily and monthly temperatures.

Precipitation

Precipitation data can be used to estimate the frequency that any granular layers will be at or approaching saturation levels. The local depth of the water table and the amount of time that a pavement is exposed to precipitation both influence a granular material's saturation level. Following a rainfall, the capillary rise (the distance that a layer can attract moisture) in granular subgrade layers is often enough to saturate the soil at distances up to or exceeding 0.9 m above a water table. Fine-grained soils may be close to saturation at heights exceeding 6 m above the water table.[4]

Temperature

When saturated pavement layers are subjected to freezing temperatures, additional problems can develop. Frost heave can occur as moisture in the layers freezes, and the accumulation of excess moisture during freezing periods can lead to softening of unbound layers and loss of support when thawing occurs. The concept of a Freezing Index (FI) is useful in evaluating the potential for frost-related problems. The FI is the annual summation of the number of degrees the average daily temperature (T, in ° F) is below freezing for each day throughout a year.

The summation begins whenever the average daily temperature is below freezing and ends in the spring when the average temperature rises above freezing. Only those days below freezing are counted. One degree day represents one day with a mean air temperature one degree below freezing. For each degree below freezing, an additional degree day is added. In general, for areas with FIs exceeding approximately 100 degree-days, cold temperatures can be expected to contribute to moisture-related problems.[5] Figure 2-5.3 provides an illustration of the typical Freezing Index values throughout the United States.

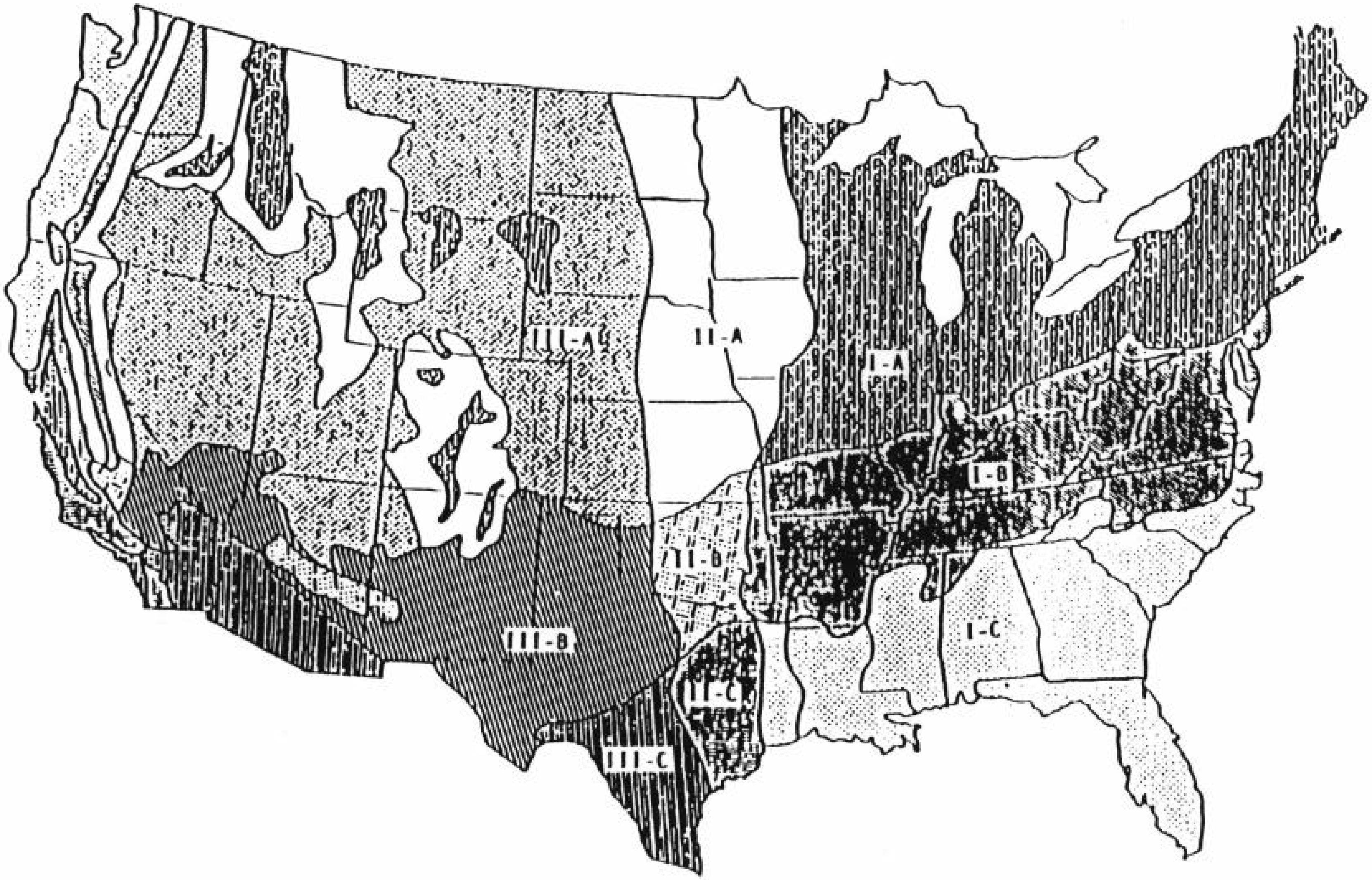
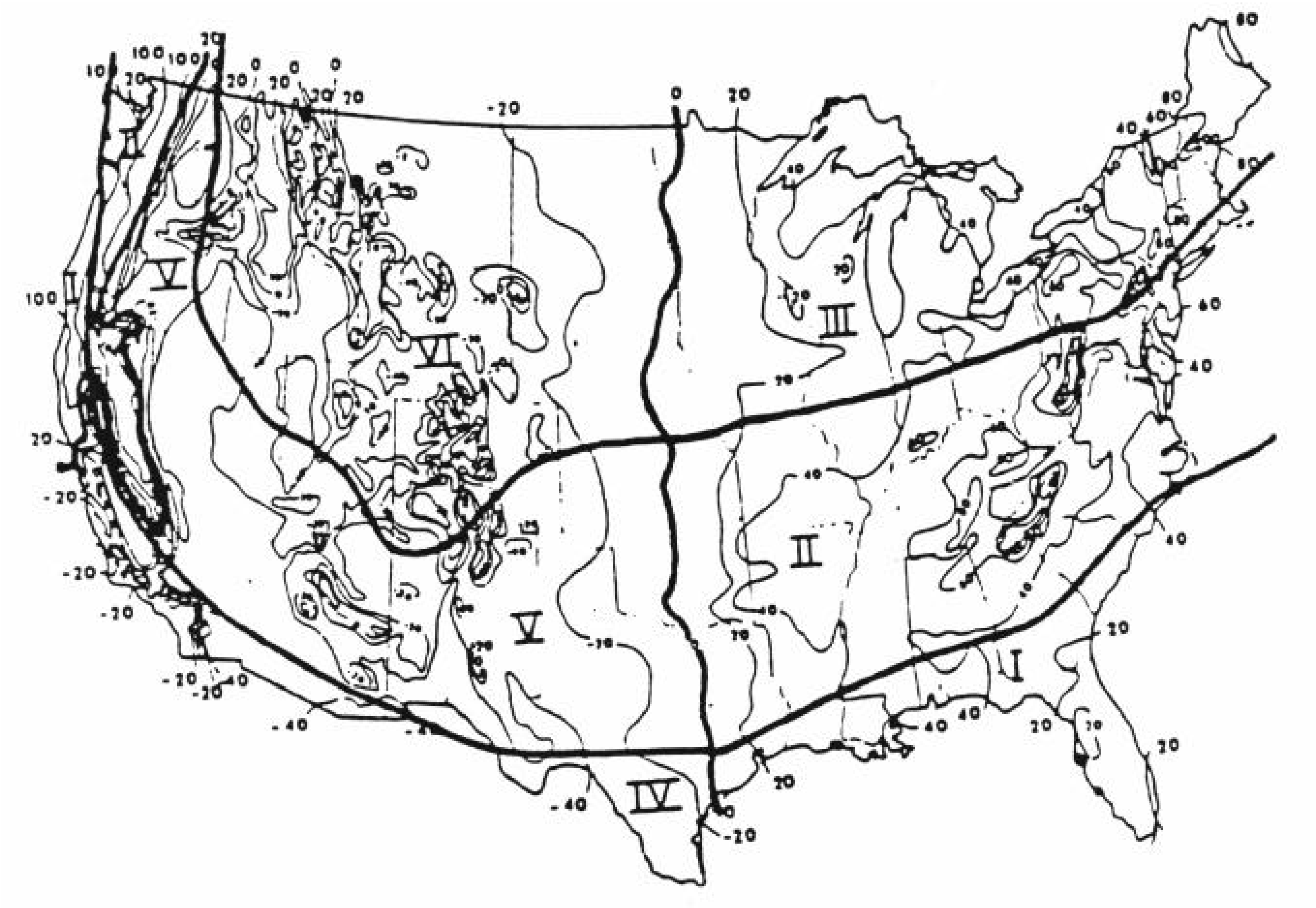


Figure 2-5.1. Nine climatic zones based on moisture and temperature influence on performance.[(1)]



REGION	CHARACTERISTICS
I	Wet, No Freeze
II	Wet, Freeze-Thaw Cycling
III	Wet, Hard-Freeze, Spring Thaw
IV	Dry, No Freeze
V	Dry, Freeze-Thaw Cycling
VI	Dry, Hard-Freeze, Spring Thaw

Figure 2-5.2. Six climatic zones as identified in the AASHTO Design Guide.[2]

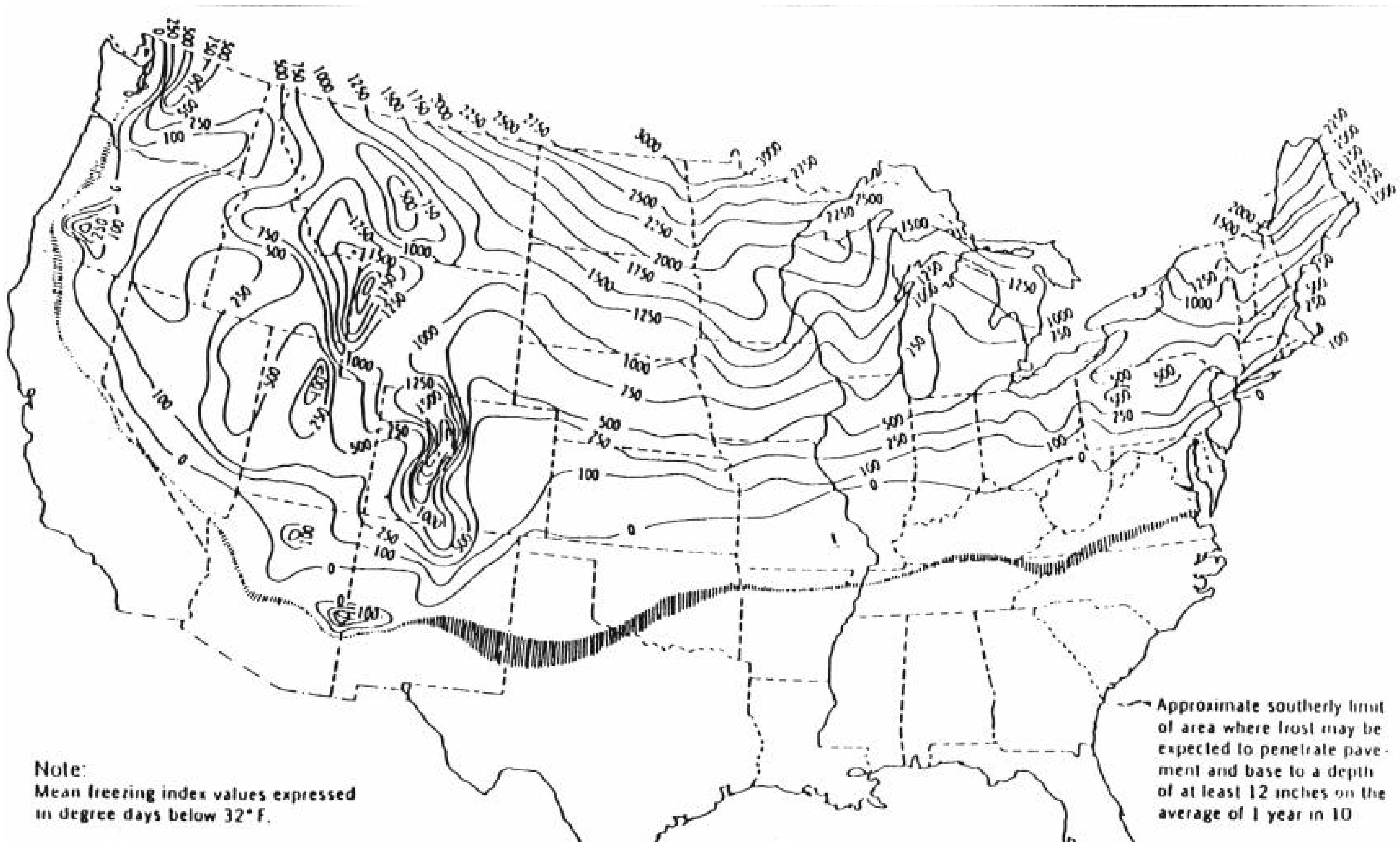


Figure 2-5.3. Mean freezing index values.[1]

Internal Drainage Factors

Internal drainage factors are the roadway geometry and material properties that affect pavement drainage and performance. Even if a pavement is located in a wet climate, it may not experience moisture damage if it is constructed of good-quality, moisture-resistant materials. Important moisture-related material factors include:

- Drainability.
- Permeability.
- Physical geometry of roadway.
- Soil type.
- Topography.
- Water table.
- Existing drainage facilities.

These factors should be considered for each material in the pavement structure. The materials most influenced by moisture are the natural subgrade soil and the untreated granular layers.

Granular Base Course Layers

The level of saturation present in a granular base course layer has a significant impact on load-related performance. Figure 2-5.4 shows the effect of saturation on the deflection for different paving materials at the AASHO Road Test. At a point above 85 percent saturation, the total deflection for the materials increases rapidly, which can lead to the accelerated development of pavement fatigue damage. Because the rebound deflection shows a uniform change with increasing saturation level, the permanent deformation will increase tremendously at higher saturation levels.[6] Thus, it is important that a pavement system be able to quickly drain water to decrease the amount of time a structure is near saturation. Figure 2-5.5 shows how the quality of drainage is affected by the time required to drain the pavement and the percent time that the pavement structure is at or near saturated conditions.[7] The lower the quality of drainage, the greater its impact on pavement performance.

The granular components of the roadbed system directly influence how long water remains in the pavement. These layers have traditionally been designed for maximum strength without consideration of their drainage characteristics. However, in addition to providing strength, a good base must be able to rapidly drain water away from the pavement system. A granular layer that has slow drainage times and that is often near saturation can reduce the performance of the pavement. The drainability of a granular layer is a function of the material's permeability, the physical geometry of the roadway, and the material composition.

Permeability

Permeability is the measure of how fast water will flow through a material. Permeability should not be confused with porosity, which is a measure of the volume of the void space within a material and an important factor influencing permeability. A nomograph for estimating the permeability of a material is given in figure 2-5.6; the figure also contains the equation on which the nomograph is based.[8] From this figure, it is observed that the permeability of a material is a function of the percentage of fines (P_{200}), the D_{10} effective grain size, the specific gravity of the material solids, G , and the dry density of the granular material, γ_d.

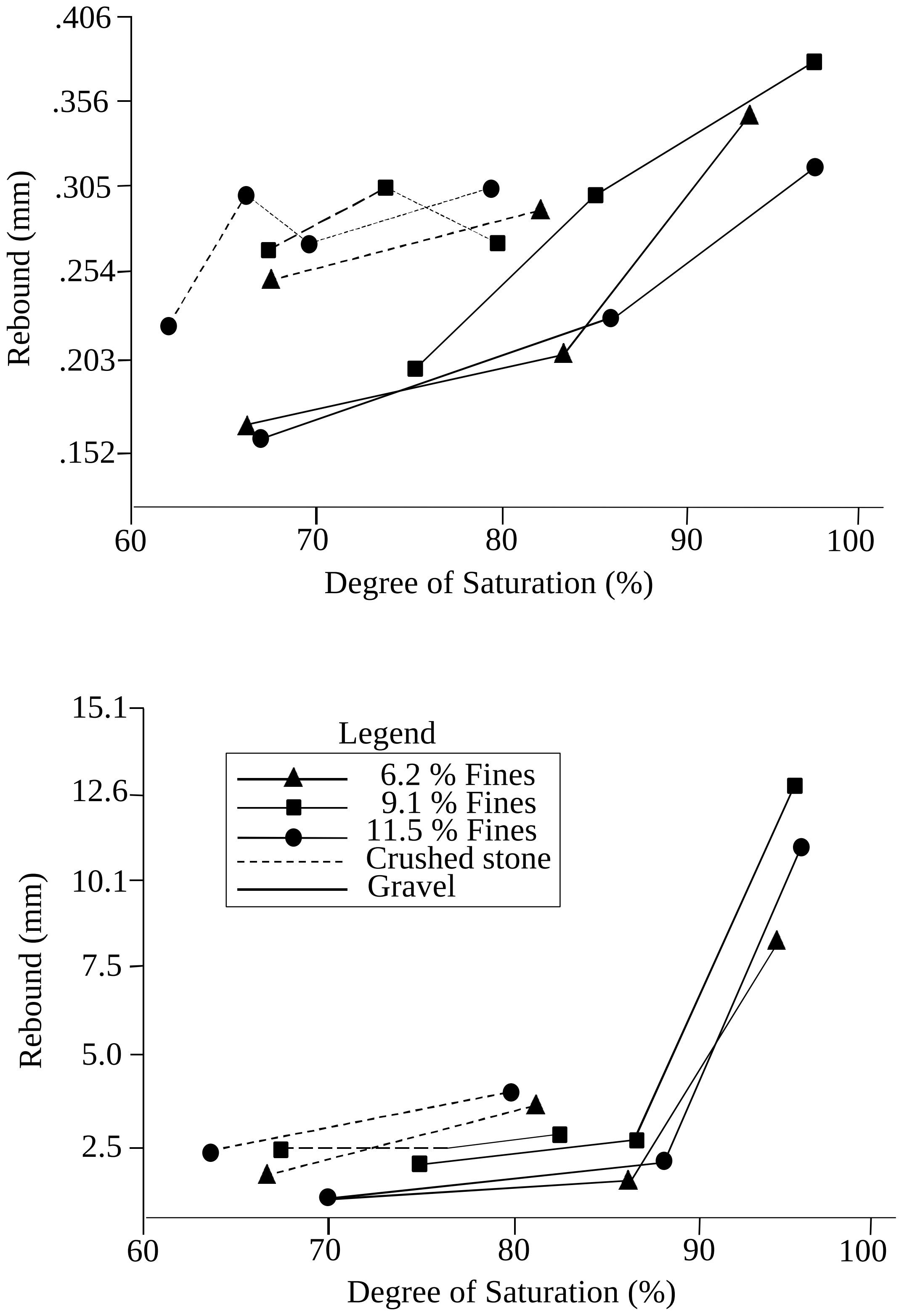


Figure 2-5.4. Effect of degree of saturation on the repeated-load deformation properties of the AASHO granular materials.[6]

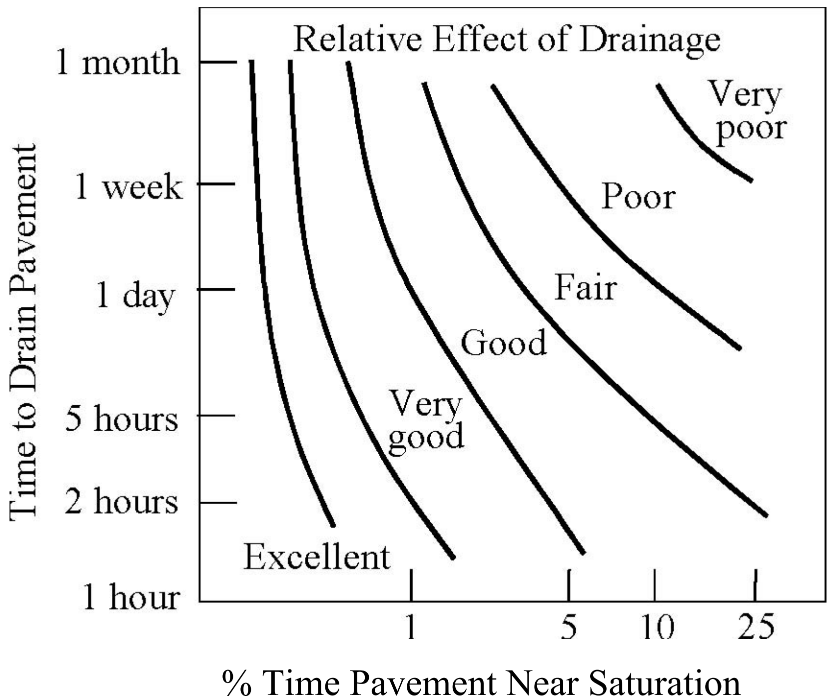


Figure 2-5.5. Quality of drainage as affected by drainage and saturation time.[7]

P_{200} and D_{10} can be obtained from a mechanical sieve analysis of the granular material; P_{200} is the percent of the material passing the No. 200 sieve and the D_{10} effective grain size is the diameter of the particle corresponding to 10 percent of the material passing. An example mechanical sieve analysis illustrating these terms is shown in figure 2-5.7.[9]

The specific gravity of the material solids, G_s, represents the density of the solids portion of the granular material. It is expressed as the quotient of the density of the solids and the density of water, i.e., γ_s / γ_w, and generally ranges from 2.60 to 2.80.[10] The density of the solids, γ_s, is defined as the weight of the solids, W_s, divided by the volume of the solids, V_s. Figure 2-5.8 provides definitions for these weight-volume parameters.

The dry density of the material, γ_d, represents the density of the material if it is oven-dry. Referring to figure 2-5.8, dry density may be calculated from equation 2-5.1:

$$\gamma_d = W_s / V_T \tag{2-5.1}$$

where:

γ_d = dry density, lb/ft^3

W_s = weight of the solid components of the material, lb

V_T = total volume of the material, ft^3

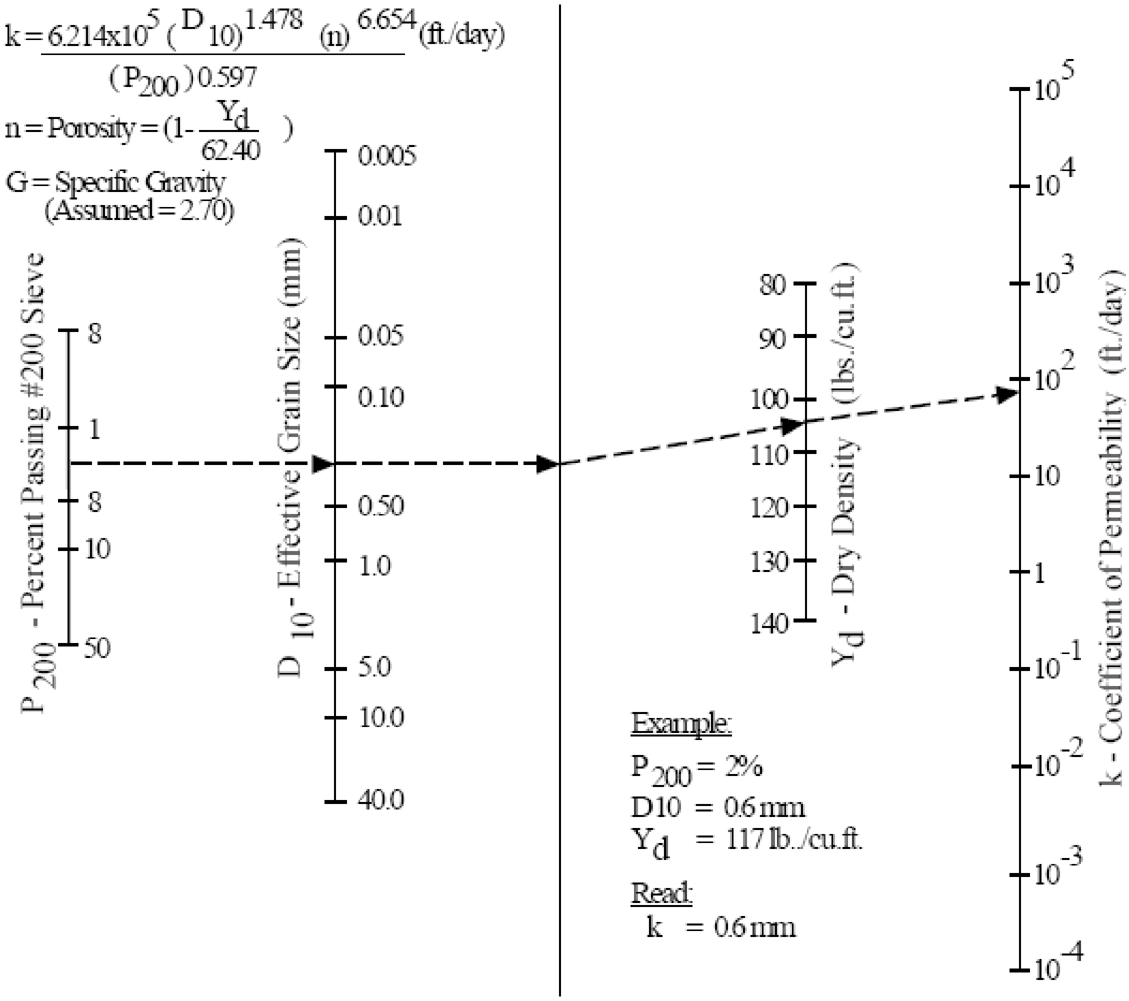


Figure 2-5.6. Chart for estimating the coefficient of permeability of granular materials.[8]

If the water content of the material is known, it may be more convenient to determine the maximum dry density from the following equation:

$$\gamma_d = 100\, r \,/\, (100 + w) \qquad (2\text{-}5.2)$$

where:

γ = density of the material at w water content, lb/ft^3

w = water content of the material, percent

= W_w/W_s

As an example in using the permeability chart, assume that a sandy-gravel subbase has a γ_d of 117 lb/ft^3 (1.87gm/cm^3), a D_{10} of 0.6 mm, a P_{200} of 2 percent, and a G_s of 2.70. Using figure 2-5.6, the permeability is estimated to be 65 ft/day (20 m/day). Note that in the early stages of a drainage evaluation, the permeability estimates obtained from figure 2-5.6 are acceptable, although it may be desirable to conduct sampling for permeability testing later.

Fines (material passing the No. 200 sieve) have a strong influence on permeability. As the fines in a granular material increase, the nomograph in figure 2-5.6 shows that permeability decreases rapidly. If the increase in fines content is caused by contamination of a granular layer with subgrade soil, the permeability decrease can be even greater, depending on the subgrade soil type. Figure 2-5.9 shows the relative effect on permeability due to contamination by fines of different types.[11] It should be noted that the effect of fines containing clay or silt is much greater than that of inert fines such as limestone or silica dust. Granular layers with estimated permeabilities that are being used as part of a drainage system should be sampled to verify that contamination with fines has not occurred, and that the assumed permeabilities are representative.

It is also worth noting that the effective grain size, D_{10}, has a large impact on the permeability of the material. Since it is an indicator of the grain size distribution, an increase in the D_{10} parameter has the effect of increasing the permeability.

Soil Grain Properties

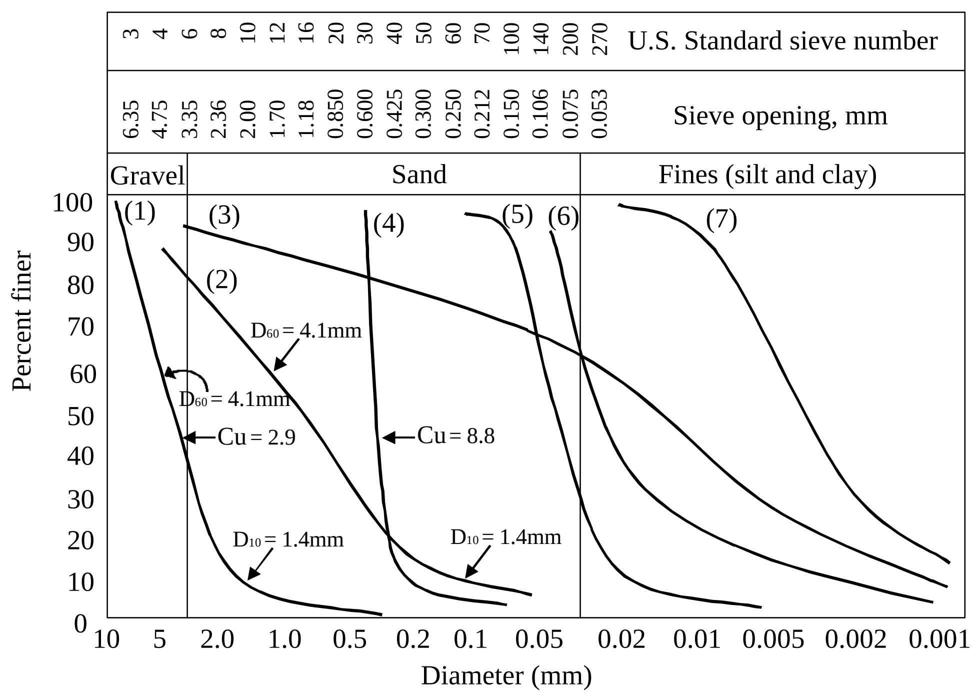


Figure 2-5.7. Example mechanical sieve analysis illustrating P_{200} and D_{10}.[9]

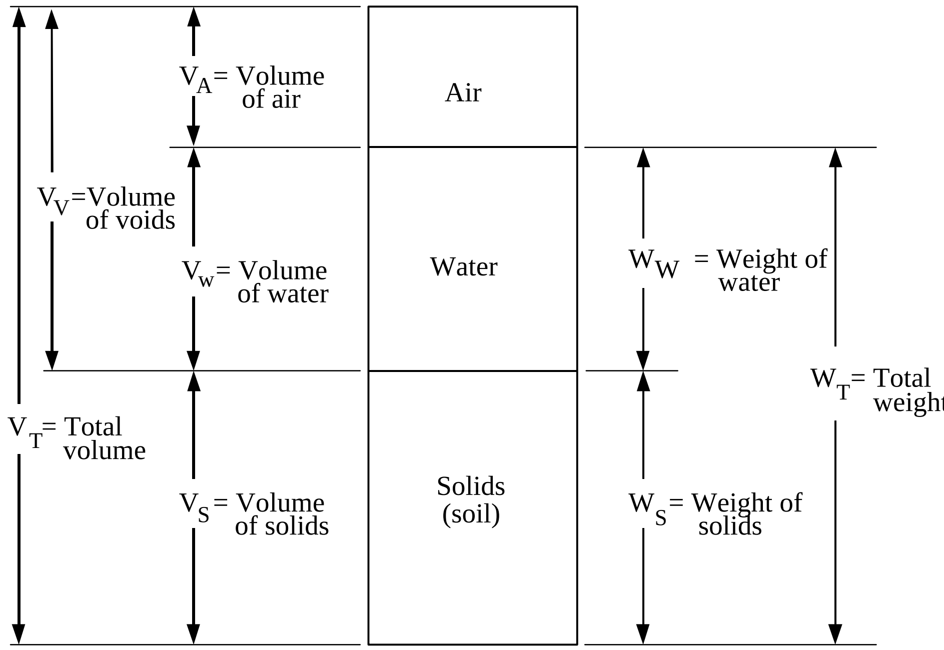


Figure 2-5.8. Definition of the weight-volume parameters.

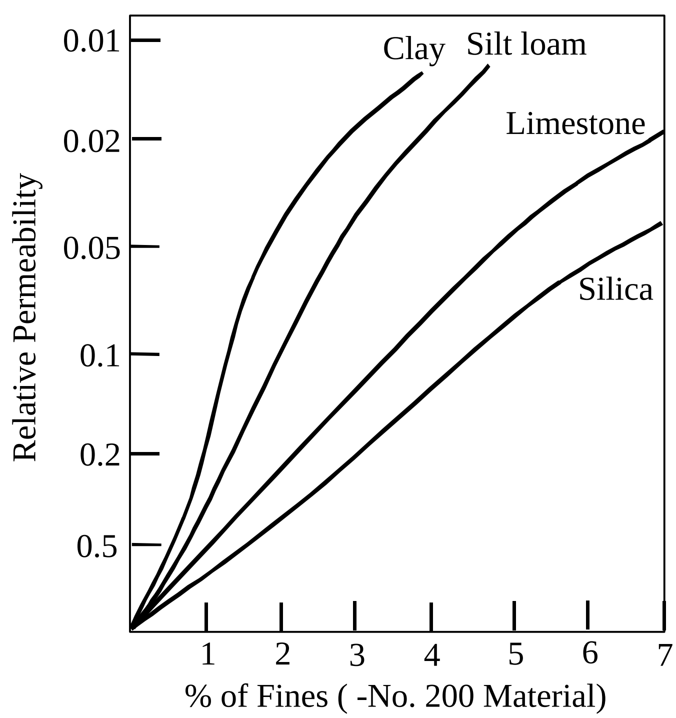


Figure 2-5.9. Effect of amount and types of fines on permeability.[11]

Effective Porosity

Effective porosity is a direct indication of how strongly a soil will hold water. It is measured by allowing a saturated soil sample to drain under the influence of gravity. The effective porosity is the ratio of the volume of the water that drains under gravity to the total volume of the sample.

The effective porosity of different soils provides a direct comparison of how long the soil will hold moisture. Clays have a much lower effective porosity than sands or gravels. The dry density, percent fines, and specific gravity of the solids are the only material properties needed to determine the effective porosity of a material.

For example, consider the sandy-gravel subbase discussed in the permeability section. Recall that the maximum dry density was 117 lb/ft^3 (1.87 gm/cm^3), the percentage of fines (P_{200}) was 2 percent, and the specific gravity of the solids was 2.70. If the total volume (V_T) of the material is assumed to be 1.00 cm^3 , then the weight of the solids (W_s) can be calculated using the weight-volume relationships in figure 2-5.8:

$$W_s = \gamma_d * V_T \quad (2\text{-}5.3)$$
$$= 1.87 * 1.00 = 1.87 \text{ gm}$$

Then, again using the relationships in figure 2-5.8, the weight of the solids can be converted to the volume of the solids (V_s):

$$V_s = W_s / G_s \quad (2\text{-}5.4)$$
$$= \quad 1.87 / 2.70 = 0.694 \text{ cm}^3$$

The volume of the voids (V_v) can then be determined as the difference between the total volume and the volume of the solids:

$$V_v = V_T - V_s \quad (2\text{-}5.5)$$
$$= \quad 1 - 0.694 = 0.306 \text{ cm}^3$$

This volume represents the absolute maximum amount of water that would drain from the sample if all the voids were full of water. The total porosity, N_{max}, can be calculated by:

$$N_{max} = V_v / V \quad (2\text{-}5.6)$$

$$N_{max} = 0.306 / 1.0 = 0.306$$

However, in reality, only a portion of this water can be drained. The percentage of the total water that actually drains under gravity is a function of the grain size distribution, the amount of fines, and, most importantly, the type of minerals in the fines. Therefore, it is more useful to express the porosity in terms of an effective porosity that represents the amount of water that can drain under the force of gravity. The effective porosity (sometimes called the specific yield, effective yield, or yield capacity) is defined as:

$$N_e = N_{max} * C \quad (2\text{-}5.7)$$

where:
N_e = Effective porosity
N_{max} = Total porosity
C = Adjustment factor to account for estimated water loss

Table 2-5.3 provides estimates of the C factor, which represents the amount of water that can be drained under gravity for different saturated materials.

Table 2-5.3. Estimate of the amount of water that can be drained from saturated granular materials under gravity.

	AMOUNT OF FINES								
	< 2.5 PERCENT			5 PERCENT			10 PERCENT		
Predominant Material	Type of Fines			Type of Fines			Type of Fines		
	Filler	**Silt**	**Clay**	**Filler**	**Silt**	**Clay**	**Filler**	**Silt**	**Clay**
Gravel	70	60	40	60	40	20	40	30	10
Sand	57	50	35	50	35	15	25	18	8

Notes: 1. *Gravel, 0 percent fines, 75 percent greater than No. 4: 80 percent water loss.*
2. *Sand, 0 percent fines, well graded: 65 percent water loss.*
3. *Gap-graded material will follow the predominant size.*

The material used in this example contains 2.0 percent fines (assumed inert) and is a combination of sand and gravel. From table 2-5.3, the percentage of total water lost is between 57 and 70 percent. For purposes of this example, a value of 60 percent will be assumed. Using this value, the calculated effective porosity is:

$$N_e = 0.306 * 0.6 = 0.184 \qquad (2\text{-}5.8)$$

Drainage Time

The effective porosity and permeability values are used to calculate drainage times for granular layers. Once drainage times are determined, they can be used as indicators of the drainage quality of the granular layers. Since materials with longer drainage times will be in or near a saturated condition for a longer period of time, they will be susceptible to more permanent deformation during that time.

Figure 2-5.10 can be used to estimate drainage times for granular materials. However, there are many hand calculations that must be performed before and after using the chart in order to determine the drainage times. To facilitate the drainage time calculation, a blank form such as that shown in table 2-5.4 is very useful.

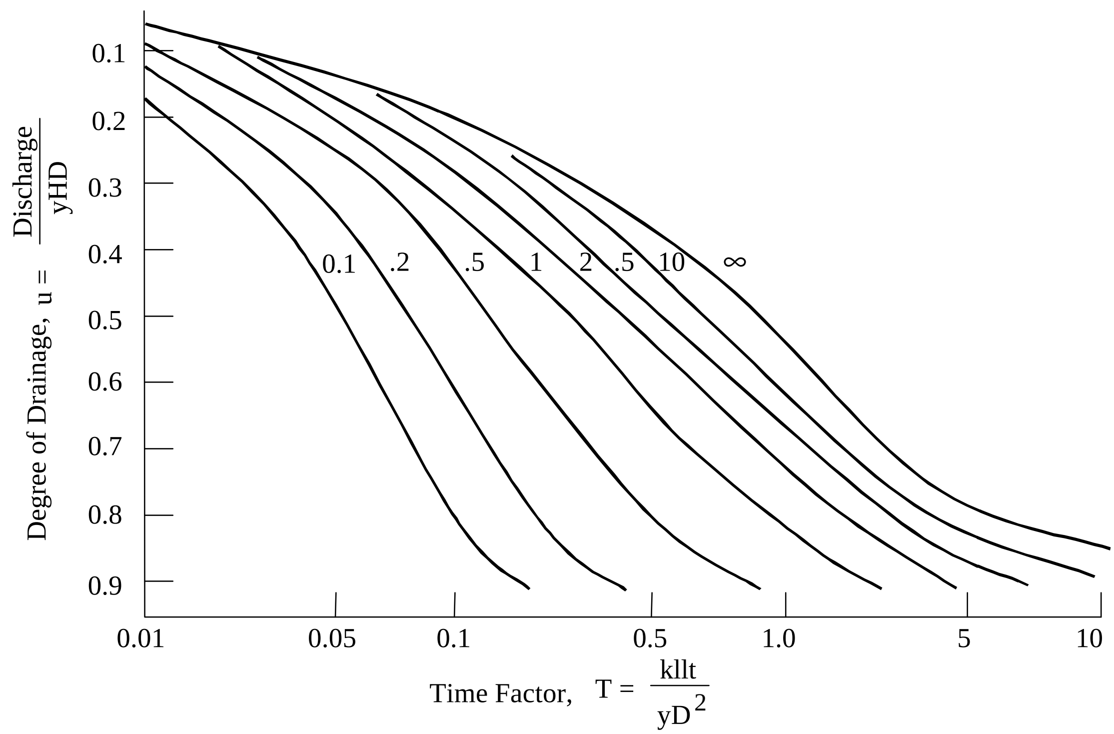


Figure 2-5.10.

Graphical procedure for calculating the rate of drainage of a flooded base course.[8]

Because of the complexity in the drainage time calculations, an example in using figure 2-5.10 and table 2-5.4 is provided in table 2-5.5. For clarity, the many variables involved in the drainage time calculations are defined below:

k	=	coefficient of permeability, ft/day
H	=	thickness of granular layer, ft
t	=	drainage time, days
N_e	=	effective porosity (specific yield)
D	=	width of granular layer, ft
g_l	=	longitudinal grade, ft/ft
g_t	=	transverse grade, ft/ft
g_e	=	effective grade = $(g_l^2 + g_t^2)^{0.5}$
L_e	=	effective drainage length, ft
	=	$D * [(g_l / g_t)^2 + 1]^{0.5}$
S	=	slope factor = $H / [(L_e)(g_e)]$
U	=	percent of drainage completed
X	=	unit conversion factor
	=	$(N_e * D^2) / (H * k)$

Table 2-5.4. Form to calculate drainage times and saturation levels for granular materials.[1]

Pavement Section ____________________

Percent Fines ____________________

Type of Fines ____________________

D_{10}, mm ____________________

γ_d, lb/ft^3 ____________________

G_s ____________________

k, ft/day ____________________

H, ft ____________________

D, ft____________________

g_t, ft/ft ____________________

g_l, ft/ft ____________________

$L_e = D*[(g_l/g_t)^2 + 1]^{0.5}$ = ____________________

$g_e = (g_l^2 + g_t^2)^{0.5}$ = ____________________

$S = H/(L_e * g_e)$ = ____________________

$X = (N_e * D_2)/(H*k)$ =____________________

V_t= 1.0 cm^3 ____________________

W_s, gm = γ_d/62.5 =____________________

V_s, cm^3 = W_s/G_s = ____________________

V_v, cm^3 = 1.0 - V_s = ____________________ =Ne_{max}

C, percent = ____________________

$N_e = V_v$ x (C/100) = ____________________

(1) **U**	*(2)* **T**	*(3)* **t, days** *(2)* * X	*(4)* **t, hours** *(3)* * 24	*(5)* **N_e * U**	*(6)* **V_w** V_v-*(5)*	*(7)* **Sat** *(6)*/V_v	*(8)* **% Sat** *(7)* * 100
0.1							
0.2							
0.3							
0.4							
0.5							
0.6							
0.7							
0.8							
0.9							

For the example given in table 2-5.5, the drainage times for a granular subbase material beneath an in-service concrete pavement are desired. The following information is available:

Material Type:	Sandy-gravel subbase	Type of Fines:	Inert
Percent Fines, P_{200} :	2 percent	Effective Grain Size, D_{10}	0.6 mm
Dry Density, γ_d :	117 lb/ft^3 (1.87 gm/cm^3)	Specific Gravity, G_s :	2.70
Layer Thickness, H:	0.75 ft (0.23 m)	Layer Width, D:	13 ft (4.0 m)
Transverse Slope, g_t :	0.02 ft/ft (2 percent)	Longitudinal Slope, g_l:	0.01 ft/ft (1 percent)

Before using the drainage time chart (figure 2-5.10), several preliminary calculations are needed. First, the permeability must be calculated. Using the percent fines, the effective grain size, and the dry density in figure 2-5.6, the permeability is estimated to be 65 ft/day (20 m/day).

Another preliminary calculation is the determination of the effective porosity. The procedure for determining the effective porosity was illustrated previously but is briefly repeated here. In the procedure, the weight of the solids, W_s , is determined by dividing the dry density by the density of water, which yielded a value of 1.87 gm. The volume of the solids, V_s , is calculated by dividing the weight of the solidsby the specific gravity. Then, by assuming a total volume, V T , of 1.0, the volume of the voids isdetermined to be 0.306 by subtracting the volume of the solids from the total volume. Finally, an estimate of the amount of water that can drain for this material type is needed. Consulting table 2-5.3 for a sandy-gravel material consisting of 2 percent inert fines, a value of 60 percent is obtained. The effective porosity is determined by multiplying the volume of the voids (0.306) by the estimated amount of water that can drain for this material type (0.60) to obtain a value of 0.184. These calculations are all shown in table 2-5.5.

The calculation of the "*X*" value provides a single term to multiply by in order to determine the drainage time. Table 2-5.5 shows the calculation of the *X* value, which is a function of the effective porosity, the layer width, the layer thickness, and the material permeability.

The final calculation before the drainage time chart may be used is the determination of several geometric factors. These include the effective grade, the effective drainage length, and the slope factor. These values are calculated as shown at the top of table 2-5.5.

Figure 2-5.10 may now be entered to determine drainage times for different levels of drainage. The table at the bottom of table 2-5.5 can be used to determine drainage times for different degrees of drainage, U (shown in column 1). For example, for U = 0.1 (meaning that 10 percent of the maximum drainage that the layer can discharge has occurred), figure 2-5.10 yields a time factor, T, of 0.025 for a slope factor of 2.4. This is entered in column 2 of the table at the bottom of table 2-5.5. Column 3 is the product of column 2 and the *X* value, providing the drainage time in days. In this case, it has taken 0.0159 days to drain 10 percent of the water from the saturated layer. Column 4 more conveniently expresses the time in column 3 in terms of days. Thus, it took about 0.382 hours (about 23 minutes) to drain 10 percent of the water from the saturated layer. This procedure can be repeated for different levels of drainage to determine the time to reach certain levels of drainage.

Table 2-5.5. Example of drainage time and saturation level calculations.[1]

Pavement Section	Sandy-Gravel Subbase		
Percent Fines	2.0	Type of Fines	Inert
D_{10}, mm	0.6	γ_d, lb/ft^3	117
G_s	2.70	k, ft/day	65
H, ft	0.75	D, ft	13
g_t, ft/ft	0.02	g_l, ft/ft	0.01
$L_e = D^*[(g_l/g_t)^2 + 1]^{0.5} =$	14.5	$g_e = (g_l^2 + g_t^2)^{0.5} =$	0.022
$S = H/(L_e * g_e) =$	2.4	$X = (N_e * D^2)/(H^*k) =$	$0.184*13^2/0.75*65 = 0.635$
$V_t = 1.0$ cm^3			
W_s, gm = $\gamma_d/62.5$ =	1.87		
V_s, cm^3 = W_s/G_s =	0.694		
V_v, cm^3 = 1.0 - V_s =	0.306 = Ne_{max}		
C, percent =	60		
$N_e = V_v$ x (C/100) =	0.306 * 0.6 = 0.184		

(1) **U**	*(2)* **T**	*(3)* **t, days** *(2)* * X	*(4)* **t, hours** *(3)* * 24	*(5)* **N_e * U**	*(6)* **V_w** V_v -*(5)*	*(7)* **Sat** *(6)*/V_v	*(8)* **% Sat** *(7)* * 100
0.1	0.025	0.0159	0.382	0.0184	0.2876	0.94	94
0.2	0.071	0.0451	1.08	0.0368	0.2692	0.88	88
0.3	0.14	0.0889	2.13	0.0552	0.2508	0.82	82
0.4	0.24	0.152	3.65	0.0736	0.2324	0.76	76
0.5	0.37	0.235	5.64	0.0920	0.2140	0.70	70
0.6	0.53	0.336	8.06	0.1104	0.1996	0.64	64
0.7	0.77	0.489	11.7	0.1288	0.1772	0.58	58
0.8	1.10	0.699	16.8	0.1472	0.1588	0.52	52
0.9	1.80	1.14	27.4	0.1656	0.1404	0.46	46

It is useful to determine the degree of saturation that corresponds with the drainage times. This is accomplished using columns 5 through 8. Column 5 is the product of the effective porosity and the degree of drainage, which, for U = 0.1, is 0.0184. Column 6 is the subtraction of column 5 from the volume of the voids, V_v. A value of 0.2876 is determined for a 10 percent degree of drainage. This represents the volume of water remaining in the layer. Column 7 is the quotient of column 6 (V_w) and the volume of the voids (V_v), which provides a value of 0.94. By definition, the percent saturation is the ratio of the volume of water to the volume of the voids, i.e.:

$$\text{Percent Saturation} \quad = \quad (V_w / V_v) * 100 \qquad (2\text{-}5.9)$$

Thus, the value shown in column 8 is the result shown in column 7 multiplied by 100, and represents the saturation condition of the material after 10 percent of the water has drained. The importance of the saturation level and the corresponding drainage times are discussed in the next section.

Critical Drainage Time

It has been shown before that the 85 percent saturation level is critical for granular base layers in terms of the magnitude of deformation. Thus, it is expected that the time required to reach an 85 percent saturation level will directly relate to the amount of damage done by repeated traffic loading. A drainage time of 5 hours to a saturation level of 85 percent is considered acceptable, a drainage time between 5 and 10 hours to an 85 percent saturation level represents a marginal condition, and a drainage time greater than 10 hours to an 85 percent saturation level is unacceptable.(12) A granular material that is classified as unacceptable will hold water and will exhibit large deformations, resulting in the increased probability of moisture-related distresses developing.

Figure 2-5.11 contains suggested drainage criteria levels for granular materials based on moisture-related behavior. Calculated drainage times to different saturation levels can be plotted on figure 2-5.11 to determine the acceptability of the drainability of granular materials. While preliminary drainage evaluations may be made from construction records, the final calculations should be based on the material properties of the *in situ* material.

It must be pointed out that the drainage recommendations shown in figure 2-5.11 are for high-volume interstate pavements carrying a large percentage of trucks. Lower-volume roads can tolerate a longer drainage time without sustaining excessive damage. However, a long drainage time indicates that the granular layer is a poor draining material, and moisture-accelerated distress has a much higher likelihood of developing, all other parameters being equal.

Subgrade Soil

The drainage time calculations described in the preceding sections provide insight into the drainage quality of a granular base or subbase layer, assuming the presence of an impermeable subgrade soil. However, the soil can also influence the moisture-related performance of the pavement and should be considered as part of the drainage evaluation. The drainability of the subgrade soil is a function of the soil grain size, the depth of the water table, the soil plasticity, and the topography. These factors are all included in the drainage classification of soils.

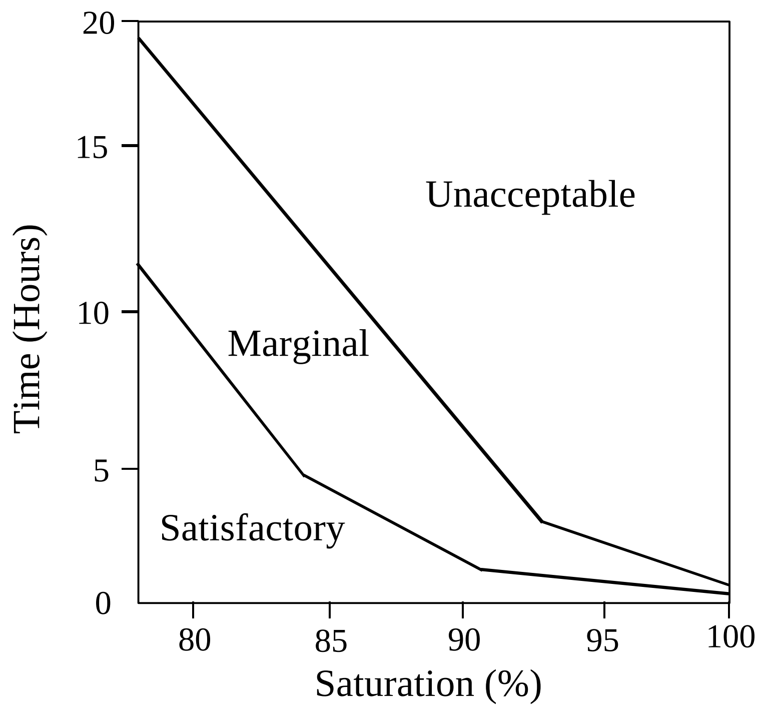


Figure 2-5.11. Drainage criteria for granular layers.

The drainability of the subgrade soil is often evaluated for agricultural purposes. County soil maps can be consulted for ratings on the drainability of the prevailing soil types within the county. These soils will be classified into one of seven distinct categories:

- Very poorly drained (VPD).
- Poorly drained (PD).
- Somewhat poorly drained (SPD).
- Moderately well drained (MWD).
- Well drained (WD).
- Somewhat excessively drained (SED).
- Excessively drained (ED).

Each of the seven drainage classifications can be given a numerical ranking based on the potential for poor moisture-related performance. This value is termed the Natural Drainage Index, or NDI.[1] The relationship between NDI and the drainage classification is shown in figure 2-5.12. NDI values between -2 and + 2 represent the majority of soils found, and are representative of an average or typical drainage situation. For purposes of classifying the drainability of the subgrade soil, the following recommendations are provided:

- NDI of -10 to -2 Good
- NDI of -2 to 2.5 Average
- NDI of 2.5 to 10 Poor

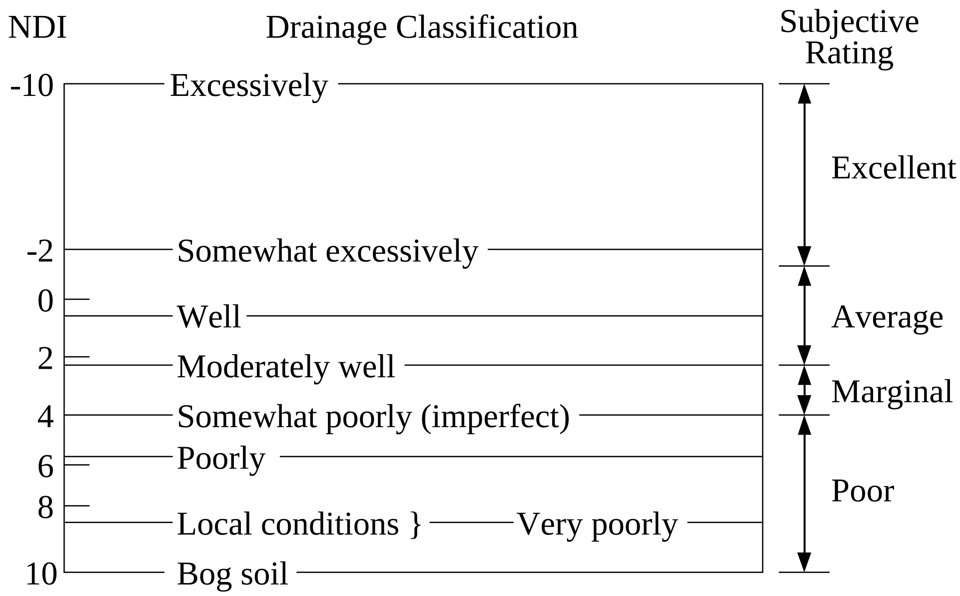
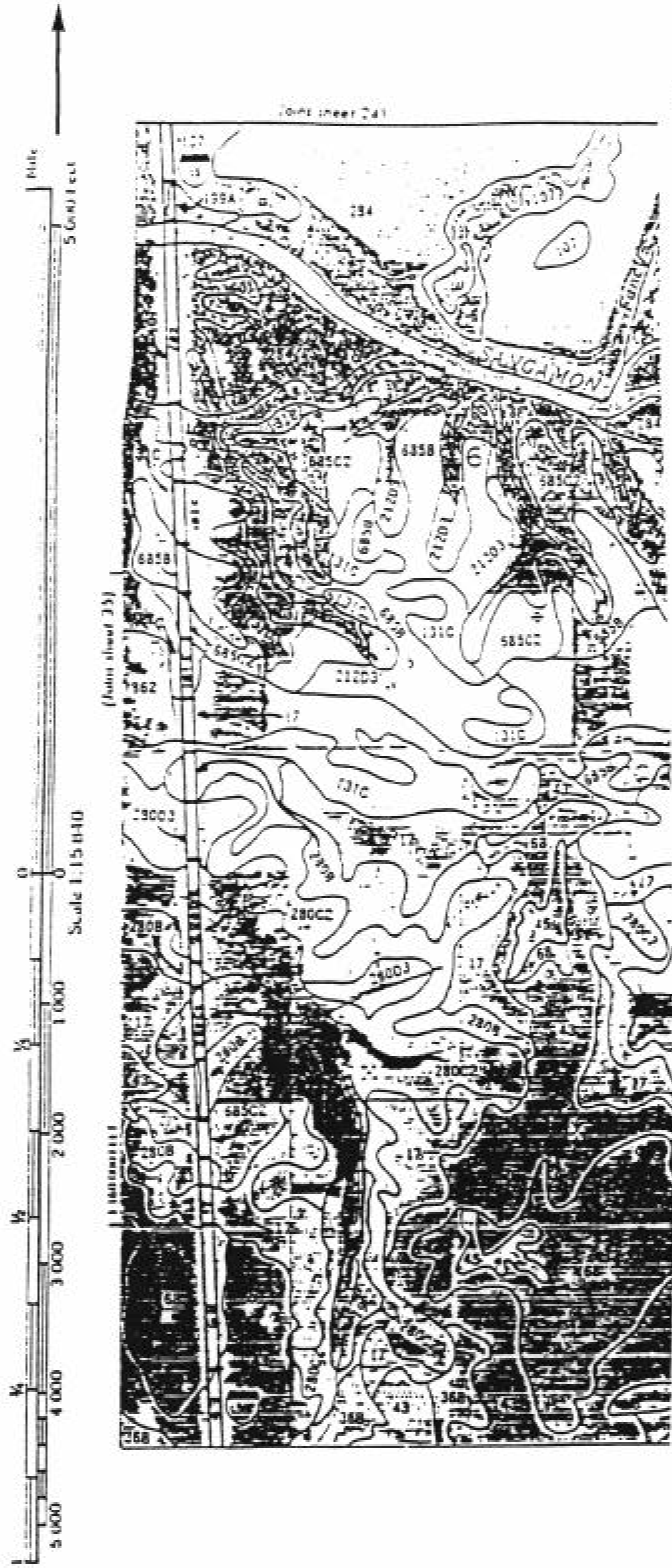


Figure 2-5.12. Relationship between drainage classification and Natural Drainage Index (NDI).[1]

In addition to the assignment of an NDI value, it is also useful to group the soils into hydrologic soil groups that estimate runoff rates from precipitation. The groups are:

- Group A—Soils with high infiltration rates when thoroughly wet.
- Group B—Moderate infiltration rate when thoroughly wet.
- Group C—Slow infiltration rate when thoroughly wet.
- Group D—Very slow infiltration rate when thoroughly wet.

As an example in assigning the NDI values and the hydrologic soil groups, consider a portion of Interstate 55 in Sangamon County, Illinois. The county soil map for this project is shown in figure 2-5.13, which also shows a summary of the percent occurrence of each soil type and its corresponding NDI value and hydrological group, these latter two values being obtained from tables in the county soil report. The percent occurrence of each NDI value can be plotted as shown in figure 2-5.14 to provide a visual indication of the potential areas where drainage problems may be expected. It is recommended that these NDI occurrences be noted at their approximate locations on the distress survey strip map. This will allow comparisons to be made and relationships developed between distresses and soil type drainage characteristics.



North End

Soil Type	Percent Occurrence	NDI	Hydrologic Group
107	4.6	PD	B/D
199A	3.1	WD	B
107	12.4	PD	B/D
8F	3.1	WD	C
131C	1.0	WD	B
8F	1.6	WD	C
131C	4.6	WD	B
685B	3.6	WD	B
685C2	3.6	WD	B
131C	4.6	WD	B
17	3.1	SPD	C
131C	1.0	WD	B
280B	3.6	WD	B
280D3	4.6	WD	B
280D2	7.8	WD	B
280B	1.6	WD	B
280C2	5.7	WD	B
280B	4.6	WD	B
685C2	3.1	WD	B
280B	2.1	WD	B
17	3.1	SPD	C
68	10.9	PD	B/D
43	6.2	SPD	B

South End

Figure 2-5.13. Information obtained from County Soil Maps for a portion of I-55 in Sangamon County, Illinois.

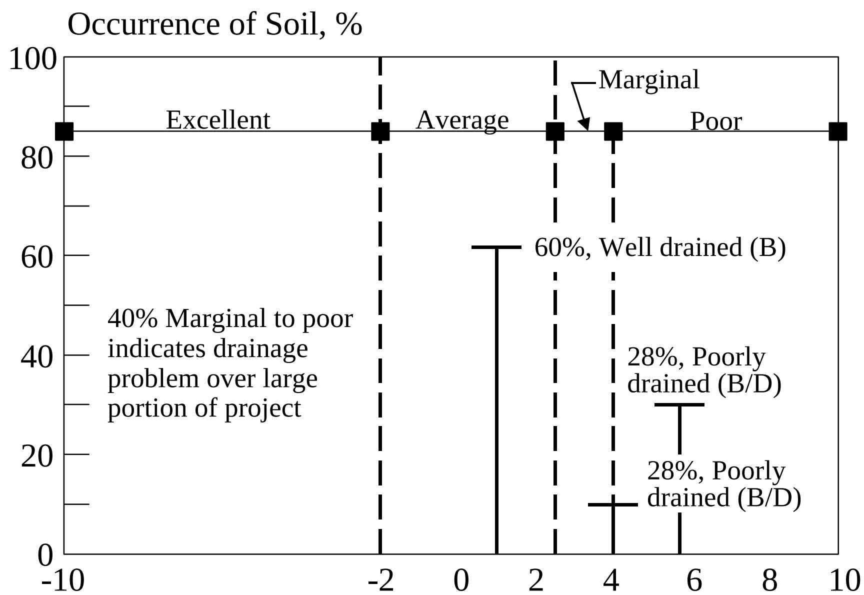


Figure 2-5.14. Occurrence of drainage classification along I-55 in Sangamon County, Illinois (hydrologic group in parentheses).

As a general rule, subgrade soils with permeabilities greater than the rainfall intensity will provide positive contributions to the drainage of the pavement structure. There are some cases in which the subgrade can serve as a source of moisture in the pavement system. Side slope cuts and "bathtub" sections can allow groundwater to flow into a pavement section, where it may pond or be held for an extended period of time. In these cases, effective drainage design is necessary to reduce the amount of time that the layers are at or near a saturation level.

6. AASHTO DRAINAGE COEFFICIENTS

Acknowledging the importance of drainage on pavement performance, the 1986 AASHTO Design Guide added a drainage term to the asphalt and concrete pavement design equations.[(2)] The purpose of the drainage coefficient is to adjust the structural design of the pavement to account for the impact of drainage on pavement life. A drainage coefficient greater than 1 represents drainage conditions better than those that existed at the AASHO Road Test, whereas a value less than 1 represents drainage conditions worse than those at the AASHO Road Test.

The determination of drainage coefficients are also important to rehabilitation design. Perhaps their most important use is that they serve as prime indicators of the drainage characteristics of the entire pavement. Low drainage coefficients indicate that a drainage problem exists and that positive steps are needed to address the problem. Drainage coefficients are also needed for the revised AASHTO overlay design procedure.

The base and subgrade drainage criteria, supplemented by the "time near saturation," can be used to determine the AASHTO drainage coefficients for PCC pavements (C_d) and for AC pavements (m_i). The drainage time and the time near saturation are the variables that must be defined to determine an AASHTO drainage coefficient for a pavement. The NDI value of the subgrade soil can be used to alter the resultant drainage coefficient within the tolerances given, depending on the subgrade soil drainage.

Saturation Time

The time the pavement is exposed to moisture levels approaching saturation comes from local knowledge and the climatic moisture map. If the subgrade has an NDI value that is poor, the time increases; if the hydrologic group is C or D, the time further increases. The time factor can be estimated by excluding the time during the year when the base may be frozen, or at least when the surface temperature is near or below freezing. The time during the dry portion of the year should also be eliminated, as any rain during this period will just add moisture to a dry soil, and usually will not saturate the soil. The Thornthwaite Moisture Index values provided in figure 2-5.2 can assist in determining moisture levels, as negative numbers indicate long dry periods while positive numbers indicate short dry periods.

The entire spring thaw period comprises part of the time near saturation for pavements constructed in areas where moisture is available and frost heave is possible. This is Zone I-A in figure 2-5.1. Other zones that have less water available or a lower frost heave potential may consider reducing the number of days accordingly.

In addition to spring thaw days, the number of days during which there is rainfall should be estimated or obtained from National Oceanic and Atmospheric Administration (NOAA) records for the nearest weather station. It can be assumed that when precipitation occurs in a measurable amount during the time the pavement is not frozen, dry, or already saturated during spring thaw, the pavement structure will be placed in a condition approaching saturation. The percent of time the pavement approaches saturation is:

$$P = (S + R)/365 \times 100 \qquad (2\text{-}5.10)$$

where:

- P = Percent of time pavement approaches saturation
- S = Days of spring thaw
- R = Remaining days with rain if pavement will drain to 85 percent saturated in 24 hours or less. If drainage time exceeds 24 hours, use days of rain times drainage time in days.

This procedure for calculating the time near saturation is good for the high moisture zones (zones I and II in figure 2-5.1), but must be adjusted for local conditions that may exist in the dry zone (zone III). Local variability can alter the results.

Combining Base and Subgrade Soil

The drainability of the base and subgrade soil must be combined to determine the overall drainability of the pavement structure, since they both influence the drainability. The combination scheme shown in figure 2-5.15 represents one method of arriving at the overall pavement drainability.[12] For example, if a base has a drainability of marginal (drainage time to 85 percent saturation between 5 and 10 hours) and the subgrade soil has a poor drainability (NDI of 4), then figure 2-5.15 indicates that the overall drainability for the pavement is poor to very poor.

		Base Drainabilities		
		A Acceptable	M Marginal	U Unacceptable
Subgrade Soil Durability	G Poor k	EXC	G	F to P
	F Poor j	G	F	P to VP
	P Good j	F to P	P to VP	VP

Quality of Drainage Criteria

EXC - Excellent G - Good F-Fair P-Poor VP-Very Poor

Figure 2-5.15. Chart to assess overall quality of drainage.[12]

Determining the AASHTO Drainage Coefficient

Having estimated the time near saturation and the overall drainability of a pavement section, table 2-5.6 can be used to determine AASHTO drainage coefficients for flexible pavements and table 2-5.7 can be used to determine AASHTO drainage coefficients for rigid pavements. For example, if a flexible pavement has an overall drainability of poor, and the pavement is saturated about 15 percent of the time, the drainage coefficient from table 2-5.6 is 0.80 to 0.90, indicating a poor overall drainability. The selection of a single AASHTO drainage coefficient may be done by considering other drainage factors, such as the presence of a drainage system, the cross slope and geometrics of the pavement, and the depth and location of the ditches.

Again, it must be emphasized that the preferred use of the AASHTO drainage coefficient is as an overall indicator of the drainage capabilities of the pavement. Values less than one indicate that the existing pavement has poor drainage characteristics that should be addressed in the pavement rehabilitation; values greater than one indicate that the existing pavement possesses adequate drainage characteristics.

Table 2-5.6. AASHTO drainage coefficients for flexible pavements.[2]

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1 - 5%	5 - 25%	Greater Than 25%
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.00 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

Table 2-5.7. AASHTO drainage coefficients for rigid pavements.[2]

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1 - 5%	5 - 25%	Greater Than 25%
Excellent	1.25 - 1.20	1.20 - 1.15	1.15 - 1.10	1.10
Good	1.20 - 1.15	1.15 - 1.10	1.10 - 1.00	1.00
Fair	1.15 - 1.10	1.10 - 1.00	1.00 - 0.90	0.90
Poor	1.10 - 1.00	1.00 - 0.90	0.90 - 0.80	0.80
Very Poor	1.00 - 0.90	0.90 - 0.80	0.80 - 0.70	0.70

7. ANALYZING THE DATA

The variation of the drainage coefficient along the length of the project should be noted on a strip map. This map should also contain any pertinent distress data found during the visual survey, including any unusual concentrations of distress. Any occurrences of moisture-related distress and poor draining materials should be noted for closer observation and comparison with the AASHTO drainage coefficient. A numerical value for each section can be developed to show the relative quality of the in-place materials related to their drainage properties. These factors must be used in any design that uses the AASHTO design procedures.

Identifying distressed areas and occurrences of poor materials will aid in selecting a rehabilitation procedure. Localized distresses in areas where the material properties do not vary appreciably indicate that problems are just beginning to appear for that material combination. A localized repair of the distress in this situation may not be useful because widespread deterioration is likely to develop shortly.

Conversely, if the location of poor materials is localized, rehabilitation of these localized areas is warranted. All the collected information should be examined together to obtain a visual picture of what moisture is doing to the pavement, where the moisture damage is occurring, and what factors are present that allow this moisture damage to occur.

The calculations described in section 5 can be easily performed in the office from construction records. However, for project evaluation purposes, they should be done using test data from cores to obtain a true indication of exactly what is present in the pavement before any final recommendations are made. The drainability of a granular base course should not vary appreciably over the length of the project. The subgrade type, topography, and whether the pavement is on a cut or fill section, will show the most variation along the length of the project, and hence will have the biggest impact on moisture-related performance.

Because of the complex and tedious nature of the various drainage calculations, a computer program has been developed to facilitate the determination of the drainability of a pavement structure and the selection of AASHTO drainage coefficients. This program is called DAMP (Drainage Analysis and Modeling Programs) and was developed for the FHWA.[14]

8. SUMMARY

Distresses in existing pavements can be quantified quite accurately as to type, severity, and amount. Certain distress types are directly related to the presence of moisture in the pavement system. Examination of the distress will indicate the extent to which moisture-related problems have developed in the pavement. Which material is producing this damage cannot be necessarily inferred from distress observations, in which case physical testing of the subsurface materials may be needed.

The following climatic and material observations are required for a drainage analysis:

- External or climatic factors.
  - Potential for moisture to exist in the pavement structure.
  - Potential for temperature interaction with the moisture.

- Internal or material properties.
  - Ability of granular layer to pass moisture to drains.
  - Ability of subgrade to assist drainage of granular layer and maintain acceptable moisture characteristics.

The internal material properties indicate the most likely candidate for moisture problems if moisture is present. Variation in material properties along the length of the pavement can be related to distress appearance and also to the structural analysis to provide a very clear picture of where and why moisture damage is occurring.

The ability to very easily indicate where excessive moisture exists and what materials are affected will greatly aid in selecting among the various 4R techniques that are available to the design engineer.

9. REFERENCES

1. Carpenter, S.H., M.I. Darter, and B.J. Dempsey, "A Pavement Moisture-Accelerated Distress (MAD) Identification System, Volume I," FHWA/RD-81/079, Federal Highway Administration, September 1981.

2. "AASHTO Guide for Design of Pavement Structures," American Association of State Highway Transportation Officials, 1986.

3. Thornthwaite, C.W., "An Approach Towards a Rational Classification of Climate," Geographical Review, Vol. 8, No. 1, 1948.

4. Janssen, D.J. and B.J. Dempsey, "Soil-Water Properties of Subgrade Soils," Civil Engineering Studies, Transportation Engineering Series No. 27, University of Illinois, 1980.

5. Rutherford, M.S., J.P. Mahoney, R.G. Hicks, and T. Rwebangiri, "Guidelines for Spring Highway Use Restrictions," Final Report, Washington State Department of Transportation, August 1985.

6. Haynes, J.A. and E.J. Yoder, "Effects of Repeated Loading on Gravel and Crushed Stone Base Course Materials Used in the AASHO Road Test," Highway Research Record 39, Highway Research Board, 1963.

7. Elzeftawy, A. and B.J. Dempsey, "A Method of Predicting Hydraulic Conductivity and Water Diffusivity for Pavement Subgrade Soils," Civil Engineering Studies, Transportation Engineering Series No. 16, University of Illinois, 1976.

8. Moulton, L.K., "Highway Subdrainage Design," FHWA-TS-80-224, Federal Highway Administration, 1980.

9. Peck, R. B., W.E. Hanson, and T.H. Thornburn, Foundation Engineering, John Wiley & Sons, Second Edition, 1974.

10. "Drainable Pavement Systems," FHWA-SA-92-008, Federal Highway Administration, Demonstration Project 87, March 1992.

11. Barber, E.S. and C.L. Sawyer, "Highway Subdrainage," Proceedings, Highway Research Board, Volume 31, 1952.

12. Smith, K.D., D.G. Peshkin, M.I. Darter, A.L. Mueller, and S.H. Carpenter, "Performance of Jointed Concrete Pavements, Volume V—Appendix B, Data Collection and Analysis Procedures," FHWA-RD-89-140, Federal Highway Administration, March 1990.

13. Haas, W.M., "Drainage Index in Correlation of Agricultural Soils with Frost Action and Pavement Performance," Highway Research Record 111, Highway Research Board, 1966.

14. Carpenter, S.H., "Highway Subdrainage Design by Microcomputer: DAMP—Drainage Analysis and Modeling Programs," FHWA-IP-90-012, Federal Highway Administration, August 1990.

15. Koerner, R.M., G.R. Koenmer, A.K. Jahim, and R.F. Wilson-Fahmy, “NCHRP Report 367 Long-Term Performance of Geosynthetics in Drainage Application,” Transportation Research Board, National Academy Press, Washington, DC, 1994.

MODULE 2-6

TRAFFIC LOADING EVALUATION

1. INSTRUCTIONAL OBJECTIVES

This module provides information on the collection and evaluation of traffic loading data for rehabilitation design. A traffic loading evaluation should be performed as part of the overall engineering evaluation of a pavement to assist in determining rehabilitation needs. An evaluation of traffic loading is of importance to all rehabilitation designs, but is of particular significance in the design of reconstruction and overlay projects. For example, future traffic levels may dictate whether patching or sealing may be sufficient for a pavement or if an overlay is required. If an overlay is required, the magnitude of that future traffic loading will determine the thickness of the overlay.

A complete traffic evaluation provides information on the estimation of past and current loadings, on the structural adequacy of the existing pavement, and on the expected future traffic loadings. By knowing the past and current traffic loadings, comparisons can be made with the design traffic to provide an indication of how well the pavement is performing and if a structural deficiency exists. The consideration of future traffic loadings is an important part of rehabilitation planning and programming and may also influence the ultimate selection of the rehabilitation strategy.

This module presents basic information for conducting an evaluation of pavement traffic loadings. Upon completion of this module, the participant should be able to accomplish the following:

1. Describe the important role that traffic loadings play in rehabilitation design.
2. Define a load equivalency factor (LEF) in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Design Guide.
3. Develop appropriate truck factors (TFs) for the various truck classifications.
4. Discuss the need to sample traffic, to convert general traffic data to project-specific data, and to improve the traffic loadings through the use of weigh-in-motion (WIM) and automatic-vehicle-classification (AVC) equipment.
5. Use historical traffic data to calculate the accumulated 80 kN equivalent single-axle loads (ESAL) for a pavement section (back-casting).
6. Use growth factors to project future traffic and compute the future expected cumulative 80 kN ESALs for a pavement section over the rehabilitation design period (forecasting).

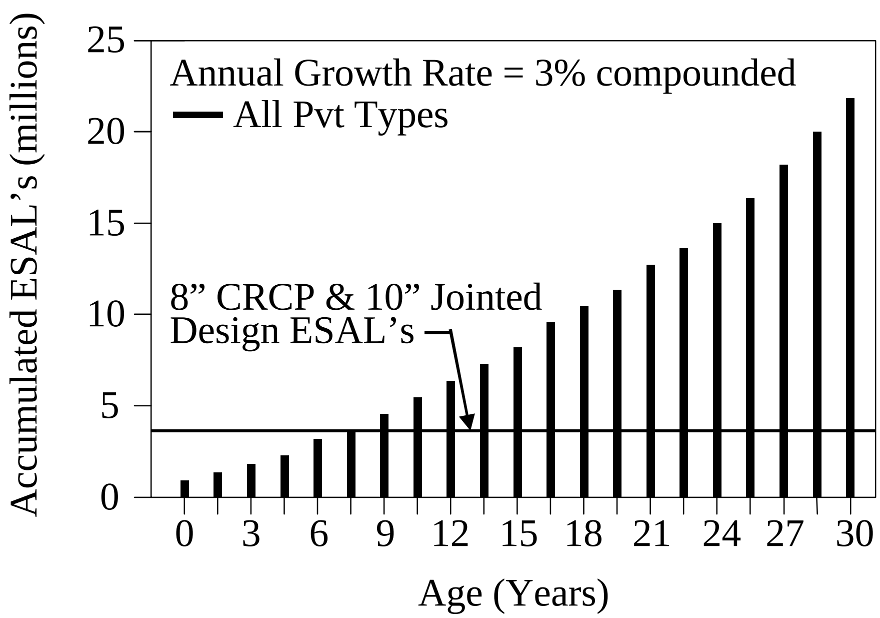
2. INTRODUCTION

An evaluation of traffic loadings is one of the most important factors in any aspect of pavement design and rehabilitation. The collection of representative data and its proper interpretation and analysis is critical in achieving the most cost-effective designs.

The most significant challenge of evaluating traffic loadings is taking into account the wide variety of vehicles that utilize the roadways. These widely varying vehicle characteristics include different gross weights, axle types and weights, and axle configurations. AASHTO is presently in the process of developing new mechanistic design procedures that will dramatically improve current procedures for taking into account this “mixed traffic flow.” These new procedures are expected to be available by the year 2002. However, until then, we continue to use the current AASHTO procedure of converting “mixed traffic flow” to common units that can be accounted for in design. The current procedure consists of employing a standard measurement of traffic loading in which all axle loads applied to a pavement structure by the mix of vehicle types are converted. This standard measurement is an equivalent number of loadings of a standard axle. The standard most highway agencies use, and indeed the one employed in most design methods, is the 80 kN equivalent single-axle load (or ESAL, for short).

In the past, most traffic projections have been grossly underestimated. The result has been that many new and rehabilitated pavements have been subjected to their design traffic loadings long before they have reached their design life (years). In turn, many of these pavements have failed prematurely in terms of their design life, although they may have sustained far more than their design traffic loadings at the time of failure.

Figure 2-6.1 illustrates this occurrence for Illinois interstate highways.[1] This figure shows the mean accumulated 80 kN ESAL applications by age for downstate Illinois interstate highways. The 20-year design traffic loading for these pavements [200 mm continuously reinforced concrete pavements (CRCP) and 255 mm jointed reinforced concrete pavements (JRCP)] was 4.8 million 80 kN ESAL applications, a value that was typically exceeded by the time the pavements were 9 years old.



1 in = 2.54 cum

Figure 2-6.1. Accumulated 80 kN ESAL applications by age for pavement in down State Illinois.[1]

Table 2-6.1 shows the inadequacy of past traffic projections for various interstate pavements across the country. The projected and actual ESAL applications for 1981 are shown, and the actual number ranges from 127 percent to 524 percent of the predicted value.

Table 2-6.1. Predicted and actual traffic loadings for various pavements across the country.[2]

LOCATION	1971 DAILY ESAL	1981 DAILY ESAL	PREDICTED 1981 DAILY ESAL	1981 ESAL AS PERCENT OF PROJECTED
I-80, NJ	3405	7838	5877	133
I-81, VA	2474	4194	3314	127
I-20, MS	1480	3121	2083	150
I-70, IL	1024	2687	1629	165
I-40, AK	860	2112	1218	173
I-35, KS	868	6629	1266	524
I-90, SD	869	1747	1165	150
I-10, CA	4155	7705	5503	140
I-5, OR	1175	4487	1804	249
			AVG.	**201**

3. DEFINITIONS

Weigh-In-Motion (WIM). A technique of weighing vehicles while in motion. The process uses any one of a number of different technologies and is commonly used in pavement research and design.

Automatic Vehicle Classification (AVC). Any one of several technologies which are capable of counting and classifying vehicles by axle spacings and axle grouping. The technology is commonly used in pavement research and design.

Equivalent Single-Axle Load (ESAL). Unit of measure in pavement design equivalent to the damage caused by the passing of a single 80 kN axle.

Load Equivalency Factor (LEF). The equivalent number of 80 kN ESALs for a specific combination of pavement type (flexible or rigid), terminal serviceability, axle type, and axle weight.

Truck Factor (TF). The equivalent number of 80 kN ESALs for a specific combination of truck axle configuration, gross vehicle weight, pavement type (flexible or rigid), and pavement terminal serviceability.

4. ESTIMATION PROCESS

This section provides background information on the steps which should be followed to develop an accurate estimate of the traffic loadings on a given pavement section for the purposes of evaluating alternative rehabilitation designs. Of course, the resources within different agencies will vary substantially and many agencies may not have the capability to obtain the most accurate data available using the latest WIM technology. However, the resources are generally available for developing an accurate estimate of present day traffic on a given pavement section. A later section of this module provides an abbreviated procedure for estimating traffic loadings on a given pavement section.

Traffic Data Collection

Traffic data collection can be achieved in many different ways. The effort put forth to obtain traffic data will depend upon many factors, including your agencies resources, the nature of the proposed rehabilitation design methodology, and the functional classification of the roadway. Generally, a more concerted effort will be put forth to evaluate roadways of higher functional classification, i.e., interstates and primary highways. Estimates for traffic loading on local roads and rural county roads can usually be obtained using simpler means, requiring far less agency resources. In either case, the more accurate the estimate of present day and future traffic loadings, the more cost-effective the final rehabilitation design.

Most highway agencies collect vehicle volume and classification data on a regular basis at many locations throughout their highway network. This data can be used to estimate both past and future traffic loadings. As a minimum, average daily traffic (ADT) and average daily truck traffic (ADTT) should be obtained since the last construction date (new construction or rehabilitation). The ADT is generally recorded as the two-way (both direction) traffic count. The ADT and ADTT data can be obtained from historical traffic volume maps or reports published by the agency, and may be supplemented by current traffic counts on the roadway under evaluation. The historical ADT and ADTT data should then be plotted versus time as illustrated in figure 2-6.2. This figure also shows an example of the different rates of growth exhibited by the ADT and by single-unit (SU) and multiple-unit (MU) trucks.

Table 2-6.2 contains a summary of truck distributions for various highway classifications in 1986.[9] The average percentage of commercial trucks, (excluding four-tired panels and pickups) is about 7 percent of the ADT on all classes of highways, with regional ranges from 2 percent to more than 25 percent expected. The estimation of future truck traffic estimates have nearly always been low, because there have been large increases in truck types and volumes over the past 25 years or more, particularly in urban areas.

Traffic Volume Growth Rates

In order to obtain a reasonable estimate of future traffic loadings, it is important that consideration be given to the growth in the future traffic. Future traffic volumes can be estimated by considering the following factors:

- Historical trends exhibited by ADT and ADTT traffic volumes (e.g., figure 2-6.2).
- Future highway system changes and land usage in the vicinity.
- General expected future trends in truck volumes in the vicinity, based upon economic, political, and other factors.

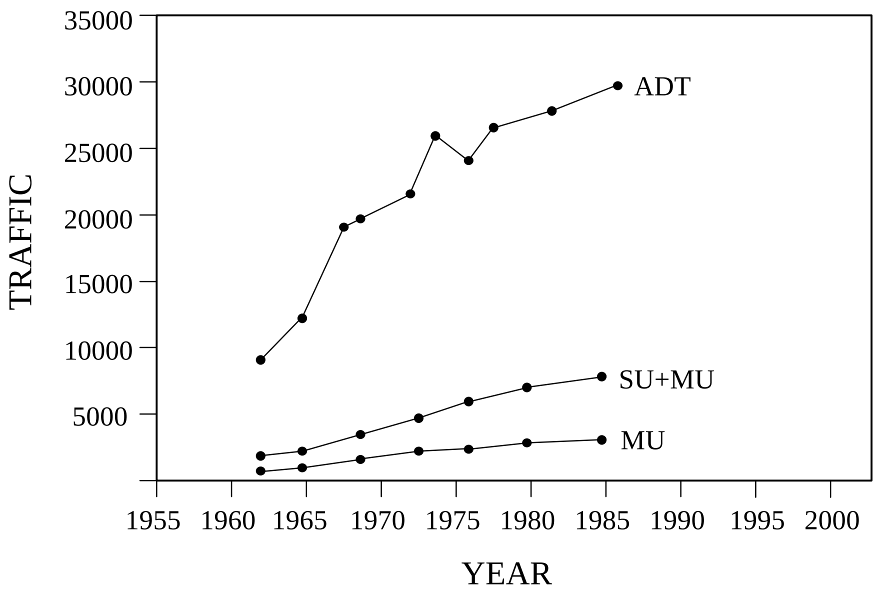


Figure 2-6.2. Traffic data for a rural interstate pavement.

This information leads to the estimation of ADT and ADTT growth over the design period. In the past, the growth in overall ADT has averaged between 2 percent and 5 percent, but the growth in truck traffic growth has been much larger. This is illustrated in table 2-6.3, which shows an average growth rate of 3.5 percent for all vehicles, a growth rate of 7.33 percent for all trucks, and a growth rate of 12.1 percent in the ESAL applications for several interstate routes in different States.[11,12] This difference in the growth rates can create large errors in the estimation of future traffic loadings if the projections are not broken out by vehicle class. The number and type of trucks should be monitored annually to best determine the range and annual growth rates for each major vehicle class.

Truck traffic growth may be expressed either as simple (growing by the same number of trucks each year) or compound (growing by the same percentage of the continually escalated truck volume each year). Compound growth rates are more commonly used, and for use in the computation of ESAL applications for rehabilitation design, it is convenient to use a traffic growth rate table, such as is shown in table 2-6.4.[9] By estimating the annual growth rate for a specific vehicle classification, and by knowing the design period for the rehabilitation design, the appropriate growth factor can be obtained for use in equation 2-6.1. This growth factor, when multiplied by the initial number of trucks, provides the total number of trucks for the design period. Again, it is strongly recommended that these projections be done by vehicle classification.

Table 2-6.2. Distribution of trucks on different classes of highways.[9]

Truck Class	PERCENT TRUCKS											
	RURAL SYSTEMS						URBAN SYSTEMS					
	Interstate	Other Principal	Minor Arterial	Collectors Major	Collectors Minor	Range	Interstate	Other Freeways	Other Principal	Minor Arterial	Collectors	Range
Single-unit trucks												
2-axle, 4-tire	43	60	71	73	80	43-80	52	66	67	84	86	52-86
2-axle, 6-tire	8	10	11	10	10	8-11	12	12	15	9	11	9-15
3-axle or more	2	3	4	4	2	2-4	2	4	3	2	<1	<1-4
All single-units	53	73	86	87	92	53-92	66	82	85	95	97	66-97
Multiple unit trucks												
4-axle or less	5	3	3	2	2	2-5	5	5	3	2	1	1-5
5-axle**	41	23	11	10	6	6-41	28	13	12	3	2	2-28
6-axle or more**	1	1	<1	1	<1	<1-1	1	<1	<1	<1	<1	<1-1
All multiple units	47	27	14	13	8	8-47	34	18	15	5	3	3-34
All trucks	100	100	100	100	100		100	100	100	100	100	

*Compiled from data supplied by the Highway Statistics Division, U.S. Federal Highway Administration.
**Including full-trailer combinations in some states.

Table 2-6.3. Example growth rates for different classes of trucks.[11,12]

	ANNUAL GROWTH RATES (PERCENT)			
LOCATION	**ALL VEHICLES**	**ALL TRUCKS**	**TRUCKS 5 AXLE OR GREATER**	**80 kN ESALS**
I-94, MT *(Wilbaux to ND)*	3.4	5.4	6.3	10.3
I-90, MT *(Billings to Laurel)*	4.0	8.1	13.1	18.9
I-90, MT *(Butte)*	2.6	4.2	9.9	N/A
I-90, MT *(Superior West)*	3.9	9.5	10.4	10.4
I-90, WA *(Cle Elum)*	2.1	N/A	5.6	8.5
I-5, WA *(Vancouver to Olympia)*	3.6	N/A	10.1	13.2
I-5, OR *(Ashland)*	4.1	8.8	11.7	12.6
I-84, OR *(Oregon-Idaho Border)*	4.4	8.0	10.4	11.1
AVERAGE	**3.5**	**7.3**	**9.7**	**12.1**

Static Weight Systems

The collection of accurate and representative truck weight data is extremely critical in estimating past or future traffic loadings. Axle type and loading have a large impact on the damage done to a pavement. In fact, axle type and weight are far more critical for pavements than vehicle gross weight. Two different trucks could have the same gross weight but cause greatly different amounts of damage to a pavement, depending upon their axle configuration.

Permanent Weigh Stations

A common source of information on truck weights is from weigh stations. These are permanent static scales, installed adjacent to a highway, that are used to weigh the trucks utilizing the highway. Results collected from these permanent weigh stations as, well as WIM stations, are summarized by the FHWA in a series of “W” tables, of which the W-4 table is of the most interest to design engineers. The W-4 table consists of information on truck axle loadings and the equivalent number of 80 kN ESAL applications.[5]

Several deficiencies exist with the use of data from permanent weigh stations. First, the number of stations in any given State is limited. According to a recent survey, the number of weigh stations varies from a low of five in one State to a high of 64 in another State, with an average of 15 locations per State.[13] Unless a loadometer station is located close to the pavement under consideration and carries the same type of traffic, it is questionable whether the load and distribution data will apply to the pavement under consideration.

Table 2-6.4. Growth factors for traffic estimates.[9]

Design Period, Years (n)	Annual Growth Rate, Percent (r)							
	No Growth	2	4	5	6	7	8	10
1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
2	2.0	2.02	2.04	2.05	2.06	2.07	2.08	2.10
3	3.0	3.06	3.12	3.15	3.18	3.21	3.25	3.31
4	4.0	4.12	4.25	4.31	4.37	4.44	4.51	4.64
5	5.0	5.20	5.42	5.53	5.64	5.75	5.87	6.11
6	6.0	6.31	6.63	6.80	6.98	7.15	7.34	7.72
7	7.0	7.43	7.90	8.14	8.39	8.65	8.92	9.49
8	8.0	8.58	9.21	9.55	9.90	10.26	10.64	11.44
9	9.0	9.75	10.58	11.03	11.49	11.98	12.49	13.58
10	10.0	10.95	12.01	12.58	13.18	13.82	14.49	15.94
11	11.0	12.17	13.49	14.21	14.97	15.78	16.65	18.53
12	12.0	13.41	15.03	15.92	16.87	17.89	18.98	21.38
13	13.0	14.68	16.63	17.71	18.88	20.14	21.50	24.52
14	14.0	15.97	18.29	19.16	21.01	22.55	24.21	27.97
15	15.0	17.29	20.02	21.58	23.28	25.13	27.15	31.77
16	16.0	18.64	21.82	23.66	25.67	27.89	30.32	35.95
17	17.0	20.01	23.70	25.84	28.21	30.84	33.75	40.55
18	18.0	21.41	25.65	28.13	30.91	34.00	37.45	45.60
19	19.0	22.84	27.67	30.54	33.76	37.38	41.45	51.16
20	20.0	24.30	29.78	33.06	36.79	41.00	45.76	57.28
25	25.0	32.03	41.65	47.73	54.86	63.25	73.11	98.35
30	30.0	40.57	56.08	66.44	79.06	94.46	113.28	164.49
35	35.0	49.99	73.65	90.32	111.43	138.24	172.32	271.02

*Factor $= \frac{(1+r)^n - 1}{r}$, where r $= \frac{rate}{100}$ and is not zero. If Annual Growth is zero, Growth Factor = Design Period.

Second, few permanent weigh stations operate continuously. Some are open only on weekdays or only during daylight hours, while others that operate on a 24-hour basis may do so only for one or two days per week. Thus, the data collected represents a very limited sample of the actual truck loading. Furthermore, numerous studies have demonstrated that the truck weight distribution varies significantly by the time of day, by the week, by the month, and by the season.[12] Thus, adjustments to the data collected must often be made following the procedures outlined in references 5, 14, and 15.

Finally, it is well known that overloaded trucks can bypass weigh stations by selecting an alternate route, or by traveling during periods when most stations are closed. Therefore, the data collected may not be representative of the actual loadings that the pavement is experiencing.

Portable Static Scales

Portable static scales are often used by agencies to collect site-specific information for rehabilitation design purposes. The fact that the data is collected on the pavement under consideration makes it more applicable and useful, but the data is plagued by many of the same problems that afflict the data collected by permanent weigh stations. Primarily, this is the fact that the data represents a very limited sampling that may not be representative of the actual conditions. That is, the data may only represent a short time period that would require weekly or seasonal adjustments, and the data again may not include overloaded vehicles, since truckers quickly learn of portable scale set-ups and can easily avoid them.

Weigh-in-Motion (WIM)

WIM scales are an important advancement in the traffic monitoring area. In existence for over 20 years, WIM scales are devices that are installed on a roadway and record the dynamic axle weights of vehicles as they travel at highway speeds. While most WIM scales are portable, permanent scales are occasionally installed.

WIM offers a high degree of flexibility in data collection and reporting with the use of high-speed digital processors. WIM devices can be installed in each lane of a multi-lane facility to provide a distribution of the loadings and traffic in each lane.

The primary advantages of WIM include:

- Elimination of truck delays, as trucks travel at highway speeds (this may be of particular importance for high-volume, urban roadways).
- Minimization of trucks bypassing scales, as there is some concealment of the devices (and they are usually not used for enforcement).
- Increase in safety by eliminating need for slow moving traffic.
- Ability to process a large number of vehicles.
- Reduction in weight data collection costs.
- Improvement in the quality and quantity of weight data.

As previously mentioned, the ESALs calculated from WIM data is believed to be more representative of actual loading conditions than the W-4 tables, since a lot of overloaded vehicles bypass permanent weigh stations.

There are several types of WIM devices currently in use. The different devices include the following:[14,16,17]

- Bridge weighing devices, in which the active weight sensing device (strain transducer) is clamped or permanently fixed to the longitudinal support beams of a highway bridge. These systems may either be portable or permanent with the portable system requiring about 2 hours to set up.

- Capacitance pads, in which three layers of steel separated by soft rubber make up the weight sensors. Capacitance pads are quite portable and can be installed in 2 hours, although they should not be installed on wet or damp pavements.

- Piezoelectric cables, consisting of small diameter, approximately 3 mm cable that generates a small electrical current when compressed. This WIM scale is most commonly used in the United States because of ease of installation and low cost.

- Strain gauge load cells, in which electrical resistance strain gauges are mounted on a load plate support. This system is available as either a permanent or portable installation.

- Strain gauge bending plates, in which steel plate load sensors are used to measure strain under load. This type of device is usually permanent.

- Hydraulic load cells, in which two rectangular platforms containing a central oil-filled piston (sensing element) are permanently affixed to the pavement in the wheel cells. Set-up time is approximately 2 hours.

One question that always arises regarding the use of WIM is their accuracy. Some comparisons between WIM scales and static scales have indicated axle weight differences on the order of 8 percent and gross weight differences of 6 percent.[17] Vehicle classification accuracy is typically on the order of 94 percent to 99 percent.[17] It should be realized that the WIM devices measure a dynamic loading effect from the passing trucks that, due to road roughness and truck suspension systems, will be different than the static truck weight. At lower speeds or on roads with an extremely smooth profile, better agreement is expected between the dynamic and the static weights.

Automatic Vehicle Classification (AVC)

AVC equipment is another technology that is similar to WIMs. In fact, AVC equipment provides data on axle spacing, axle type, vehicle type, and vehicle speed. It counts and classifies vehicles in the same fashion as WIM technology. The exception of course is that is does not provide any data on gross vehicle weight or axle weight. Therefore, it is imperative that AVC data be used in conjunction with other data sources on vehicle and axle weight such as permanent weight stations or portable WIM equipment.

The big advantage of AVC equipment is that it does not include any of the necessary hardware to weigh vehicles. AVC equipment is far less susceptible to the tortuous treatment of vehicle traffic. As a result, it can remain in place for longer periods of time without maintenance. Also, calibration of AVC equipment is much simpler than WIM equipment, since you are typically only dealing with axle spacing and vehicle speed.

Conversion

In the previous section, a variety of different methodologies were discussed for collecting raw traffic data and developing estimates for future traffic growth. In the next section, an overview of the recommended processes for converting this mixed traffic flow into a single unit of measure for design purposes is provided. To accomplish this, we will introduce the concepts of load equivalency factors and truck factors based on the AASHTO Design Guide.

LEF and TF play an integral part in the development of the number of 80 kN ESAL applications for rehabilitation design. However, these two factors are often confused and it is important that the distinction between the two be understood. These factors are defined and described in this section.

Load Equivalency Factors

Each vehicle traversing a pavement produces deflections, stresses, and strains in the pavement/subgrade layers. Each response inflicts an infinitesimal amount of damage to the pavement. With repeated applications, the amount of damage accumulates and reduces the remaining service life of the pavement. Different vehicle types, load magnitudes, and axle load configurations all produce different pavement responses that result in different levels of damage to the pavement.

Since it is very difficult to assess the damage done by every vehicle type that may utilize a pavement, design engineers find it useful to express the amount of damage done by each vehicle type in terms of the equivalent amount of damage done by a standard axle. As was mentioned earlier, the standard axle used by most highway agencies and design procedures is the 80 kN single axle. The basis for the conversion of the mixed traffic loads to the equivalent number of standard axle load applications was developed from data collected at the AASHO Road Test, conducted in Ottawa, Illinois from 1958 to 1960.[3] At the Road Test, similar pavement designs were loaded with different axle types and loadings so that the direct effect of each axle type and load on pavement damage (expressed in terms of present serviceability loss) could be ascertained. A load equivalency factor was defined as follows:

$$LEF = \frac{\begin{array}{c}\text{Number of 80kN single / axle load applications}\\ \text{to cause a given loss of serviceability}\end{array}}{\begin{array}{c}\text{Number of X / kN single / (or tandem /) axle load applications}\\ \text{to cause the same loss of serviceability}\end{array}} \qquad (2\text{-}6.1)$$

where:

X = Axle load for which equivalency is desired

For example, consider two identical pavement structures that are subjected to loadings from two different axle types. Assume that the first pavement structure sustains 100,000 applications of an 80 kN single axle for a serviceability drop from 4.2 to 2.5, while the second pavement structure sustains 14,347 applications of a 133 kN single axle for the same serviceability loss. The load equivalency factor for the 133 kN single-axle load is then:

$$LEF_{133\ kN} = 100{,}000 \,/\, 14{,}347 = 6.97$$

This means that 14,347 passes of the 133 kN single axle produce as much damage as 100,000 applications of the 80 kN single axle, or, in other words, that 1 pass of the 133 kN single axle causes the same amount of damage as about 7 passes of the 80 kN single axle.

Empirical regression equations were developed from the AASHTO Road Test data that express the relative amount of damage done by each axle load and type in terms of the equivalent amount of damage done by an 80 kN single-axle load. Load equivalency factors for flexible pavements and a terminal serviceability of 2.5 are illustrated in table 2-6.5 (for single axle) and table 2-6.6 (for tandem axles).[4] It is important to note that these equivalencies are a function of the pavement type (flexible or rigid), pavement structure (structural number, SN, for flexible pavements and slab-thickness, D, for rigid pavements), the axle type (single, tandem, and tridem), the axle load, and the terminal serviceability (P_t). It should also be pointed out that these equivalency factors were developed based on serviceability loss. Different equivalency factors would have resulted had they been based on a specific distress (e.g., rutting).

Table 2-6.5. AASHTO load equivalency factors for flexible pavements.[4]

Axle LEFs for flexible pavements, single axles and p_t of 2.5						
Axle Load (kNs)	**Pavement Structural Number**					
	1	**2**	**3**	**4**	**5**	**6**
9	.0004	.0004	.0003	.0002	.0002	.0002
18	.003	.004	.004	.003	.002	.002
26	.011	.017	.017	.013	.010	.009
53	.168	.198	.229	.213	.189	.176
71	.591	.613	.646	.645	.623	.606
80	1.00	1.00	1.00	1.00	1.00	1.00
107	3.69	3.49	3.09	2.89	3.03	3.27
133	10.3	9.5	7.9	6.8	7.0	7.8
160	24.0	22.0	17.7	14.4	13.9	15.5
187	49.3	45.0	35.6	27.8	25.6	27.7
213	92.2	83.9	65.7	50.1	44.5	46.5

As an example in the use of the load equivalency tables, consider a flexible pavement with a structural number of 5.0 and a terminal serviceability of 2.5. Referring to table 2-6.5 and table 2-6.6, the following equivalency factors are obtained for the specified axle loads and types:

LEF	Axle Load and Type
0.49	67 kN single axle
1.00	80 kN single axle
6.97	133 kN single axle
0.08	80 kN tandem axle
1.38	160 kN tandem axle
4.17	213 kN tandem axle

Table 2-6.6. AASHTO LEFs for flexible pavements (tandem axles).[4]

Axle LEFs for flexible pavements, tandem axles and p_t of 2.5						
Axle Load (kN)	**Pavement Structural Number**					
	1	**2**	**3**	**4**	**5**	**6**
9	.0001	.0001	.0001	.0000	.0000	.0000
18	.0005	.0005	.0004	.0003	.0003	.0002
27	.002	.002	.002	.001	.001	.001
53	.015	.024	.023	.018	.014	.013
80	.070	.097	.109	.092	.077	.070
107	.231	.273	.315	.292	.260	.242
133	.611	.648	.703	.695	.658	.633
160	1.38	1.38	1.38	1.38	1.38	1.38
187	2.76	2.67	2.49	2.43	2.51	2.61
213	5.08	4.80	4.25	3.98	4.17	4.49

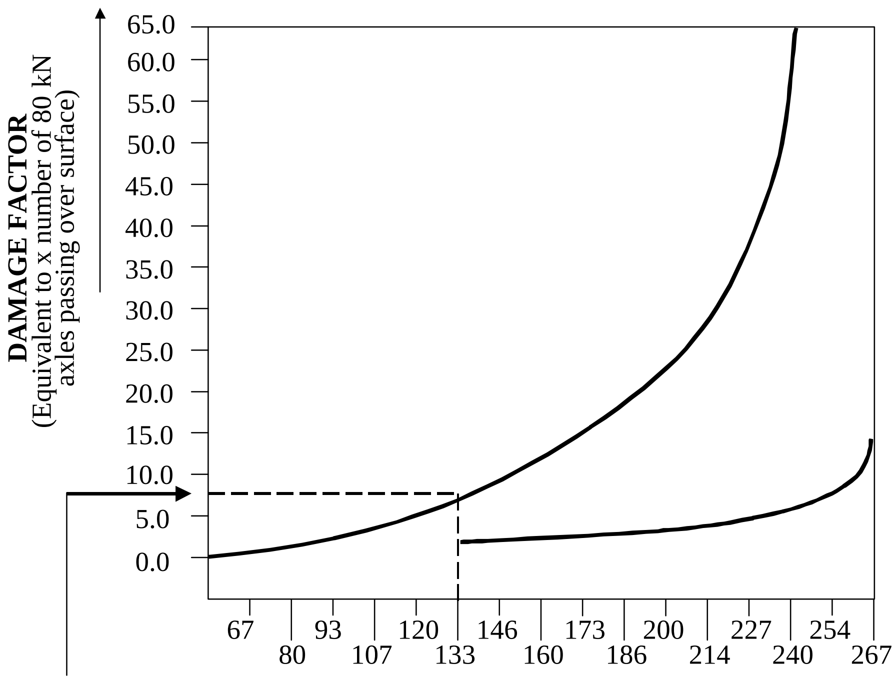
Thus, it is observed that one pass of a 98 kN single axle produces the same amount of damage as 2.18 passes of an 80 kN single axle, one pass of a 213 kN tandem axle produces as much damage as 4.17 passes of an 80 kN single axle, and so on. The LEFs will change with structural number and terminal serviceability. For purposes of clarification, a single axle is defined as an axle on a vehicle that is separated from any previous or succeeding axle by more than 2.4 m, a tandem axle is defined as two consecutive axles that are more than 1.0 m, but less than 2.4 m apart, and a tridem axle is three consecutive axles that are more than 1.0 m, but not more than 2.4 m from one of the other axles in the group.[5]

The traditional relationship between pavement damage and the applied load is that damage increases to the fourth power as the axle load increases. For example, in the above table where the load is doubled (67 kN single axle to 133 kN single axle and 80 kN tandem axle to 160 kN tandem axle), the LEF increases approximately by a factor of 2^4, or 16. This concept is illustrated in figure 2-6.3, which shows the increase in the damage factor (LEF) as the gross axle load is increased. The benefits of distributing a load over a tandem axle are also apparent, as for the same gross weight the damage factors are much less for the tandem axle than for the single axle.

Some work has been conducted to evaluate the accuracy of the AASHTO LEFs and to develop new LEFs that are more representative of current truck characteristics (axle types, configurations, and tire pressures).[6-8] Instead of conducting a full-scale road test, the approach taken is to determine critical stresses per strains in the pavement using elastic layer or finite element programs and then relate those stresses per strains to the development of pavement distress (such as rutting or fatigue cracking). Load equivalencies are then produced based on the resulting distresses caused by different axle types, loads, and configurations.

Truck Factors

While the LEFs provide a means of expressing equivalent levels of damage between axles, it is more convenient to express that damage in terms of the average amount of damage inflicted by a particular vehicle. That is, the average damage done by each axle on a vehicle are added together and expressed as



***EXAMPLE**
1 each 133 kN single axle is equivalent to 8 each 80 kN axles passing over surface

GROSS AXLE LOAD (kN)

NOTES:
1. Residual Performance Value P=2.5
2. Slab thickness for rigid pavement D=228 mm
3. Reference-these curves derived from AASHTO data

Figure 2-6.3. Relationship between damage factor (LEF) and gross axle load.[5]

the total amount of damage done by the passing of that one vehicle. This addition is incorporated within the concept of the truck factor, which is defined as the number of 80 kN ESAL applications per truck (or vehicle). A truck factor may be computed for each general truck classification or for all commercial trucks as an average for a given traffic stream. It is more accurate to compute truck factors for each general truck classification.

Typical truck factors for flexible pavements are given in table 2-6.7.[9] This table provides truck factors for various truck classifications and also for different highway classifications. For example, a truck factor of 1.09 for "tractor semi-trailers 5-axle or more" means that 100 passes of this truck causes the same amount of damage as 109 passes of an 80 kN single-axle load. The values given in table 2-6.7 are averages that may not be representative of today's heavier trucks. It is strongly recommended that design engineers obtain actual loadometer data for the specific highway under consideration and not use these averages if at all possible. It should be noted that these values are only for flexible pavements and cannot be used for rigid pavement. Typically, a truck factor for a particular truck type on a rigid pavement is about 50 percent greater than the truck factor for the same truck on a flexible pavement.

For many years, truck factors have been increasing for all categories of trucks. Reasons for this include the increase in the legal weight limit in the early 1980s, changes in axle configurations, and increased utilization of trucks. That is, a larger number of trucks are running full on both legs of their journeys, so that the number of "empty runs" has been reduced. It appears that the truck factor will continue to increase; a trend that must be considered in projecting future traffic loadings.

Example Application of LEF and TF

In order to obtain a TF that can be used for 4R design projects, LEFs must first be determined. To do this, the LEFs are used to reduce a stream of different axle loads into an equivalent number of 80 kN ESAL applications. Then, the average TF can be computed by dividing the number of 80 kN ESAL applications by the number of trucks weighed. For example, consider the following information collected from 331 trucks weighed over a specific pavement section:

Axle Type	Mass (kN)	Number of Axles	LEF	Number of 80 kN ESALs
Single	80	100	100	100
Single	98	100	2.18	218
Tandem	80	1,000	0.08	80
Tandem	213	10	4.17	42
	80 kN ESALs for all trucks weighed			440
	Total number of trucks weighed			331
	Truck Factor (= 440/331)			1.33

Thus, a total of 440 80 kN ESAL applications have been applied to the pavement, and since there were 331 trucks, this results in an average truck factor of 1.33 ESALs per truck.

Table 2-6.7. Distribution of truck factors for different classes of highways and vehicles.(9)

Vehicle Type	Truck Factors											
	Rural Systems						Urban Systems					
	Interstate	Other Principal	Minor Arterial	Collectors		Range	Interstate	Other Freeways	Other Principal	Minor Arterial	Collectors	Range
				Major	Minor							
Single-unit trucks												
2-axle, 4-tire	0.003	0.003	0.003	0.017	0.003	0.003-0.017***	0.002	0.015	0.002	0.006	---	0.006-0.015***
2-axle, 6-tire	0.21	0.25	0.28	0.41	0.19	0.19-0.41	0.17	0.13	0.24	0.23	0.13	0.13-0.24
3-axle or more	0.61	0.86	1.06	1.26	0.45	0.45-1.26	0.61	0.74	1.02	0.76	0.72	0.61-1.02
All single-units	0.06	0.08	0.08	0.12	0.03	0.03-0.12	0.05	0.06	0.09	0.04	0.16	0.04-0.16***
Tractor semi-trailers												
4-axle or less	0.62	0.92	0.62	0.37	0.91	0.37-0.91	0.98	0.48	0.71	0.46	0.40	0.40-0.98
5-axle**	1.09	1.25	1.05	1.67	1.11	1.05-1.67	1.07	1.17	0.97	0.77	0.63	0.63-1.17
6-axle or more**	1.23	1.54	1.04	2.21	1.35	1.04-2.21	1.05	1.19	0.90	0.64	---	0.64-1.19
All multiple units	1.04	1.21	0.97	1.52	1.08	0.97-1.52	1.05	0.96	0.91	0.67	0.53	0.53-1.05
All trucks	0.52	0.38	0.21	0.30	0.12	0.12-0.52	0.39	0.23	0.21	0.07	0.24	0.07-0.39

*Compiled from data supplied by the Highway Statistics Division, U.S. Federal Highway Administration.
**Including full-trailer combinations in some states.
***See Article 4.05 for values to be used when the number of heavy trucks is low.

Another example of the calculation of a truck factor is illustrated in table 2-6.8.[4] This table shows data from the weighing of "5-axle or greater tractor" semi-trailer trucks at a specific weigh station. The LEFs (called traffic equivalency factors in table 2-6.8) were obtained from table 2-6.5 and table 2-6.6. The number of axles recorded represents the grouping or distribution of weights within the indicated axle load intervals. The ESALs by axle load interval are summed to produce the total ESALs for 165 trucks of the type which were weighed. The average truck factor is shown to be 1.55 ESALs per truck. Similar calculations must be made for each class of truck in the traffic stream to provide more accurate estimates of projected traffic loadings.

Conversion of Mixed Traffic Into ESAL Applications

The conversion of mixed traffic into the number of ESAL applications for rehabilitation design applies the concepts discussed in the preceding section. This calculation can be done to estimate the past traffic that a pavement has sustained (back-casting), or to estimate the future traffic loadings that the pavement is expected to carry (forecasting). However, additional information is needed in order to perform the computation. The basic equation for the computation of the number of ESAL applications for 1 year is shown below:

$$ESAL = ADT \times TKS \times DD \times LD \times TF \times 365 \qquad (2\text{-}6.2)$$

Equation 2-6.2 provides the number of ESAL applications for 1 year in a given lane. To obtain ESAL estimates over some design period, the computation must be done for each year with any appropriate growth rates (say, on truck volumes or on the truck factor) applied over that design period. The ESALs from each year are then added up to determine the cumulative ESAL estimate. This section describes the various elements needed for that ESAL computation. It should be recognized that the ESALs calculated represents the loads applied to a single pavement lane, often referred to as the design lane. Thus, the total ESAL calculation must reflect directional and lane distributions.

Growth Rates

In order to obtain a reasonable estimate of future traffic loadings, it is important that consideration be given to the growth in the future traffic. Future traffic volumes can be estimated by considering the following factors:

- Historical trends exhibited by ADT and ADTT traffic volumes (e.g., figure 2-6.2).
- Future highway system changes and land usage in the vicinity.
- General expected future trends in truck volumes in the vicinity, based upon economic, political, and other factors.

This information leads to the estimation of ADT and ADTT growth over the design period. In the past, the growth in overall ADT has averaged between 2 percent and 5 percent, but the growth in truck traffic growth has been much larger. This is illustrated in table 2-6.9, which shows an average growth rate of 3.5 percent for all vehicles, a growth rate of 7.33 percent for all trucks, and a growth rate of 12.1 percent in the ESAL applications for several interstate routes in different States.[11,12] This difference in the growth rates can create large errors in the estimation of future traffic loadings if the projections are not broken out by vehicle class.

Table 2-6.8. Computation of truck factor for 5-axle or greater trucks on flexible pavements.[4]

Axle Load	Traffic Equivalency Factor		Number of Axles		A 80 kN EALs
Single Axles	P = 2.5, SN = 5				
Under 13.3 (kN)	0.0002	X	0	=	0.000
13.4 - 31.1	0.0050	X	1	=	0.005
31.2 - 35.6	0.0320	X	6	=	0.192
35.7 - 53.4	0.0870	X	144	=	12.528
53.5 - 71.2	0.3600	X	16	=	5.760
115.7 - 133.4	5.3890	X	1	=	5.3890
Tandem Axle Groups					
Under 26.7 (kN)	0.0100	X	0	=	0.000
26.7 - 53.3	0.0100	X	14	=	0.140
53.3 - 80.0	0.0440	X	21	=	0.924
80.0 - 106.8	0.1480	X	44	=	6.512
106.8 - 133.4	0.4260	X	42	=	17.892
133.4 - 142.3	0.7530	X	44	=	33.132
142.3 - 166.6	0.8850	X	21	=	18.585
144.6 - 151.2	1.0020	X	101	=	101.202
151.2 - 160.1	1.2300	X	43	=	52.890
	80 kN EALs for all trucks weighed			=	255.151

$$Truck\ Load\ Factor = \frac{80\ kN\ EALs\ for\ all\ trucks\ weighed}{Number\ of\ trucks\ weighed\ 165} = \frac{255.151}{165} = 1.5464$$

where:

ESAL	=Number of 80 kN ESAL applications in design lane for 1 year.
ADT	=Initial two-way average daily traffic, vehicles per day.
TKS	=Percent of ADT that is heavy trucks (FHWA class 5 or greater).
DD	=Directional distribution of truck traffic (decimal, not percent).
LD	=Lane distribution of trucks in design lane (decimal, not percent).
TF	=Average truck factor for <u>all</u> trucks, ESALs per truck.

Truck traffic growth may be expressed either as simple (growing by the same number of trucks each year) or compound (growing by the same percentage of the continually escalated truck volume each year). Compound growth rates are more commonly used, and for use in the computation of ESAL applications for rehabilitation design, it is convenient to use a traffic growth rate table, such as is shown in table 2-6.4.[9] By estimating the annual growth rate for a specific vehicle classification, and by knowing the design period for the rehabilitation design, the appropriate growth factor can be obtained for use in equation 2-6.2. This growth factor, when multiplied by the initial number of trucks, provides the total number of trucks for the design period. Again, it is strongly recommended that these projections be done by vehicle classification.

Table 2-6.9. Example growth rates for different classes of trucks.[11,12]

	ANNUAL GROWTH RATES (PERCENT)			
LOCATION	**ALL VEHICLES**	**ALL TRUCKS**	**TRUCKS 5 AXLE OR GREATER**	**80 kN ESALS**
I-94, MT *(Wilbaux to ND)*	3.4	5.4	6.3	10.3
I-90, MT *(Billings to Laurel)*	4.0	8.1	13.1	18.9
I-90, MT *(Butte)*	2.6	4.2	9.9	N/A
I-90, MT *(Superior West)*	3.9	9.5	10.4	10.4
I-90, WA *(Cle Elum)*	2.1	N/A	5.6	8.5
I-5, WA *(Vancouver to Olympia)*	3.6	N/A	10.1	13.2
I-5, OR *(Ashland)*	4.1	8.8	11.7	12.6
I-84, OR *(Oregon-Idaho Border)*	4.4	8.0	10.4	11.1
AVERAGE	**3.5**	**7.3**	**9.7**	**12.1**

It was noted in a previous section that truck factors have increased over many years, primarily because of increased legal weight limits and more efficient utilization of trucks. It is expected that these truck factors will continue to increase and it is important that these increases be considered. As with the truck volume growth factors, either simple growth or compound growth can be applied with compound being more commonly used.

Historical data on truck weights and truck factors should provide some indication of the growth in truck factors. This was illustrated in figure 2-6.3 presented earlier. The growth in the truck factor should be estimated for each vehicle classification in order to obtain a more representative estimate of future traffic loadings.

Directional Distribution

The directional distribution (DD) is the percent of truck traffic traveling in one direction. Since the ADT or ADTT are normally reported as traffic in both directions, it is necessary to compute the value for each direction of travel. In most cases, it is reasonable to assume that 50 percent of the truck traffic is traveling in each direction (i.e., DD = 0.50). For a few situations, however, more trucks may be traveling in one direction than the other. Traffic count data collected should indicate any bias in directional truck travel and the direction having the higher truck volume should be considered the design direction.

Lane Distribution

Just as the traffic may vary by direction, it will also vary across lanes on multiple-lane facilities. For example, the outer lane of a four-lane interstate highway (two lanes in each direction) will carry a higher proportion of truck traffic than the inner lane. Thus, that outer lane will also carry a larger number of the 80 kN ESAL applications. The actual distribution of truck traffic across lanes varies with the roadway type, roadway location (urban or rural), the number of lanes in each direction, and the traffic volume. Because of these many factors, it is suggested that lane distribution be measured for the project under consideration.

In lieu of project-specific data on the lane distribution, table 2-6.10 and table 2-6.11 may be consulted. However, these percentages should be applied with caution, as they are rough guidelines only. Note that a lane distribution adjustment is not necessary for a two-lane highway (one lane in each direction), since all of the trucks in each direction can only travel in one lane.

Approximation Procedures

It is sometimes useful for the design engineer to obtain a quick approximation of the past or projected number of 80 kN ESAL applications for the highway under consideration. To do this, a simplified calculation procedure can be used. The procedure is termed "simplified" because it uses an average-truck factor instead of a class-specific truck factor. As such, results obtained using the simplified procedure may not be accurate.

The 80 kN ESAL estimate using the simplified procedure may be computed from equation 2-6.2. All of the required inputs for that equation were described in the preceding section.

To illustrate the use of the simplified procedure, consider the following example:

Four-lane Rural Interstate Highway
Initial ADT (two-way) = 30,000 — 15 percent heavy trucks (class 5 or greater)
Current Truck Factor = 0.84 — Directional Distribution = 50 percent
Lane Distribution = 80 percent — Truck Volume Growth Rate = 4 percent
Truck Factor Growth Rate = 2 percent

Estimate: The number of 80 kN ESAL applications for a 5-year rehabilitation design.

To assist in determining the number of 80 kN ESAL applications, it is convenient to set up a spreadsheet to help perform the calculations. For this example, a table has been set up below:

Year	ADT	ADTT	DD	LD	TF	Yearly ESAL*	Cumulative ESAL*
1	30,000	4,500	0.5	0.8	0.840	551,880	551,880
2		4,680	0.5	0.8	0.859	586,938	1,138,818
3		4,867	0.5	0.8	0.874	621,049	1,759,867
4		5,062	0.5	0.8	0.891	658,495	2,418,362
5		5,264	0.5	0.8	0.909	698,606	3,116,968

** One direction, outer lane.*

Thus, a total of 3,116,968 80 kN ESAL applications are estimated for the roadway under consideration. Note that the ADTT for year 1 is 15 percent of the ADT and that the 4 percent growth in truck volume is applied directly to the ADTT. The truck factor is escalated by 2 percent each year to account for the growth in that value. The yearly ESAL is equal to the ADTT x DD x LD x TF x 365 (days in a year). These values are then summed in the cumulative column to come up with the estimated traffic loading.

Table 2-6.10. Lane distribution guidelines from the AASHTO Design Guide.[4]

Number of Lanes In Both Directions	% of 80kN ESAL Traffic In Design Lane
1	100
2	80 - 100
3	60 - 80
> 4	50 - 75

5. SUMMARY

This module presents basic information on traffic loading evaluation for rehabilitation design purposes. Past and projected future traffic loadings are two of the most important input parameters for rehabilitation design. Results from the traffic evaluation provide estimates of past and current loadings, the structural adequacy of the existing pavement, and future traffic projections.

The module presents background information on the use of LEFs and on the development of truck factors for estimating traffic loading. The use of historical traffic vehicle classification counts and axle load distribution data is described (W-4 tables and WIM) in order to calculate the past and projected future cumulative 80 kN ESALs for a pavement section. The need for accurate volume, classification, and weight data specific to the project site is essential, and the various types of WIM equipment available can assist in collecting that data.

Simplified and rigorous calculation procedures are presented for both back-casting and forecasting of traffic loading. While the basis for both the simplified and rigorous procedures is the same, the rigorous procedure is preferred because it breaks traffic loading out by vehicle classification so that a more representative estimate may be obtained.

The module provides a brief summary of some of the key factors affecting the ESAL calculation. Primary among these factors are the accuracy of the load equivalency factors, the collection of accurate truck weights, the presence of different axle configuration, and the presence of higher tire pressures.

Table 2-6.11. Lane distribution factors for multiple-lane highways.[18]

One-Way ADT	2 Lanes (One Direction) % Inner	% Outer	3+ Lanes (One Direction) % Inner*	% Center	% Outer
2,000	6**	94	6	12	82
4,000	12	88	6	18	76
6,000	15	85	7	21	72
8,000	18	82	7	23	70
10,000	19	81	7	25	68
15,000	23	77	7	28	65
20,000	25	75	7	30	63
25,000	27	73	7	32	61
30,000	28	72	8	33	59
35,000	30	70	8	34	58
40,000	31	69	8	35	57
50,000	33	67	8	37	55
60,000	34	66	8	39	53
70,000	---	---	8	40	52
80,000	---	---	8	41	51
100,000	---	---	9	42	59

* *Combined inner one or more lanes.*
** *Percent of all trucks in one direction.*

Another way to perform this calculation is to make use of the growth factors shown in table 2-6.4. However, it must be noted that since there are two growth rates being used (4 percent on truck volume and 2 percent on truck factor), a composite growth rate must be determined:

$$\text{Composite growth rate} = [\,(1 + g_{tv}) * (1 + g_{tf})\,] - 1 \qquad \text{2-6.3}$$

where:

g_{tv} = Truck volume growth rate (expressed as a decimal)
g_{tf} = Truck factor growth rate (expressed as a decimal)

For the example above, the composite growth rate is 6.08 percent ([1.04 * 1.02] - 1). Entering table 2-6.7 with this value and for the 5-year design period, a growth factor of approximately 5.64 is obtained. Thus, the cumulative 80 kN ESAL applications can be estimated as follows:

$$30{,}000 * 0.15 * 5.64 * 0.5 * 0.8 * 0.84 * 365 = 3{,}112{,}603$$

This value is about the same as was obtained in the table above. The difference is due to using a growth factor from table 2-6.7 for 6 percent instead of 6.08 percent.

6. REFERENCES

1. Darter, M.I., R.A. Salsilli, M.E. Dwiggins, T. Fitch, and A. Lundberg, "Analysis of Traffic Loadings on Interstate Highways in Illinois," FHWA-IL-UI-222, Illinois Department of Transportation, June 1989.

2. FHWA Memorandum, October 20, 1982, HHP-44.

3. "The AASHO Road Test, Report 5—Pavement Research," Special Report 61E, Highway Research Board, 1962.

4. "Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials, 1986.

5. "Traffic Monitoring Guide," Federal Highway Administration, June 1985.

6. Uzan, J. and A. Sidess, "Extension of Load Equivalency Factors for Various Pavement Conditions," Transportation Research Record 1286, Transportation Research Board, 1990.

7. Hajek, J.J. and A.C. Agarwal, "Influence of Axle Group Spacing on Pavement Damage," Transportation Research Record 1286, Transportation Research Board, 1990.

8. Kenis, W. J. and C.M. Cobb, "Computer Simulation of Load Equivalence Factors," Transportation Research Record 1286, Transportation Research Board, 1990.

9. "Thickness Design—Asphalt Pavements for Highways and Streets," Manual Series No. 1 (MS-1), The Asphalt Institute, February 1991.

10. Smith, K.D., D.G. Peshkin, M.I. Darter, A.L. Mueller, and S.H. Carpenter, "Performance of Jointed Concrete Pavements, Volume V, Appendix B—Data Collection and Analysis Procedures," Federal Highway Administration, FHWA-RD-89-140, March 1990.

11. Hallin, J.P., "Meeting Traffic Loading Data Needs for Pavement Analysis," Second National Conference on Weigh-In-Motion Technology and Applications, Atlanta, Georgia, May 1985.

12. Hallin, J.P., "Traffic Data for Pavement Design," Proceedings, Workshop in Pavement Rehabilitation, FHWA-TS-84-224, Federal Highway Administration, March 1985.

13. Skok, E.L., et al., "Traffic Factors Used in Flexible Pavement Design," Transportation Research Circular 240, Transportation Research Board, April 1982.

14. Desai, H.R. R. Reel, R.C. Deen, H.F. Southgate, R. Noble, V. L. Tabery, M. Hallenback, D. Crimmins, and W. Cunagin, "Traffic Forecasting for Pavement Design," FHWA-TS-86-225, Federal Highway Administration, March 1988.

15. French, A. and D. Soloman, "Traffic Data Collection and Analysis: Methods and Procedures," NCHRP Synthesis of Highway Practice 130, Transportation Research Board, December 1986.

16. Cunagin, W. D., "Use of Weigh-In-Motion Systems for Data Collection and Enforcement," NCHRP Synthesis of Highway Practice 124, Transportation Research Board, September 1986.

17. Baladi, G.Y. and M.B. Snyder, “Highway Pavements,” FHWA-HI-90-027, Federal Highway Administration, May 1990.

18. Darter, M.I., J.M. Becker, M.B. Snyder, and R.E. Smith, “Portland Cement Concrete Pavement Evaluation System (COPES),” NCHRP Report 277, Transportation Research Board, 1985.

19. Carpenter, S.H., and T.J. Freeman, “Characterizing Premature Deformation in Asphaltic Concrete Overlays of Heavily Trafficked PCC Pavements,” Transportation Research Record 1070, Transportation Research Board, 1986.

20. Kim, O.K. and C.A. Bell, “Measurement and Analysis of Truck Tire Pressures in Oregon,” Transportation Research Record 1207, Transportation Research Board, 1988.

21. Bartholomew, C.L., “Truck Tire Pressures in Colorado,” CDOH-DTP-R-89-1, Colorado Department of Highways, February 1989.

22. Elliott, R.P., R.P. Selvam, and L.K. Mun, “Effect of Truck Tire Pressure,” FHWA/AR-91/006, Arkansas Highway and Transportation Research Center, February 1991.

23. Gruver, J., P. Hazen, and K. Petros, “Importance of Trucks in Pavement Management,” Traffic Module for Advanced Pavement Management Course, Office of Environment and Planning, Federal Highway Administration.

MODULE 2-7

OVERALL PROJECT EVALUATION

1. INSTRUCTIONAL OBJECTIVES

This module presents an overview of pavement evaluation and indicates how results from the different pavement evaluation activities (described in earlier modules) are brought together in an overall project evaluation. The results of the overall evaluation are used to assist in assessing the structural capacity of the existing pavement and in selecting and developing cost-effective rehabilitation alternatives.

The participant should be able to accomplish the following upon successful completion of this module:

1. Describe the potential benefits of conducting a thorough pavement evaluation and the typical consequences of not conducting such an evaluation.
2. Outline a systematic step-by-step procedure to obtain the necessary data to conduct a pavement evaluation.
3. List key data that may be required to accomplish a satisfactory pavement evaluation.
4. Develop an overall pavement evaluation checklist that provides a summary for the design engineer to use in determining causes of deterioration, in developing cost-effective alternatives, and in communicating the results of the evaluation to management.
5. Describe the approach to conducting a rational structural evaluation of the existing pavement.

2. INTRODUCTION

Concept

The overall objective of pavement rehabilitation design is to provide a cost-effective solution that addresses the deficiencies of the pavement and that meets all of the imposed constraints (such as available funding and constructibility). This objective cannot be achieved without conducting a thorough pavement evaluation to determine the causes (underlying mechanisms) and extent of deterioration. This requires systematic data collection and an analysis of the structural and functional condition of the pavement as well as several other factors. The approach described in this module is consistent with that describe in the *AASHTO Guide for Design of Pavement Structures*, Part III, Chapter 2, “Pavement Design Procedures for Rehabilitation of Existing Pavements: Rehabilitation Concepts.”[1]

Importance of a Thorough Pavement Evaluation

Pavement rehabilitation design is a very complex engineering task, often requiring more engineering than new design. However, in the past this part of a project has often been emphasized the least. Many agencies have learned from experience that spending a little more time and effort to adequately evaluate and design a rehabilitation project will more than pay for itself in savings in initial construction costs and in future maintenance and rehabilitation costs. In fact, when evaluated in terms of life-cycle cost, it can be demonstrated that the $10 to $50 thousand in up-front cost for a thorough project evaluation will pay for itself if it results in only 1 month of additional pavement service life. The expected benefit, however, is likely to be in additional years.

The size of a project, of course, dictates the amount of time and funds that can justifiably be spent on pavement evaluation. Large projects, projects on major highways, and projects in high traffic volume areas 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 prior to rehabilitation 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 in all the different areas), or a decision made to replace the car. Even if the car is driven every day. It is still necessary to perform such an overall evaluation. The consequences of neglecting such an evaluation could result in a very poor (and expensive) decision.

3. DATA REQUIRED TO ACCOMPLISH A PAVEMENT EVALUATION

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

- Pavement condition (e.g., distress, roughness, surface friction, deflections).
- Shoulder condition.
- Pavement design (e.g., layer thicknesses, structural characteristics, construction requirements).
- Materials and soil properties.
- Traffic volumes and loadings.
- Climatic conditions.
- Drainage conditions.
- Geometric factors.
- Safety aspects (e.g., accidents, surface friction).
- Miscellaneous factors (e.g., utilities, clearances).

Actually, the specific data to be collected under each of these general categories also depends upon the pavement rehabilitation alternatives to be considered. For example, if recycling of an asphalt surface course is to be considered, samples of the material are needed and a mix design should be performed to determine approximate proportions of materials, and to determine if the mix is suitable for the level of traffic expected after rehabilitation. If grinding of a concrete pavement is to be considered, then the hardness of the aggregate and the faulting condition must be know. The earlier modules in this block provide detailed information on certain important aspects of pavement data collection, processing, and interpretation. This module describes how those data and other information are brought together to conduct a thorough pavement evaluation.

With this in mind, a summary table (table 2-7.1) has been prepared that shows the need for specific data collection for selected rehabilitation alternatives.[1] The data are classified as “Definitely Needed,” “Desirable,” or “Not Normally Needed.” Each agency should prepare a similar list for each pavement type, governed by local conditions.

Table 2-7.1. Suggested data collection needs for designing and constructing portland cement concrete (PCC) rehabilitation alternatives.[(1)]

Data Item	Full-Depth Repair	Partial-Depth Repair	Overlay	Grinding	Recycling	Undersealing	Slab Jacking	Subdrains	Joint Resealing	Pressure Relief Joints	Load Transfer Restoration	Surface Treatment
Pavement Design	!	!	!	!	!	!	!	!	!	!	!	!
Originl Construction Data			◦	◦	◦ '			◦	◦	◦	◦	
Age	◦	◦	◦	◦	◦			◦				◦
Materials Properties	◦	◦	!	!	!	◦		!				
Subgrade			!		!	◦	◦	!	!			
Climate			!		"	!		!	!	◦		!
Traffic Loading and Volumes	!	!	!	!	◦	◦		!	◦	◦	!	!
Distress	!	!	!	!	!	!	!	!	!	!	!	!
Skid			◦	◦	◦							◦
Accidents			◦	◦	◦							
NDT	◦		!		◦	!					!	◦
Destructive Testing/ Sampling	!	!	!	◦	!	◦		!			◦	
Roughness			◦	◦	◦		◦					◦
Surface Profile			◦	!			◦					

KEY: **!** Definitely Needed ◦ Desirable [*blank*] Not Normally Needed

Table 2-7.1. Suggested data collection needs for designing and constructing PCC rehabilitation alternatives.[1] (cont'd)

Data Item	Full-Depth Repair	Partial-Depth Repair	Overlay	Grinding	Recycling	Undersealing	Slab Jacking	Subdrains	Joint Resealing	Pressure Relief Joints	Load Transfer Restoration	Surface Treatment
Drainage	!		!	!	!	!		!	!			◦
Previous Maintenance	◦	◦	◦	◦	◦	◦		◦	◦	◦		◦
Bridge Pushing			◦						!	!		
Utilities	!		!		!	◦	◦	◦				
Traffic Control Options	!	!	!	!	!	!	!	!	!	!	!	!
Vertical Clearances			!		!							
Geometrics			!		◦							

KEY: **!** Definitely Needed ◦ Desirable [*blank*] Not Normally Needed

Table 2-7.2 presents a checklist of many of the factors that should be evaluated and specific questions that must be addressed for a thorough pavement evaluation. Of these factors, the structural evaluation is the most important, because if there is a structural deficiency the only logic rehabilitation strategy is to increase the pavement structure through an overlay or reconstruction (with recycling as an option). Each agency should modify this overall list of evaluation items and questions to meet their own needs. It is also desirable to develop procedures to answer the questions in a consistent manner. For example, some items could be obtained from direct testing results such as roughness, coring, soil classification, permeability calculations, and previous research on materials performance. Many questions could be answered using the procedures provided or referenced in various modules of this manual, or in other sources such as the AASHTO Design Guide.

Data collection serves the following important purposes in pavement evaluation and rehabilitation:

- 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 for rehabilitation projects (e.g., labor, materials), to develop rehabilitation alternative designs (e.g., overlay thicknesses), to assess the rate of deterioration of the pavement and the consequences of delaying rehabilitation, and to perform life-cycle cost comparisons of rehabilitation alternatives.

In pavement evaluation as in rehabilitation design, the engineer's objective is economy: to make the most efficient use of data collection resources so that sufficient information can be obtained to design feasible and cost-effective rehabilitation alternatives. This is best achieved by an iterative approach such as is described in this module.

4. PROJECT EVALUATION FLOWCHART

The data must be carefully evaluated and the results summarized in a systematic fashion. The exact procedures used will vary from project to project, depending on what is found during the data collection process. Figure 2-7.1 provides an example flowchart illustrating a step-by-step procedure for project evaluation. It describes how the data collection activities discussed in earlier modules fits into the overall process. Following is a discussion of the individual steps in this process.

Step 1: Office/Historical Data Collection

This step involves the collection of useful information from office files and other historical records that can have a significant impact on the pavement evaluation and ultimate selection of the preferred rehabilitation alternative. Included under this data search 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.

Table 2-7.2. Overall pavement evaluation summary and checklist.(1)

STRUCTURAL EVALUATION			
	Existing Load-Associated Distress G Major G Minor	Structural Load-Carrying Capacity Deficiency (to carry future traffic over design period)?	
FUNCTIONAL EVALUATION			
	Roughness or Present Serviceability Rating G Very Good G Good G Fair G Poor G Very Poor	Friction Resistance G Satisfactory G Marginal G Unsatisfactory	
	______ Present Serviceability Index/Rating	Rutting Severity G Low G Medium G High	
VARIATION OF CONDITION EVALUATION			
	Systematic Variation along project? (e.g., condition varies from one end of project to other) G Yes G No	Systematic Variation between lanes? G Yes G No	Localized Variation (e.g., bad areas) along project? G Yes G No
CLIMATIC EFFECTS EVALUATION			
Climatic Zone	Moisture Region I Moisture throughout year II Seasonal moisture III Very little moisture	Temperature Region A Severe frost penetration B Freeze-thaw cycles C No frost problems	Severity of moisture-accelerated damage G Low G Medium G High ____________________
	Base drainage quality G Excellent G Good G Fair G Poor G Very Poor	Subgrade drainage quality G Excellent G Good G Fair G Poor G Very Poor	Surface drainage capability G Acceptable G Needs improvement Describe ____________________

Table 2-7.2. Overall pavement evaluation summary and checklist.(1) (cont'd)

PAVEMENT MATERIALS EVALUATION			
	Surface G Sound Condition G Deteriorated Describe ________________	Base G Sound Condition G Deteriorated Describe ________________	Subbase G Sound Condition G Deteriorated Describe ________________
ROADBED SOIL EVALUATION			
Structural Support (Stiffness) G Low G Medium G High	Moisture Saturation Softening G Low G Medium G High	Temperature Problems G None G Frost Heaving G Freeze-Thaw Softening	Swelling Potential G Yes G No
PREVIOUS MAINTENANCE PERFORMED			
	Previous Maintenance Performed? G Minor G Normal G Major	Has lack of maintenance contributed to deterioration? G Yes G No Describe ________________	
RATE OF DETERIORATION EVALUATION			
	Long-term, from construction or last overlay G Low G Normal G High	Short term, over past 1 to 2 years G Low G Normal G High	
PROJECT TRAFFIC CONTROL			
	Are detours available so that facility can be closed? G Yes G No	Must construction be accomplished under traffic? G Yes G No	Could construction be done at off-peak hours? Describe ________________

Table 2-7.2. Overall pavement evaluation summary and checklist.[1] (cont'd)

GEOMETRIC AND SAFETY FACTORS			
	Current Traffic Volume Capacity G Adequate G Inadequate	Future Traffic Volume Capacity G Adequate G Inadequate	New lane required over design period G Yes G No
	Lane widening required now? G Yes G No	List high accident locations __________ __________	Bridge clearance problems
	Lateral obstruction problems __________ __________	Utilities problems __________ __________	Bridge pushing problems __________ __________
TRAFFIC LOADINGS			
	Current ADT (2-way) __________ Current % trucks (2-way) __________	Accumulated and projected 18-kip ESALs, each lane __________ __________	Current 18-kip ESALs/year, each lane __________ __________
SHOULDERS			
	Overall Condition G Good G Fair G Poor	Localized deteriorated areas? G Yes G No	

Data of this nature is usually not easy to assemble; however, it is usually less expensive and time-consuming (to gather) than trying to re-establish it.

Step 2: Initial Site Visit

During this initial inspection of the project, the design engineer and maintenance engineer should work closely to; 1) determine the scope of the primary field survey (step 3), 2) begin to assess the potential distress mechanisms, and 3) identify the candidate rehabilitation alternatives. As part of this activity, subjective information on distress, road roughness, surface friction, and moisture/drainage problems should be gathered. Unless traffic volume is a hazard, this type of data can be collected without any traffic control, through both "windshield" and road shoulder observations. In addition, an initial assessment of traffic control options (both during the primary field survey and during rehabilitation construction), obstructions and safety aspects should be made during this visit.

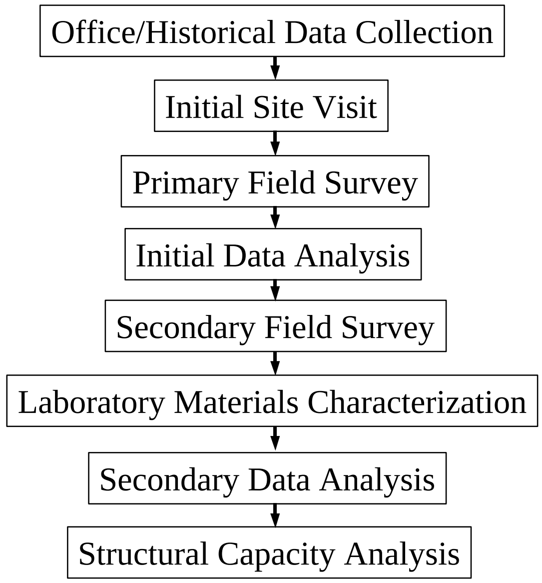


Figure 2-7.1. Example flowchart for project level pavement evaluation.

In terms of the impact on the scope of subsequent primary field survey:

- 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 sagging/swelling problems.

- Observation of moisture/drainage problems (e.g., pumping, corner breaks, standing water, etc.) May indicate the need for a more intensive nondestructive testing (NDT) data collection program and/or a more thorough investigation of subsurface drainage testing.

Step 3: Primary Field Survey

Because of the expense associated with conducting field surveys (especially if traffic delays/control are involved), it is usually not practical to conduct more than one detailed field survey of the project. (The exceptions are when data from the primary survey indicate that more detailed investigations such as destructive sampling and testing are required). To minimize the number of lane/road closures and overall disruption to the road user (even for this primary field survey), the process should be timed and coordinated as efficiently as possible.

The essential data covered under this important activity include:

- Condition survey.
- Nondestructive testing.
- Drainage survey.
- Traffic survey.

Condition Survey

Collection of pavement condition data relative to distress, roughness, and friction is well covered under module 2-2. Of the three, distress is the most rigorous in terms of manpower because of its likely impact on the estimation of repair quantities as well as the rehabilitation selection. Besides the systematic process of collecting these data using trained/calibrated technicians, it is important to re-emphasize the need for both the pavement design engineer and the maintenance engineer to "walk the project." Done together, this will help them both get a better understanding of the working distress mechanisms (as described in module 2-1) as well as the feasible rehabilitation alternatives.

For project level evaluation, recall that it is also reasonable to assume that subjective assessments of roughness and surface friction (by the engineer) will be satisfactory. The reason for this is that once it has been determined that a rehabilitation treatment is called for, the "actual" level of roughness or friction would not have much additional impact on the engineer's selection of an appropriate treatment.

Nondestructive Testing (NDT)

The NDT data collection process is thoroughly covered under module 2-3. It, along with the distress surveys, make up the backbone of a comprehensive pavement evaluation and rehabilitation design process. The scope of the NDT program should be established by the design engineer during/after the initial site visit.

Drainage Survey

Collection of information on drainage conditions is addressed in module 2-5. Like roughness and friction data, information on moisture and drainage conditions at the project site can initially be collected using a subjective (in this case, visual) approach. Unlike roughness and friction data collection, however, the engineer may have observed some conditions during the initial site visit (e.g., pumping, corner breaks, standing water in ditches, etc.) That may dictate the need for a more rigorous drainage testing program or a more intense NDT data collection. If so, these would be carried out at this time.

Traffic Survey

In cases where the information needed for estimating past traffic and forecasting future traffic is unavailable (or very poor), it may be necessary to collect current traffic information using a weigh-in-motion (WIM) installation such as that described in module 2-6. If vehicle weighing is not practical, then a vehicle classification survey can be used to estimate vehicle weights using average weight per class measured in the area.

Step 4: Initial Data Analysis

For each element of the field survey data collection activity, there is a corresponding element of initial data analysis. The details associated with these analyses are covered under their respective modules. In general, the end result is a "strip chart" indicating the condition of various points along the project. Figure 2-7.2 provides an example of a compound strip chart.

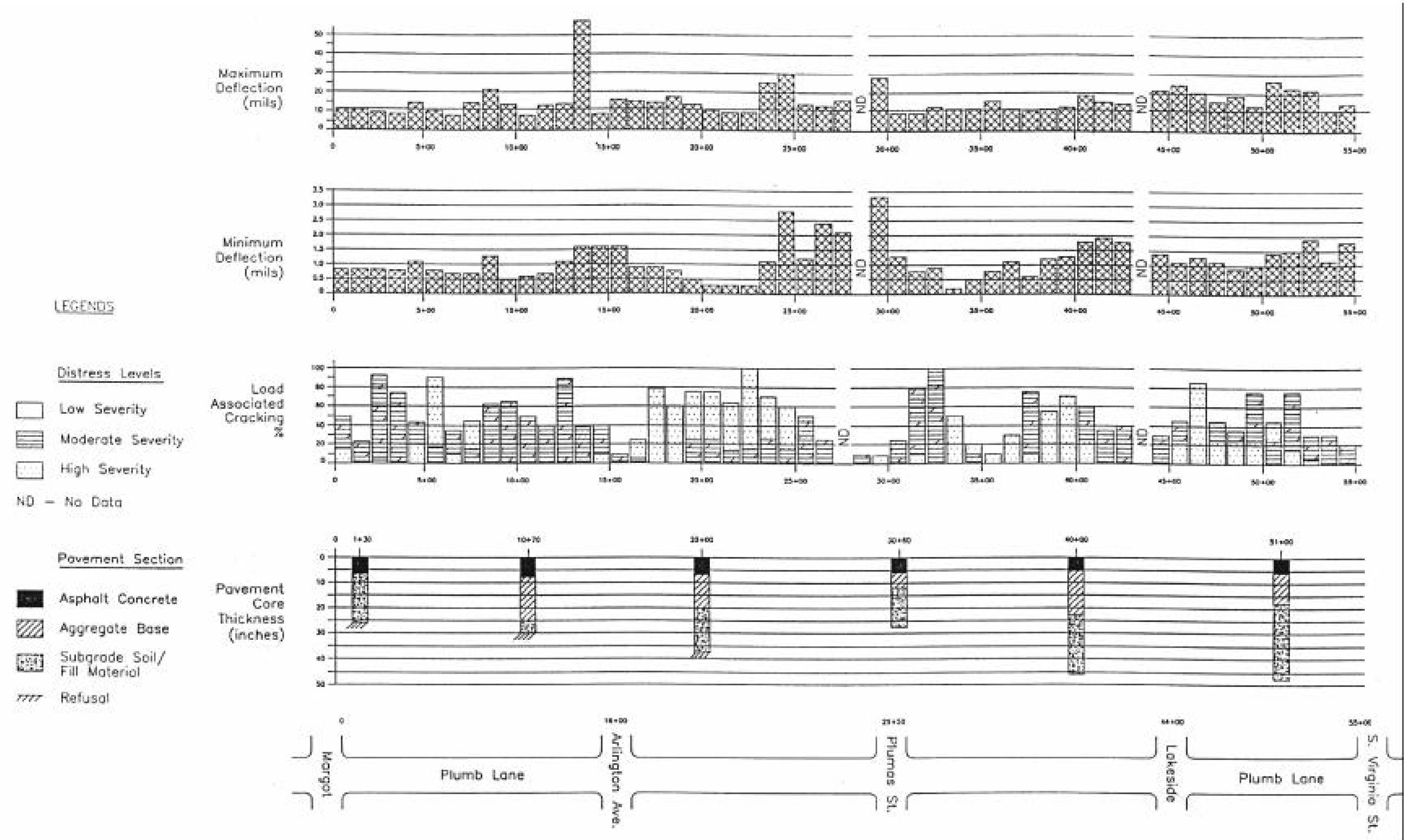


Figure 2-7.2. Example composite "strip chart" showing pavement structure, distress severity levels, minimum deflections, and maximum deflections.

Nondestructive Testing

As discussed in module 2-3, NDT data can be processed and interpreted in several ways:

- Deflection profile plots.
- Backcalculation.
- Load transfer.
- Void potential.

Deflection Profile Plots

By preparing profile plots of the maximum (sensor 1) and minimum deflections at each location along the project, it is possible to get a sense of where the good to poor subgrade soil conditions are as well as the strong to weak pavement sections. In fact, this information is very helpful in identifying subsections within the project (i.e., unit delineation) and indicating locations where distress, poor moisture conditions, cut/fill, etc. may be having an adverse impact. If cross section information is unavailable, it can also help identify appropriate core locations along the project for layer thickness

Backcalculation

Through the use of certain types of software, it is possible to estimate the elastic moduli of the individual pavement and soil layer at each NDT location so that a better assessment of structural capacity can be made. Once these layer moduli are "backcalculated," they too can be plotted (in addition to or in lieu of the NDT data). In many respects, these plots would have more significance than the NDT plots.

Load Transfer

The observation of significant differential vertical movement between one side of a joint (in a rigid pavement) and the other is an indication that poor load transfer conditions exist. The statistic for severity of this distress mechanism is known as load transfer efficiency and is a function of certain NDT measurements. It too can be calculated and plotted on a longitudinal profile plot to illustrate varying conditions along the project and, ultimately, where load transfer restoration may be appropriate.

Void Potential

There are various statistics that can be calculated as a function of the NDT measurements made at the corner of every leave slab of a jointed concrete pavement to indicate the potential for an underlying void. Regardless of which statistic is used, it too can be plotted to graphically illustrate what locations along the project are in need of slab stabilization (undersealing).

Condition Data

As with NDT data, pavement condition data can be plotted versus distance 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. Fatigue cracking and permanent deformation are obvious candidates for these kinds of illustrations, however, thermal cracking, corner breaks, midslab cracks, faulting, and spalling can also be shown.

Moisture/Drainage Data

By now, it should be apparent that if data were collected to represent some condition along the project, it can be depicted in a profile plot. Conditions relative to high moisture/poor drainage and cuts/fills are, obviously, no exception. When examined on a profile plot (along with other factors), it can help identify areas where more detailed subsurface drainage testing is required.

Traffic Loading Analysis

Under this element, available traffic information is used to estimate the number of 80-kN ESALs the pavement has carried since it was constructed as well as the number it will carry for the remaining design life. "Backcasted" traffic is used to estimate remaining life while forecasted traffic is used to determine the structural rehabilitation needs.

Step 5: Secondary Field Survey

A second field survey is warranted only if data collected in the first survey are inadequate or insufficient to assess the underlying distress mechanisms or properly define the most appropriate rehabilitation alternative. Typically, the need for a second survey is identified after examining the data from the first field survey. Following are examples of data that might be collected in this survey to include both destructive samples and additional nondestructive testing.

Destructive Samples

Pavement cores, auger samples, or test pits or dynamic cone penetrometer (DCP) may be required if:

- NDT indicate significant variability along the project that cannot be attributed to changes in soil or cross section.
- The backcalculation process struggles with two layers of similar stiffness (e.g., an hot-mix asphalt (HMA) layer and a stabilized base layer).
- Problems with stripping are discovered.

Additional Nondestructive Testing

Possibilities for additional NDT include:

- Intensive deflection testing because of an undiagnosed problem with load transfer, voids or moisture.
- The use of other types of NDT devices (e.g., radar, infrared thermography, SASW, etc.) to evaluate thickness variations or better investigate an underlaying mechanism.

Step 6: Laboratory Materials Characterization

As discussed in module 2-4, laboratory testing is a more limited component of a thorough project evaluation. The reason for this is the emergence of improved backcalculation techniques for interpreting NDT data. These techniques provide results for actual in-situ conditions at much more frequent intervals than traditional field sampling and laboratory testing.

Unfortunately, NDT does not always provide all the needed data. Examples where laboratory testing might be required include:

- Indirect tensile strength of HMA surface or PCC layer.
- Resilient modulus of one or more bound layers that have similar stiffnesses.
- Permeability of pavement/soil layers.
- As-constructed mix characteristics for HMA or asphalt stabilized layers (e.g., asphalt content air void content, gradation, stability, etc).
- Density and gradation of underlying layers.
- Freeze-thaw durability of PCC layer.
- Petrographic testing of PCC layer.

The results of these laboratory tests are used to complement the data and analyses of the primary survey.

Step 7: Secondary Data Analysis

If necessary, secondary data analysis is carried out to examine the results of any secondary field surveys (step 5) or laboratory materials characterization (step 6). These analyses are dependent upon the types of mechanisms being investigated and range from empirical to mechanistic in nature. The goal of this analysis is to provide information upon which to make a better assessment of the underlying mechanisms and identify the appropriate treatments to address them.

Step 8: Structural Capacity Assessment

Structural capacity refers to a pavement's ability to support current and future traffic loadings. A pavement is considered structurally deficient if it is unable to sufficiently withstand current or future traffic. Knowledge of a pavement's structural capacity provides valuable information in the selection and design of feasible alternatives to rehabilitate that pavement.

Three methods for assessing structural capacity are discussed below (with emphasis on the third). The key feature of a structural evaluation is the determination of the structural capacity of an existing pavement, of which load-related visual distress is one indicator and magnitude of measured surface deflections is another.

Structural Evaluation by Existing Distress

Pavement distresses provide critical insight into a pavement's structural condition. A pavement traffic lane is considered to have failed structurally if it meets or exceeds the levels of (load-related) distress shown in table 2-7.3.[7] When a pavement exhibits these levels of distress, the rate of deterioration is usually very high and a major structural improvement is required.

Plotting existing structural distress along the length of each lane of a project (as described in step 4) is an effective way of illustrating the location and amount of distress. A significant amount of load-related distress along any section is indicative of a pavement structural deficiency. However, data from NDT and materials testing are often needed in addition to the distress data to assist in determining the structural adequacy of the pavement.

Table 2-7.3. Structural failure levels defined by distress types and densities.[7]

DISTRESS	PAVEMENT TYPE			
	AC	JPCP	JRCP	CRCP
Alligator Cracking	> 10%			
Rutting	> 13 mm			
Cracked Slabs		> 10%		
Deteriorated Transverse Cracks			> 45m/lane km	
Deteriorated Joints			> 50%	
Punchouts, Steel Ruptures, and Patches				> 6/lane km

Structural Evaluation by Component Analysis

In component analysis, materials sampling and tests are performed to obtain the type, thickness, and condition of the various pavement layers. This information can be used to assign structural coefficients to each layer based on its structural adequacy, as described in reference 1. This procedure is employed primarily for assessing the structural capacity of existing flexible pavements or fractured (e.g., cracked and seated) rigid pavements.

The structural coefficients are used to calculate a structural number (SN), which is an indicator of the overall load carrying capacity of a flexible pavement system. Using the SN and the subgrade soil resilient modulus, the remaining pavement life can be determined in terms of the estimated number of furure traffic loadings that can be sustained. If the pavement materials have not deteriorated substantially due to moisture, traffic, and the environment, the structural adequacy of the pavement may be determined using this method in conjunction with the distress survey. However, it is difficult to apply this procedure successfully if the materials have deteriorated considerably. This approach may not provide reliable results due to the inherent variability of pavement performance.

Structural Evaluation by Nondestructive Testing

NDT used in conjunction with the distress survey and limited materials sampling and layer thickness determination, is by far the most reliable method for determining the structural adequacy of both rigid and flexible pavements. NDT measurements of pavement deflection under a given load can provide the following:

- Deflection variability along the project for use in selecting distinct design sections.
- Detailed deflection studies at localized areas to ascertain causes of distress, to locate inadequate support or voids, and to determine load transfer efficiencies at joints and cracks.
- Critical pavement deterioration periods based on seasonal deflection variation due to environmental effects.
- An indication of the pavement's ability to support present and future traffic loadings.

- Inputs to an overlay design procedure to determine the required structural improvement.
- The data required to conduct a fatigue damage analysis using backcalculated modulus values.

Following is an example of a structural evaluation for a simple flexible pavement structure having a 50 mm HMA surface layer, a 150 mm crushed stone base coarse and an A-6 (clayey) subgrade soil. The visual survey indicates only the initiation of load-related distress (i.e., alligator cracking).

NDT was performed using a falling weight deflectometer (FWD) and testing was not performed in any cracked areas (assuming that these areas will be repaired prior to any rehabilitation). The testing load was approximately the same as the design load (40-kN). Using the measured deflections and structural characteristics, backcalculation was performed to estimate the in-situ layer moduli. After correction for design temperature, these modulus values were:

HMA Surface:	1,500 mPa
CS Base:	165 mPa
Subgrade Soil:	57 mPa

A multilayer elastic theory computer program (ELYM5) was then used to estimate the maximum tensile strain at the bottom of the HMA surface layer as a result of the 40-kN design wheel load (80-kN design axle load). The result was a value of 0.000387. When incorporated into the Corps. of Engineers HMA fatigue equation,

$$\log N_{ac} = -[5 \cdot \log(e_{ac}) + 2.665 \cdot \log(0.0102 \cdot E_{ac}) + 0.392] \quad (2.7.1)$$

where:

N_{ac} = Allowable number of axle load applications to onset of fatigue cracking.
e_{ac} = Tensile strain at the bottom HMA layer, and
E_{ac} = Elastic modulus of HMA layer (kPa),

the outcome was an allowable number of ESALs of 300,800. If the pavement had already experienced 200,000 ESALs, then it would have a remaining life (structural capacity) of 100,800 ESALs or 34 percent.

This example applies to only one NDT location. To be rigorous, the evaluation should consider most (if not all) of the NDT data to come up with average structural capacity estimates for each project subsection. Furthermore, an analysis of the effect of cumulative wheel load applications on permanent deformation must also be considered to evaluate the other major load-related distress mechanism.

5. SENSITIVITY TO EVALUATION FACTORS

When conducting the project evaluation and subsequent rehabilitation design, it is important to recognize the sensitivity of the process to the factors which affect it. This type of understanding will help focus the attention on the data collection/analysis efforts which have the most impact.

Figure 2-7.3 provides a conceptual illustration of a chart that can be prepared to evaluate the sensitivity. For various percent changes from a given mid-level for each of these factors, they produce a corresponding change in the design flexible pavement structural number (for example). Those factors which produce the greatest change in design SN are the factors which should receive the most attention in the project evaluation and subsequent rehabilitation design. In this example, HMA thickness (D_{HMA}) and subgrade soil modulus (E_{SG}) have the most effect.

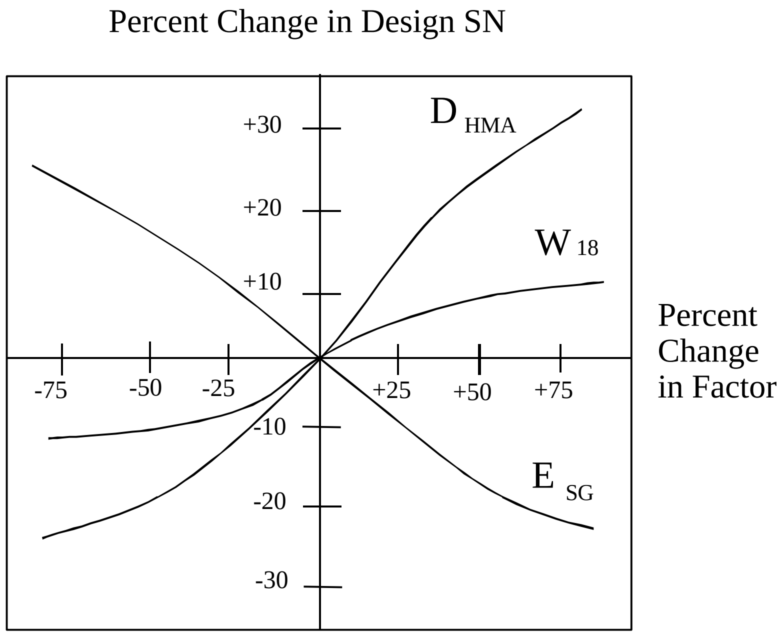


Figure 2-7.3. Conceptual illustration of the factors which have the greatest influence on project evaluation and rehabilitation design.

6. SUMMARY

This module presents guidelines and procedures on conducting an overall pavement project evaluation. A thorough pavement evaluation is absolutely essential to the development of cost-effective rehabilitation designs. Many premature failures can be attributed to a lack of understanding about the cause or extent of the deterioration.

It is believed that a systematic approach to data collection facilitates the development of appropriate project alternatives. A checklist of data deemed necessary for the design of several rehabilitation alternatives is provided. An overall pavement evaluation checklist is provided that should be completed as part of a systematic evaluation process. Using this information, the engineer can apply professional judgment in assessing the causes of pavement deterioration and developing cost-effective rehabilitation alternatives. One source of information that is not discussed heavily in this module bears emphasis. There is a lot to be learned from discussions with maintenance engineers, supervisors, and other staff in the field that have had day-to-day responsibility for the pavement under consideration. Such individuals often have a good idea of what the problems are and what has contributed to their development.

The guidelines and procedures recommended herein should be helpful to agencies in developing their own specific pavement evaluation procedures that fit their pavements and equipment. The development of standard pavement evaluation procedures and the acquisition of the required equipment and computers by State agencies is highly recommended.

7. REFERENCES

1. "AASHTO Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials, 1993.

2. O'Connor, L., "Systematic Approach in Developing Rehabilitation Strategies for Concrete Pavement," FHWA-TS-84-224, Proceedings, Workshops in Pavement Rehabilitation, Salt Lake City, Utah, September 17-20, 1984.

3. "Pavement Rehabilitation Manual, Volume I: Pavement Evaluation (1990) and Volume II: Treatment Selection (Draft—1991)," New York State Department of Transportation, 1991.

4. Brown, J.L., "Pavement Rehabilitation Methodology," Chapter XIII of Pavement Design Training, Texas State Department of Highways and Public Transportation, 1989.

5. Horton, S., "Project Level Pavement Management System Development," CDOH-DH-SD-90-7, Colorado Department of Highways, August 1990.

6. Hall, K.T., J.M. Connor, M.I. Darter, and S.H. Carpenter, "Rehabilitation of Concrete Pavements, Volume III—Concrete Pavement Evaluation and Rehabilitation System," FHWA-RD-88-073, Federal Highway Administration, July 1989.

7. Darter, M.I. and K.T. Hall, "Structural Overlay Strategies for Jointed Concrete Pavements, Volume IV—Guidelines for the Selection of Rehabilitation Alternatives," FHWA-RD-89-145, Federal Highway Administration, June 1990.

BLOCK 3

FLEXIBLE PAVEMENT REHABILITATION TECHNIQUES

This block provides detailed information on the design and construction related issues for the most widely-used flexible pavement rehabilitation techniques. These techniques all fall under the standard 3R definitions of restoration, recycling and resurfacing. They are covered in a logical sequence of minimal to maximum impact on the existing pavement.

In addressing issues related to the most common method of rehabilitation (i.e., resurfacing), it should be understood that the information is more of a general nature and that the emphasis is more on the practices that surround the overlay design and construction operations. NHI offers several other courses that are targeted at addressing specific structural design, mix design and construction related issues.

The block concludes with a module on how to identify the most appropriate (candidate) rehabilitation treatments for a given project.

MODULE 3-1

HOT-MIX ASPHALT PAVEMENT MIXTURE OVERVIEW

1. INSTRUCTIONAL OBJECTIVES

This module describes the basic properties of paving grade asphalts and the various aggregate gradations used to produce hot-mix asphalt (HMA) concrete pavement which are used for pavement construction, rehabilitation, and maintenance and introduces Superpave mixture design system.

After completing this module, the participants will be able to accomplish the following:

1. List the basic components of asphalt binder.
2. Describe what happens to the components of asphalt binder as it ages.
3. Describe the basic viscoelastic properties of asphalt binder.
4. Describe what happens to the viscoelastic properties of asphalt binder as it ages.
5. Describe the basic mix gradations that may be used in flexible pavements and their attributes.
6. Describe the possible failure modes of the various asphalt mix types.
7. Describe what Superpave is and how it is applied to the construction of flexible pavements.

2. INTRODUCTION

Asphalt binder is used extensively throughout the world to construct the wearing surface and bound bases for paved roadways. Asphalt binder is a paving material that consists of both an asphalt binder and mineral aggregate. The asphalt binder is used as a binding agent to glue the mineral aggregate particles together to build the pavement. Both the asphalt binder and the mineral aggregate have unique properties that contribute to the mechanical behavior and the ultimate performance of the asphalt pavement. This module will describe the unique properties of asphalt binder and mineral aggregate and how they are affected by construction, traffic loading, environment, and time. It will also provide a basic overview as to how these properties determine the long-term performance of the flexible pavement.

3. HMA MATERIALS

Asphalt Binder

Liquid asphalt binder is a dark brown to black cementitious material, the constituents of which are obtained as a residue in petroleum processing and is also occasionally found in nature. As a binder, asphalt is especially valuable in road building because it adheres well to most rock, is quite waterproof, and fairly durable. It also provides limited flexibility to mixtures of aggregates with which it is usually combined. Asphalt binder is also highly resistant to reaction with most acids, alkalis, and salts.

Although asphalt is a solid or semi-solid at ordinary atmospheric temperatures, it may be readily liquefied by applying heat or by dissolving it in petroleum solvents. Asphalt binder may also be emulsified in water by shearing it into very fine droplets of asphalt through the use of a colloid mill with the help of emulsifying agents.

Asphalt's adhesive and waterproofing properties have been used for some time. Naturally occurring asphalt has been found where petroleum has been forced upward by geological forces and has hardened after exposure to the elements resulting in lakes of asphalt. Natural asphalt is also found impregnated within porous rock, such as sandstone or limestone, called rock asphalt. Natural asphalts were used by the ancient Babylonians, Egyptians, Greeks and Romans as a road-building and waterproofing material.(10) Even with the long history of asphalt usage, the evolution of the material as a widespread ingredient in paving did not occur until more modern petroleum refining techniques were developed in the early 1900s and the asphalt residue by-product became more readily available.(1)

Distillation Process

Few refineries produce asphalt as their principal product. In most cases, the asphalt used to pave roads is made from the residue that remains after the refineries remove lighter distillates in the manufacturing of gasoline, diesel fuel, jet fuel, paraffin, lubricating oils, and other materials. Figure 3-1.1 illustrates the flow of crude oil through a typical refinery. The chart emphasizes that portion of the process relating to the production of asphalt.(2)

In the refining processes, crude oil is heated and delivered to an atmospheric distillation column, where the lighter fractions are vaporized and drawn off, leaving a residue of heavy oils and asphalt. This residue may be used as fuel oil, but a large proportion is processed by further distillation under vacuum, into a short residue of very soft asphalt.

A portion of this residue is sometimes treated by further distillation under vacuum or by air-blowing, which produces a fairly hard asphalt such as roofing asphalt or 40/50 pen paving asphalt. This asphalt is also used as a blending component together with softer asphalts to produce intermediate grades of asphalt.

Composition of Asphalt

Paving grade asphalts are a complex combination of hydrocarbons. In addition to hydrogen and carbon, asphalt contains small quantities of sulfur, oxygen, nitrogen, and trace quantities of metals such as vanadium, nickel, iron, magnesium, and calcium. The 90 percent to 95 percent hydrocarbon content renders asphalt susceptible to oxidative aging. In time this, together with other forms of aging, leads to changes in the chemical and molecular structure of the asphalt binder.

The chemistry of asphalt is very complex and for descriptive purposes has been divided into components derived through chemical separation techniques. The two most frequently used methods for separating out the basic fractions are the Corbett chromatograph method (ASTM D 4124 Standard Test Method for Separation of Asphalt into Four Fractions) and the Rostler precipitation method.(10) The basic components found in both procedures are shown in the following figure 3-1.2.

Though both the Corbett and Rostler components are shown in figure 3-1.2, only the basic fractional components from the Rostler method will be covered in some detail to simplify the discussion. The fractional components from the Corbett method have similar properties roughly aligned as shown in the figure 3-1.2.

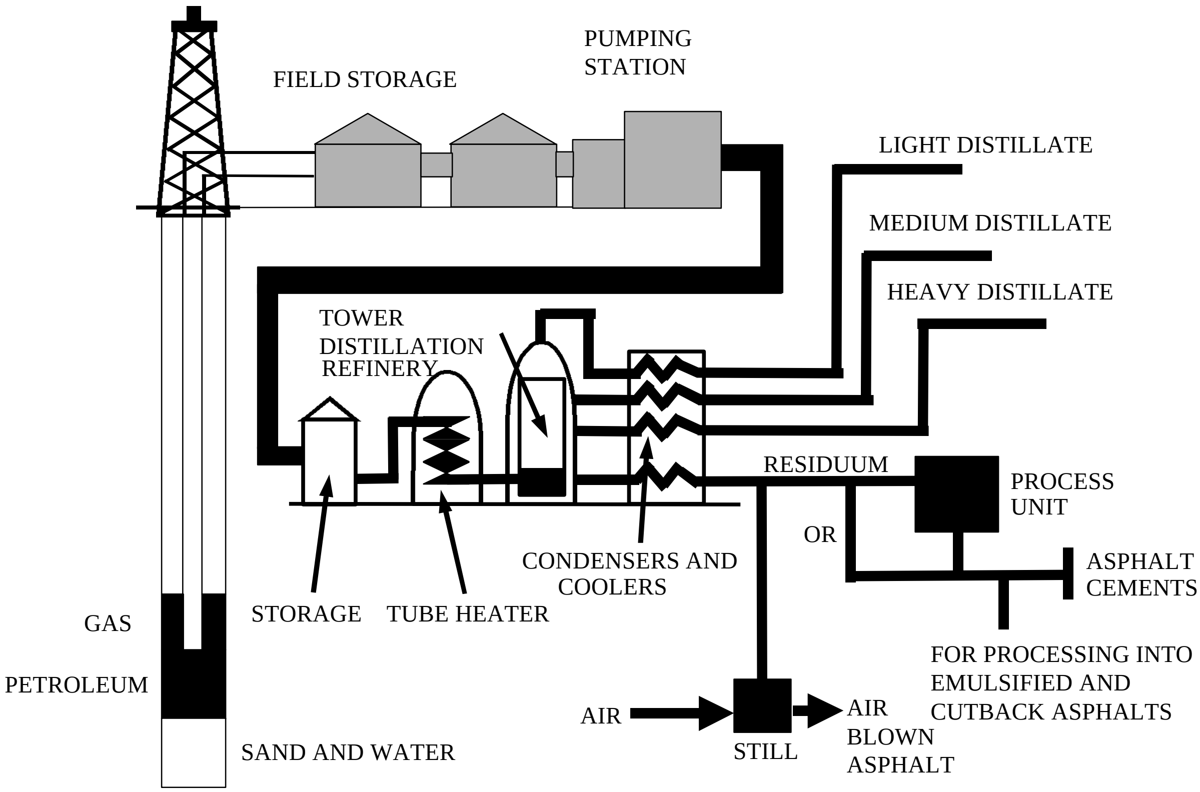


Figure 3-1.1. Petroleum asphalt flow chart for asphalt cement.[2]

Corbett	**Rostler**	Structural
(*Differential Absorption*)	(*Acid Separation*)	Characteristic
asphaltenes	**asphaltenes**	<u>saturated</u> (*structure forming*)
petrolenes	**maltenes**	
saturates	paraffins	
napthene aromatics	second acidiffins first acidiffins	slightly unsaturated <u>unsaturated</u> (*structure inhibiting*)
polar aromatics	nitrogen bases	

Figure 3-1.2. Component analysis of asphalt cement.

In the Rostler Analysis, the soluble portion of the asphalt is known as maltenes (petrolenes) and the insoluble discrete portion as asphaltenes. The maltenes are then further subdivided into paraffins, aromatics (acidifins), and nitrogen base resins.[1,10]

Asphaltenes:

- Insoluble hard, black, glassy material (highly polar fraction) constituting five to 25 percent of the asphalt. Increasing the asphaltene content makes the asphalt harder and more brittle at cold temperatures, and more viscous at high temperatures.

Maltenes:

- Saturates (paraffins) are white straw colored viscous oils containing both waxy and nonwaxy component and making up five to 20 percent of the asphalt.[10]
- Aromatics (acidiffins) are dark, brown viscous liquid making up 40 to 65 percent of the asphalt and acting as the major dispersing agent for the peptized asphaltenes.[10]
- Nitrogen base resins are dark brown solids or semi-solids that are strongly adhesive and also act as a dispersing agent or peptizer for the asphaltenes, which effects the rigidity or viscosity of the asphalt.

The molecular structure of asphalt is even more complex than the basic components imply as molecules vary in size and type of chemical bonding with each source and asphalt blend. Relatively weak chemical bonds hold the molecules together and these can be destroyed by either heat or shear stress, which gives asphalt its viscoelastic characteristics. No definite correlations have been identified between the various chemical component quantities and asphalt performance.

In addition the molecules in the asphalt can also be divided into two functional categories of polar and non-polar molecules.

- Polar molecules form the network of the asphalt and provide the elastic properties.
- Non-polar molecules provide the body or filler of the asphalt and its viscous properties.

These two categories of molecules make up the various components already discussed, forming a somewhat a homogenous mixture. Their weak interaction results in the more fluid behavior of asphalt at high temperatures where the viscosity change is proportional to the temperature change.

The balance between polar and non-polar molecules is important to ensure good performance. Asphalt with large quantities of high molecular weight polar molecules tend to exhibit poor low temperature behavior and brittleness. The complex and variable chemical and molecular structure of asphalt makes it extremely difficult to use chemical analysis to characterize performance.

Asphalt Viscoelastic Behavior

The viscoelastic character of asphalt results in varied behavior as loading and temperature change. At higher temperatures, there is more flow or plastic behavior, while at low temperatures and short duration loading the binder tends to be stiff and elastic. At intermediate temperatures, it tends to act as a combination of the two.

The affect of time and temperature on asphalt cement are related; the behavior at high temperature over short time periods is equivalent to what occurs at lower temperatures and long duration. (A slowly applied load or a load applied over a long time period produces the same viscous deformation in asphalt at cold temperatures as a more quickly applied load at warm temperatures.) An example of this is where slow moving truck traffic on a steep grade cause more rutting than the same truck traffic on a relatively flat grade. This effect is also seen at intersections where the pavement is much more rutted particularly at traffic lights where traffic slows or stops This is often referred to as the time-temperature shift or superposition concept of asphalt cement. The following discussions on the high, low, and intermediate temperature behavior of asphalt provides a little more detailed description of these properties.

<u>High Temperature Behavior</u>

In hot conditions or under slowly applied loads (e.g., slow moving or parked trucks), asphalt cement acts like a viscous liquid. Thus, under these circumstances, the aggregate is the part of the HMA that bears the load much more than the asphalt. Viscosity is the physical material characteristic used to describe the resistance of liquids to flow. If the individual molecules could be observed in hot asphalt cement as it flows, adjacent layers of molecules would be observed sliding past each other. The resisting force or friction between these layers of molecules is related to the relative velocity at which they slide by each other. As one molecule tries to pull the closest molecule along with it as it moves, the next closest molecule tries to hold the second molecule back. The relationship between the resisting force and relative velocity in the flow of liquid asphalt is the basis of asphalt viscosity, which has been used to specify various grades of asphalt.

Liquid asphalt exhibits plastic behavior because once it starts flowing, it does not return to its original position. This is why in hot weather, some less stable HMA pavements flow under repeated wheel loads and wheel path ruts form from an accumulation of plastic deformation. Rutting in asphalt pavements during hot weather is also influenced by aggregate properties and thus, it is probably more correct to say that the less stable asphalt mixture is experiencing plastic deformation.

<u>Low Temperature Behavior</u>

In both cold weather and under rapidly-applied loads (e.g., fast-moving trucks), asphalt cement behaves like an elastic solid. Elastic solids are like rubber bands; when a load is applied they deform, and when that load is removed, they return to their original shape.

Also, elastic solids have an ultimate strength and when they are loaded beyond this level, the elastic solids may break. Since asphalt cement is an elastic solid at low temperatures, it will crack when excessively loaded. For this reason, low temperature cracking can occur in asphalt pavement during cold weather. Low temperature cracking occurs when a thermal load is applied by internal tensile stresses that build up in the asphalt pavement when it shrinks while being restrained by the lower pavement layers and/or base material.

<u>Intermediate Temperature Behavior</u>

Most pavements are built in environments that experience moderate or more intermediate temperatures most of the time and experience very hot and cold temperatures for relatively short periods each year. At these intermediate temperatures, asphalt binders exhibit the characteristics of both viscous liquids and elastic solids. Because of this range of behavior, asphalt is an extremely complicated material to understand and explain.

Aging Behavior

Asphalt cements are composed of organic molecules and react with oxygen from the environment. This reaction is called oxidation and it changes the structure and composition of asphalt molecules. In asphalt, the maltenes oxidize to form asphaltenes and other products that causes the asphalt cement to become more brittle, which is referred to as oxidative hardening or age hardening. Oxidative hardening usually happens at a relatively slow rate in a pavement; however, the rate increases somewhat in warmer climates and during warmer seasons. [Because of this hardening, old asphalt pavements are more susceptible to cracking. As asphalt cement ages and oxidizes, it becomes stiffer and the temperature at which it acts like an elastic solid increases with age and oxidation which contributes to fatigue cracking and temperature cracking.] Improperly compacted asphalt pavements with higher air void contents will experience accelerated oxidative hardening. Inadequate compaction leaves a higher percentage of interconnected air voids, which allows more air to penetrate into the asphalt mixture, leading to increased oxidation and hardening of the asphalt.

In actual construction, a considerable amount of oxidative hardening occurs before the asphalt is placed. At the hot-mixing facility, asphalt cement is added to the hot aggregate and the mixture is maintained at elevated temperatures for a period of time. Because the asphalt cement exists in thin films covering the aggregate, the oxidation reaction occurs at a much faster rate.

Other forms of hardening include volatilization, temperature, and steric hardening. Volatilization occurs during hot-mixing and construction when volatile components tend to evaporate from the asphalt. Physical hardening occurs when asphalt cements have been exposed to low temperatures for long periods. When the temperature stabilizes at a constant low value, the asphalt cement continues to shrink and harden. Temperature or physical hardening is more pronounced at temperatures less than 0EC. This is due to a transition in the structure of the asphalt molecules where the asphalt becomes much stiffer with decreasing temperatures and acts more like an elastic solid. The temperature at which an asphalt cement becomes much stiffer with decreasing temperature is called the glass transition temperature.[4,5] As the asphalt cement in an asphalt concrete pavement ages and oxidizes, the glass transition temperature increases, usually from below 0℃ to above 0℃. This form of asphalt hardening contributes to both fatigue cracking and temperature cracking. Steric hardening produces thixotropic effects.[9] In other words, the asphalt cement tends to become stiffer and more viscous when it is not exposed to stress or loads. Asphalt concrete pavement placed as bike or walk paths, and as traffic lanes that do not have any traffic placed on them for several years for project staging reasons, often experience early brittle cracking. This is largely due to steric hardening and the thixotropic effect.

Modified Asphalts

When asphalt pavements are subjected to more severe environmental temperature extremes, it is now possible to improve the temperature viscosity properties of the asphalt cement.

The temperature viscosity properties of the asphalt cement can be improved by the addition of polymers, consequently the use of polymer-modified asphalt is increasing. Polymers can be broadly classified into elastomers for improving the elastic properties of asphalt cement and plastomers for improving the stiffness of the asphalt. With the use of these modifiers, a wide range of binder properties can be improved, the most noticeable being:[1]

- Temperature susceptibility.
- Adhesion to aggregates.
- Resistance to permanent deformation.
- Resistance to fatigue cracking.
- Ductility.
- Elasticity.
- Durability.

The basic types of elastomers that have been used to modify asphalts are:[7]

- Synthetic Rubber [Styrene-Butadiene-(SB) and Styrene-Butadiene Rubber (SBR)], which is also available in an emulsified form and referred to as "latex".
- Thermoplastic Rubber [Styrene-Butadiene-Styrene (SBS)].

The basic types of plastomers that have been used are:[7]

- Low Density Polyethylene (LDPE).
- Ethylene-Vinyl-Acetate (EVA).

The following table 3-1.1 lists typical polymers that are in use today to modify asphalt.[6]

Polymers have a fairly long hydrocarbon chain structure compared to asphalt thus, the addition of polymers usually increases the stiffness or viscosity of asphalt cement at high temperatures. The addition of small amounts of polymer in the range of one to two percent usually provides general reinforcement and stiffening of the asphalt cement. Somewhat larger amounts of polymer in the range of three to five percent may form a network structure.[8,9] The appropriate choice of asphalt, asphalt grade, polymer type polymer concentration, and method of mixing will determine if a network-like structure is formed.

The addition of polymers to asphalt cement primarily improves the high temperature properties of asphalt and has only limited effect on the low temperature properties of the asphalt. The low temperature properties of modified asphalt are largely determined by the grade of the base asphalt. By modifying lower viscosity asphalts (i.e., low asphalt cement grades) with the appropriate polymer, asphalt binders can be made that provide significantly lower moduli at low temperatures, while also providing higher moduli at elevated temperatures. Thus, a binder's thermal characteristics can be optimized over the full range of its operating temperature by blending the appropriate polymer with the appropriate grade and type of asphalt.[9]

Asphalt Rubber

In addition to the polymers noted above, crumb rubber is also blended with asphalt cement to make asphalt rubber. Crumb rubber is obtained from scrap tires and ground to a specified maximum particle size which may range from less than 1 mm to 5 mm, depending upon the supplier and the processes being used. One of the more common processes uses about 20 percent of rubber crumb blended with low viscosity asphalt at a mixing temperature of 170℃ to 210℃ for a period of one to four hours to allow for reaction of the rubber. During this time, some of the aromatic oils in the asphalt are absorbed by the rubber particles. The blending period and mixing temperature require careful control since they have a large effect on the quality and performance of the modified asphalt. The modified binder is known as "asphalt rubber." It is much more viscous than unmodified asphalt and is not a homogeneous binder, so it therefore requires special equipment for pumping and spraying in both asphalt plants and asphalt distributors. Asphalt rubber has been one of the more widely-used forms of modified binder and has also been used extensively in surface treatments, and in open-graded and gap-graded HMA concrete pavements.[1] Asphalt rubber has also been used in dense-graded mixes to a more limited extent. Like polymer-modified asphalt, asphalt rubber provides increased stiffness at higher temperatures and it is usually blended with a lower viscosity asphalt to provide improved low temperature characteristics.

Table 3-1.1 Typical polymers used to modify asphalt.[6]

Trade Name	Manufacturer	Polymer		
		Type	Form	Chemistry
Elastomers				
Butonal	BASF	Block Copolymer	Latex	Styrene-Butadiene (SB)
Downright	Dow	Random Copolymers	Latex	Styrene-Butadiene-Rubber (SBR)
Europrene	EniChem Elastomers Americas, Inc.	Block Copolymer	Crumb or Powder	Styrene-Butadiene-Styrene (SBS)
Finaprene	Fina	Block Copolymer	Crumb	Styrene-Butadiene (SB)
Kraton	Shell	Block Sopolymer	Crumb or Powder	Styrene-Butadiene-Styrene (SBS)
Neoprene	DuPont	Homopolymer	Latex	Poly Chloroprene
Polysar	Polysar, Inc.	Random Copolymers	Latex	Styrene-Butadiene-Rubber (SBR)
Styrelf	Elf Aquataine	Block Copolymer	Pre-Blended	Styrene-Butadiene (SB)
Ultrapave	Goodyear	Random Copolymer	Latex	Styrene-Butadiene-Rubber (SBR)
Vector	Dexco	Block Copolymer	Crumb or Powder	Styrene-Butadiene-Styrene (SBS)
Plastomers				
Elvax	DuPont	Copolymer	Crumb or Powder	Ethylene Vinyl Acetate (EVA)
Novophalt	Advanced Asphalt Technologies	Homopolymer	Pre-blended with AC	Low Density Polyethylene (LDPE)
Polybilt 100 series	Exxon	Copolymer	Crumb or Powder	Ethylene Vinyl Acetate (EVA)
Polybilt 500 series	Exxon	Copolymer	Crumb or Powder	Ethylene Methylacrylate (EMA)

4. NEW SUPERPAVE BINDER SPECIFICATIONS

Current Asphalt Property Measurements

Due to asphalts chemical complexities, most existing specifications have been developed around physical property tests. All currently used specifications use penetration, viscosity, and ductility tests. These physical property tests are performed at standard test temperatures and the test results are used to determine if the material meets the specification criteria.

However, there are limitations in what the results of the current test procedures provide. Many of the current tests are empirical, meaning that pavement performance experience is required before the test results yield meaningful information. Penetration is an example of this. The penetration test indicates the stiffness of the asphalt, but any relationship between asphalt penetration and performance has to be gained by field experience. An additional concern is that the actual performance of asphalt cement can not be directly predicted from the test results.

Another limitation of the current tests and specifications is that the tests do not give information on the characteristics of asphalt cement over the entire range of typical pavement temperatures that may be experienced. Although viscosity is a fundamental measure of flow, it only provides information about viscous behavior at higher temperatures - the standard test temperatures are 60EC and 135EC. The standard penetration test describes only the consistency at laboratory room temperature (25EC). Lower temperature elastic responses can not realistically be determined from this data to predict low temperature performance.

Superpave Asphalt Binder Specification

In 1987, the Strategic Highway Research Program (SHRP) began a $50 million research effort to develop new procedures for defining and measuring the physical properties of asphalt along with the concurrent development of new HMA mix design procedures. A major result of the binder related research effort is the Superpave binder specification which utilizes new or modified test equipment and procedures to determine and define binder properties. A unique feature of the Superpave specification is that the specified performance criteria remain constant, but the temperature at which the criteria must be achieved changes for the various grades. As discussed, existing specifications set up a range of viscosities and penetration values at set temperatures which represented laboratory, field, and plant mixing temperatures.

The Superpave tests measure physical properties that can be related directly to field performance by engineering principles. The Superpave binder tests are also conducted at temperatures that are encountered by in-service pavements to provide better binder performance for specific environmental regions. The new test procedures are noted in figure 3-1.3, along with some indication of binder performance predicted.[3] However, these test procedures are used to establish the mix design. Ultimately the pavement structure, mixture design, and of paramount importance, the as-constructed mix properties, along with binder properties, will ultimately determine the performance of the pavement over its service life.

The central theme of the Superpave binder specification is its reliance on testing asphalt binders in conditions that simulate the three critical stages during the binder's life. Tests performed on the original asphalt represent the first stage of transport, storage, and handling. The second stage represents the asphalt during mix production and construction and is simulated for the specification by aging the binder in a rolling thin film oven. This procedure exposes thin binder films to heat and air and approximates the aging of the asphalt during mixing and construction. The third stage occurs as the binder ages over a long period as part of the HMA layer. This stage is simulated for the specification by the pressure aging vessel. This procedure exposes binder samples to heat and pressure in order to simulate years of in-service aging in a pavement.

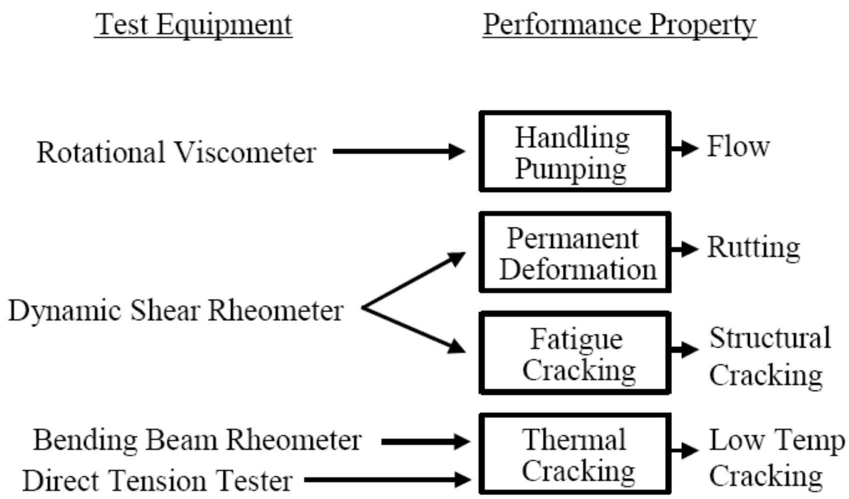


Figure 3-1.3. Superpave laboratory tests - relation to performance.[3]

Pavement Performance

The Superpave binder specification is intended to improve performance by limiting the potential for the binder to contribute toward permanent deformation, low temperature cracking, and fatigue cracking in asphalt pavements. The specification provides for this improvement by designating various physical properties which are to be measured with the new equipment.

One important distinction between currently used asphalt specifications and the Superpave specification is the overall format of the requirements. The required physical properties remain constant for all of the performance grades (PG). However, the temperatures at which these properties must be reached vary depending on the climate in which the binder is expected to be used. For example, the partial view of the Superpave specification shown in figure 3-1.4 shows that a PG 52-40 grade binder is designed to sustain the conditions of an environment where the average seven-day maximum pavement temperature is 52 ° C and the minimum pavement design temperature is -40 ° C.

Permanent Deformation

As discussed earlier, the total response of asphalt binders to load consists of two components: elastic (recoverable) and viscous (non-recoverable). Pavement rutting or permanent deformation is the accumulation of the non-recoverable component of the responses to load repetitions at high service temperatures. The Superpave specification defines and places requirements on a rutting factor, $G^*/\sin\delta$ which represents a measure of the high temperature stiffness or rutting resistance of the binder.

Performance Grade	PG-52							PG-58		
	-10	-16	-22	-28	-34	-40	-46	-16	-22	-28
Average 7-day Maximum Pavement Design Temp. C	< 52							< 58		
Minimum Pavement Design Temp. C	>10	>16	>22	>28	>34	>40	>46	>16	>22	>28
	Original Binder									
Flash Point Temp. T48 Minimum, C	230									
Viscosity, ASTM D 4402: [b] Maximum, 3 Pa-s (3000cP) Test Temp. C	135									
Dynamic Shear, TP5: [c] $G^*/\sin\delta$, Minimum, 1.00 kPa Test Temp. @ 10 rad/sec, C	52							58		

Spec Requirement Remains Constant

Test Temperature Changes

Figure 3-1.4. Superpave binder specification format.[3]

Values for the complex shear modulus which is noted as G* (G star) and the phase angle which is noted as δ (delta) for an binder are determined from the dynamic shear rheometer. G* is a measurement of the total resistance of an binder to deformation when exposed to repeated pulses of shear stress. It consists of both elastic (recoverable) and viscous (non-recoverable) strain components. δ is an indicator of the relative amounts of recoverable and non-recoverable deformation or strain.

High values of G* and low values of δ are considered desirable attributes from the standpoint of rutting resistance. More simply stated, the Superpave specification promotes the use of stiff elastic binders to address rutting resistance.

Excessive Aging From Volatilization

A mass loss requirement is also specified to guard against using an asphalt binder that would age excessively from volatilization of the non-polar maltene fraction during hot-mixing and construction.

Fatigue Cracking

Like permanent deformation, G* and δ are also used in the Superpave binder specification to help control the fatigue of asphalt pavements. Since fatigue generally occurs at low to moderate pavement temperatures after the pavement has been in service for a period of time, the specification addresses these properties using binder aged in both the rolling thin film oven (RTFO) and the pressure aging vessel (PAV).

The dynamic shear rheometer is again used to generate G* and δ. However, instead of dividing the two parameters, the two are multiplied to produce a factor related to fatigue. The fatigue cracking factor is G*sinδ, which is stated "G star sine delta." It is the product of the complex shear modulus, G*, and the sine of the phase angle, δ. The Superpave binder specification has a maximum value of 5000 kPa for G*sinδ. Low values of G* and δ are considered desirable attributes from the standpoint of resistance to fatigue cracking. Thus, the Superpave specification promotes the use of compliant and elastic binders (PAV aged) to address fatigue cracking.

Low Temperature Cracking

When the pavement temperature decreases, asphalt binder shrinks. Since friction against the lower pavement layers inhibits movement, tensile stresses build up in the pavement. When these stresses exceed the tensile strength of the asphalt mix, a low temperature crack occurs. The bending beam rheometer is used to apply a small creep load to a binder beam specimen and measure the creep stiffness (the binder's resistance to the load). If the creep stiffness is too high, the asphalt will behave in a brittle manner and cracking is more likely to occur. To prevent this cracking, creep stiffness has a maximum limit of 300 Mpa. Since low temperature cracking usually occurs after the pavement has been in service for some time, this part of the specification addresses these properties using binder aged in both the RTFO and PAV.

Studies have demonstrated that if the binder can stretch at least one percent of its original length during the very cold contraction period, cracks are less likely to occur. As a result, a direct tension is also included in the Superpave specification as an alternative requirements. The direct tensile (DTT) is used to pull an binder sample in tension at a very slow rate, simulating the pavement condition as shrinkage occurs. The amount of strain that occurs before the sample breaks is the failure or peak strain and must be a minimum of 1.0 percent.

Pumping and Handling

To ensure that asphalt binders, especially modified asphalts, can be pumped and handled at the hot-mixing facility, the specification contains a maximum viscosity requirement on the unaged binder. This value is 3 Pa and must be achieved at 135°C for all grades.

5. MINERAL AGGREGATE

Aggregate Properties

When we think of hot-mixed asphalt concrete pavements, we tend to think more in terms of the asphalt binder than of the mineral aggregate, which makes up very close to 95 percent of the mixture by mass (weight). Though the mineral aggregate is not as complex a part of the HMA pavement, it is the basic skeleton which provides the primary load distribution properties of the pavement. For this reason, mineral aggregate must have some basic properties for size, shape, grading, strength, toughness, and durability. The hardest and most durable rock or gravel that can be found in a region is usually selected for the production of mineral aggregate. In certain areas where there are limited sources or no sources of reasonably hard durable aggregate, this requires shipping in rock or gravel from sources outside the region. In some areas, rock or gravel is shipped very long distances to provide adequately hard and durable material for the aggregates. Most sources of mineral aggregate must pass fairly clear toughness and durability standards so these basic properties are reasonably well standardized and do not now seem to have a significant effect on pavement performance as long as the standards are met.

One of the most variable properties of mineral aggregate is that of aggregate grading. Aggregate gradation is the distribution of particle sizes expressed as a percent of the total mass (weight). Gradation is determined by passing the material through a series of sieves stacked with progressively smaller openings and then weighing the material retained on each sieve. Grading of the mineral aggregate is still one of the more difficult properties to control in the construction of HMA pavements and, consequently, has considerable effect on the ultimate performance of the flexible pavements. There are also several basic gradings that are used to produce distinctly different HMA pavements mixes such as open-graded friction cores (OGFC) and stone mastic asphalt pavements (SMA). In this class, we will not cover the construction quality control aspect of mineral aggregate grading, but review the basic grading that produces the different types of mixes which have different uses and performance aspects. Specifically, we will review the basic attributes of densely-graded, open-graded, and gap-graded aggregate gradations.

Dense-Graded Aggregate and HMA Mixes

Almost all of the HMA pavement that has been constructed in the past, and still a very large proportion of the HMA pavement placed today, is classified as dense-graded mix. A dense-graded aggregate is most commonly used to produce HMA pavement because the dense-grading produces the most stable load distributing skeleton with limited void space to optimize the required asphalt binder and minimize the remaining interconnected air voids. For an aggregate to be dense-graded, its gradation should be uniformly distributed through the full range of sieve sizes. Grading charts are very useful to help visualize the maximum density grading and to develop or adjust aggregate gradation in mix design and construction. The more commonly used and often recommended chart is the Federal Highway Administration (FHWA) 0.45 Power Gradation Chart, that is based on Fuller’s maximum density equation.

$$P=100(d/D)^n$$

Where;

P = total percentage passing given sieve size,

d = sieve size considered,

D = maximum sieve size in gradation,

n = parameter where values of 0.45 to 0.50 produce maximum particle density.

The chart which is shown in figure 3-1.5 is plotted on a log scale of 0.45 and thus, following Fuller’s maximum density equation, any straight line that is drawn on the chart from the origin at the lower left of the chart to the maximum aggregate size on the top will produce the maximum density grading.[11]

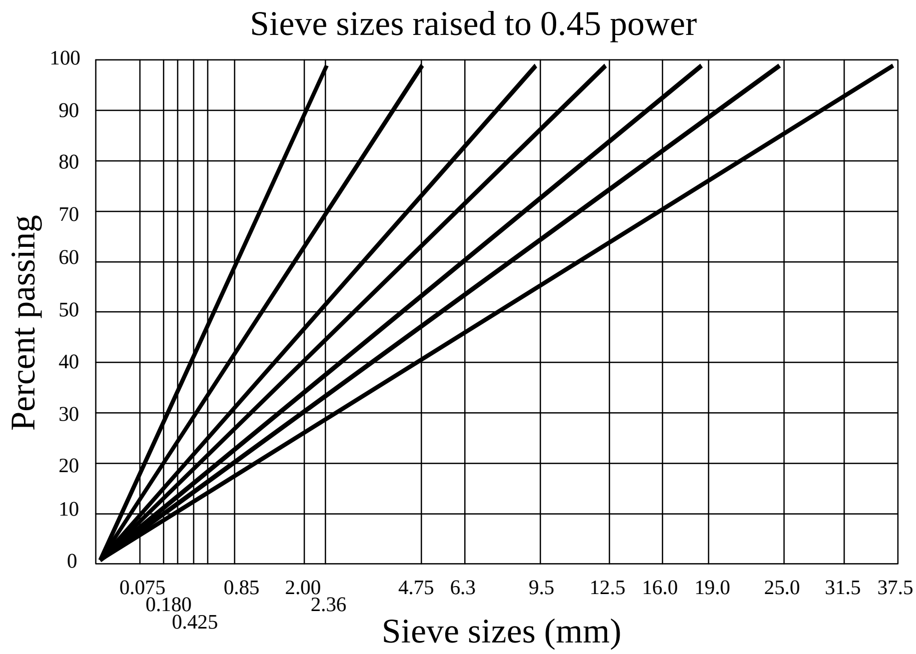


Figure 3-1.5. Maximum density curves (FHWA 0.45 Power Gradation Chart).

Gradations that too closely approach the straight lines show on the 0.45 power chart do not have enough voids in the mineral aggregate (VMA) to allow space for the asphalt and still have sufficient voids (3 to 5 percent) to allow for postconstruction compaction. Most gradings are adjusted away from the straight lines within acceptable limits to provide a specific range in the VMA and still make use of the maximum density provided by following the straight lines on the chart. A typical grading specification band is shown in figure 3-1.6 for a 16.0 mm maximum HMA mix.

As indicated, dense-graded mixes make up the bulk of the HMA pavement used around the world. They provide an economical and easily produced mix that provides a balance between fatigue cracking resistance and rutting resistance. They depend largely upon the stable nature of the dense-grading aggregate to provide stability to the HMA pavement. The asphalt films that coat the aggregate are relatively thin. The amount of binder used in the mix is based on the somewhat conflicting demands of having enough binder to resist early raveling and fatigue cracking, but not too much binder to cause early rutting and shoving. All asphalt pavements will ultimately fail by cracking, rutting, or loss of surface aggregates. A good pavement is judged by the service life it provides before these deficiencies become unacceptable from the engineering or user's standpoint. Early fatigue cracking or rutting in dense-graded mixes is most often traced back to a less than optimum combination of aggregate grading, asphalt content, and resulting low or high void space as constructed.

Superpave mix designs are being developed to help improve the performance of dense-graded hot asphalt concrete pavements by improving the mix design procedures. The Superpave mixture design processes are being developed to provide mixture designs at three specific levels with increased rigor and complexity required at each level. Level one offers an improved material selection and volumetric

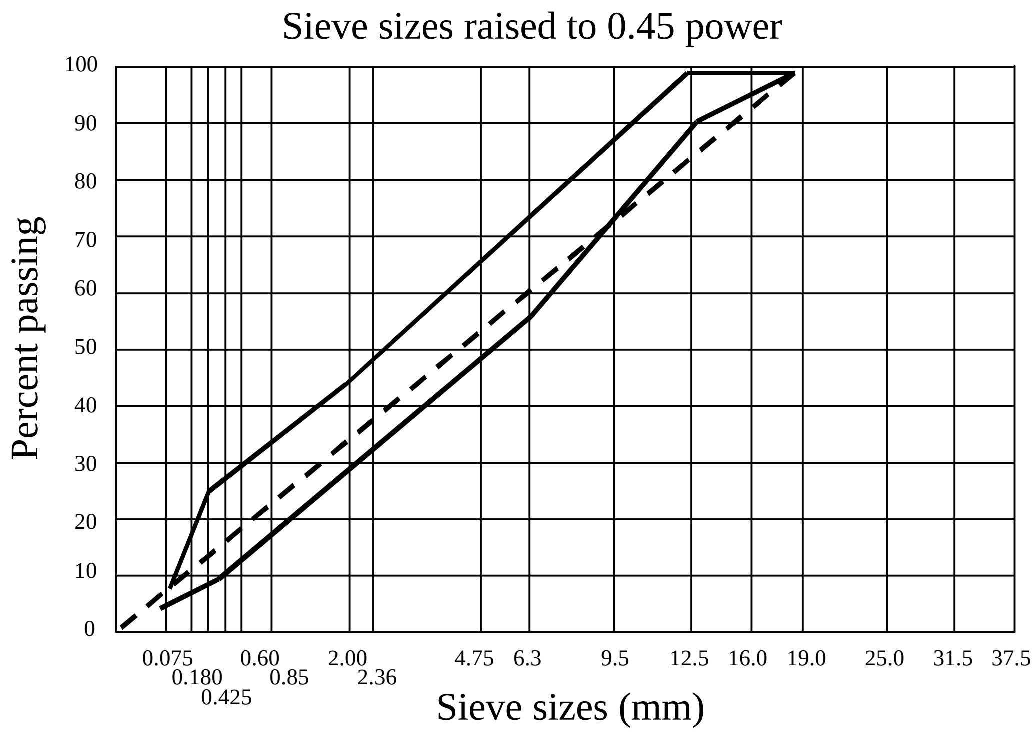


Figure 3-1.6 Typical grading bands for a 16.0 mm maximum ACP mix.[12]

mix design processes intended for roads with less than one million design ESALs. Level two mix design procedures use the volumetric mix design as a starting point and also includes a battery of tests to predict the likely performance of the mixture for roads with design seals between 1 million and 10 million. Level 3 mixture design includes a more comprehensive array of tests and results to produce more reliable performance prediction for roads with over 10 million design ESALs. At the present time, quite a few States are trying the level 1 mix design processes. Level 2 and level 3 mix design processes are still under development.

Open-Graded Aggregate and HMA Mixes

Open-graded HMA mixes have evolved from early attempts to construct a pavement with the attributes of chip seals but with normal hot-mix procedures. When compared to normal dense-graded mixes, they have demonstrated certain advantages. The following is a list of advantages listed in the Federal Highway Administration's Technical Advisory on Open-Graded Friction Courses.[13]

1. Provides and maintains good high speed, friction qualities (the frictional characteristics are relatively constant over normal operating speeds);
2. Reduces the potential for hydroplaning;
3. Reduces the amount of splash and spray; are generally quieter,
4. Often provides a 3 to 5 decibel reduction in tire noise;

5. Improves the wet weather, night visibility of painted pavement markings; and
6. Conserves high quality, polish resistant aggregates, which may be scarce in some areas, because they are placed only as a surface layer, up to 19 mm.

Open-graded mixes are made using an open-graded mineral aggregate grading. The open-graded gradation for the FHWA friction course results in most of the aggregate in the mix consisting of 3 to 10mm aggregate. This grading results in large void space that allows water to flow through the HMA mix. A typical example of an Open-Graded Friction Course is shown in figure 3-1.7 plotted on a 0.45 power chart.(12) The farther the grading moves away from a straight line plotted through the origin, the more open voids space there is in the resulting gradation.

The open-graded mix is not as stable as the dense-graded mix and it has a tendency to ravel easily. Thus, the open-graded mixes require much thicker asphalt films to provide reasonable service lives. Heavier asphalt films are achieved by:

- Using 2 to 5 percent minus 200 fines to help bulk the films,
- Mixing and placing at somewhat cooler temperatures than normal which increase the viscosity of the asphalt, and
- Using polymer-modified asphalt that has increased viscosity at higher temperatures.

In open-graded mixes, the thick asphalt films are more exposed to air and water because of the high voids in the mix, consequently the asphalt films still oxidize quickly and service life is limited to 7 to 10 years.(13) Some States have used fog seals to increase the film thickness and retard oxidation of the asphalt film which has extended the service life of the pavement, but this also results in decreased void space and water handling properties of the mix. In time, the open-graded mix does fill with dirt and other debris which also diminishes the utility of the mix to carry water. Some European countries have developed large vacuum cleaners to clean the debris out of the open-graded mixes.(14)

There has been expanding experience in Europe(14) and in a few Western States(15) with the use of thicker layers of open-graded mixes (38 to 51mm), using an open-graded aggregate with a larger maximum size (13 to 16mm) which has provided improved overall performance. The basic design issues that apply to the finer mixes also apply to the coarser mixes as well. The coarser mixes provide improved rut and wear characteristics, higher possible void contents, and can be placed at thicker more structurally enhancing thicknesses.

Gap-Graded Aggregate and HMA Mixes

Gap-graded aggregate gradations are like open-graded gradations, except they are not as open. Instead of being graded to produce high void space as in the case of open-graded mixes, they are used to enhance large aggregate contact by producing a large stone skeleton with the resulting void space filled with much finer aggregates. The resulting gradation has a “gap” in the medium to fine aggregate size.

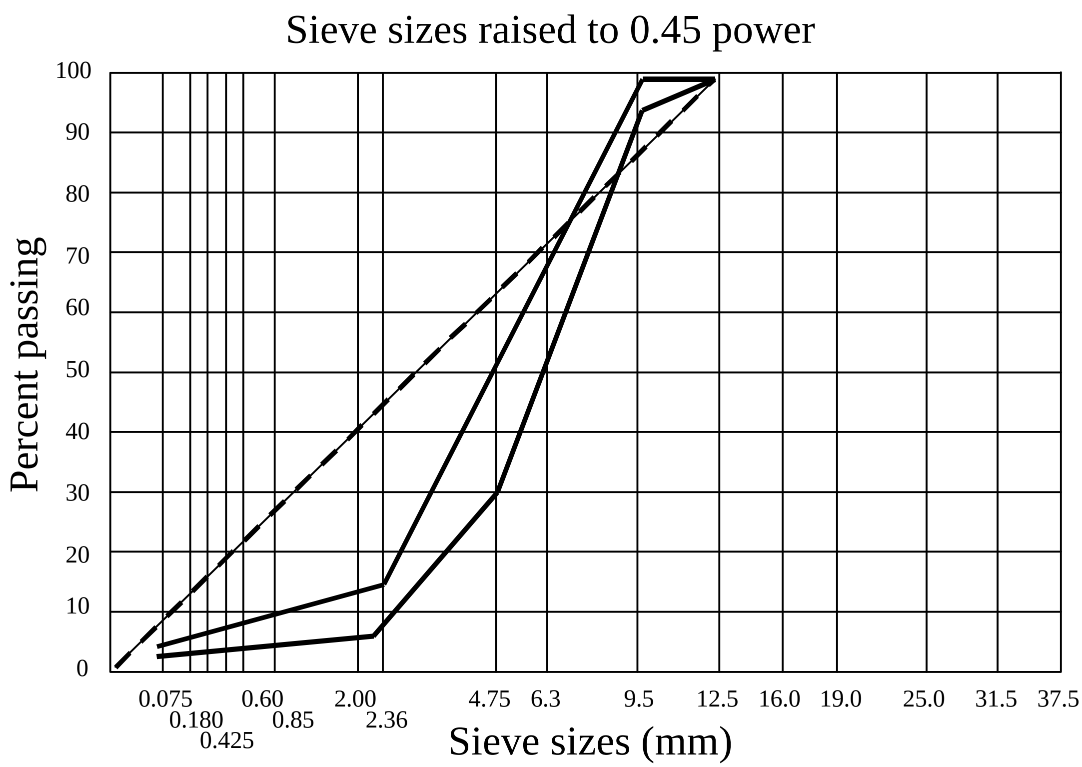


Figure 3-1.7. Typical open-graded mix plotted on a 0.45 power chart.[12]

One of the more unique and promising mixes is the stone matrix asphalt mix (SMA) (also referred to as split mastic asphalt) that has been used for some time in Europe and is gaining in use in North America. SMAs are a type of gap-graded mix which are designed to have a higher percentage of coarse aggregate in the mixture, compared to dense-graded mixes which provide greater large stone on stone contact. SMA mixes with large stone on stone contact have been found to produce a more fatigue, rut, and wear resistant mix, compared to both dense-graded and open-graded mixes.[16,17] SMAs have been shown to provide increased service life, compared to typical dense-graded mixes. SMA mixes typically have a high proportion of high quality coarse-crushed aggregate, a high proportion of asphalt binder and mineral filler, a low proportion of middle-sized aggregate, and a stabilizing additive to reduce drainage of the binder from the mix before the mix is placed. A typical aggregate gradation for a SMA is shown on the gradation chart in figure 3-1.8.

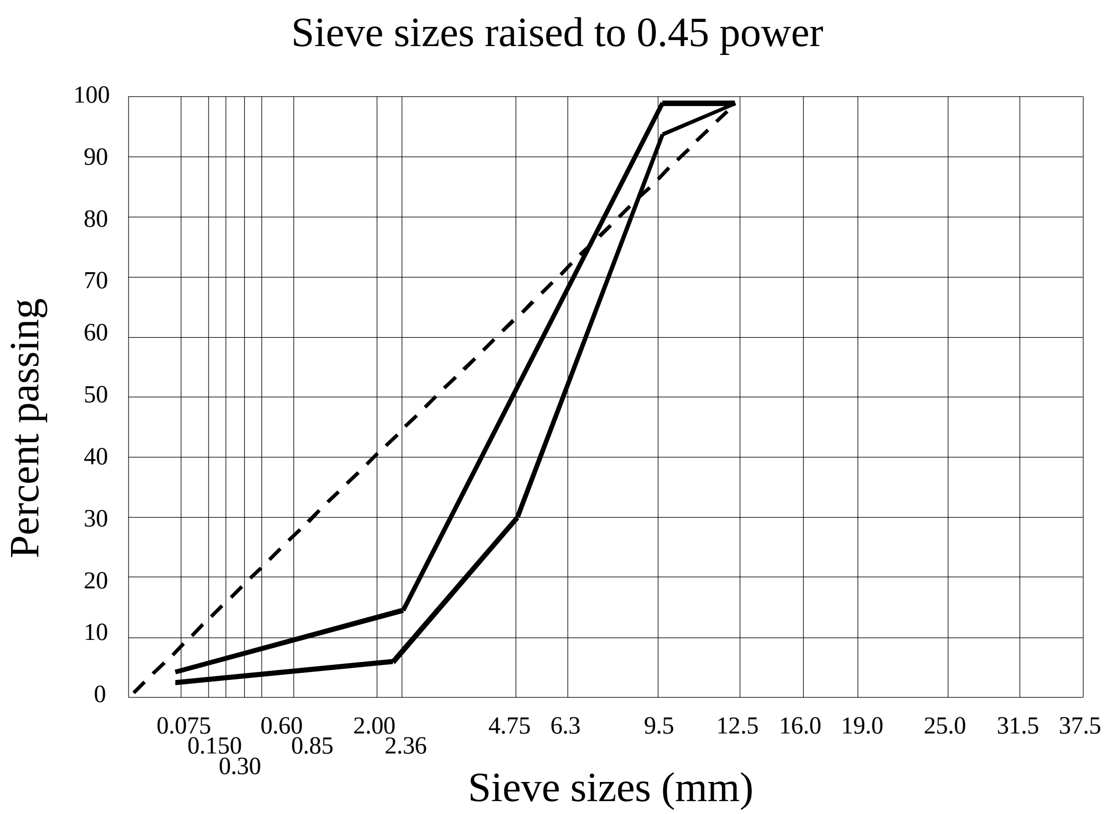


Figure 3-1.8. Proposed gradation for 13 mm SMA mixture.[16]

Open-graded and gap-graded HMA pavement mixes are more difficult to construct properly and their resulting performance can be more negatively affected by construction errors. For example, overheating an open-graded mix will cause the binder to drain down and leave a too thin film on the aggregate. This has resulted in raveling of a new pavement to a severity, in the worst case, that patching was required in less than 6 months. Thus, both open-graded and gap-graded mixes require an increased level of quality control during construction. Because of the more unique grading and mixing requirements, as well as increased quality control, both open-graded friction courses and SMA mixes cost more than normal dense-graded mixes. SMA mixes may cost as much as 40 percent more than normal dense-graded mixes. European experience indicates that SMA mixes can provide a 30 to 40 percent increase in service life over normal dense-graded mixes.[17]

5. SUMMARY

This module describes the basic properties of paving grade asphalts and the various aggregate gradations used to produce HMA concrete pavement which are used for pavement construction, rehabilitation, and maintenance; and introduces the new Superpave binder specifications and mix design processes.

Paving grade asphalts are a complex combination of hydrocarbons. In addition to hydrogen and carbon, asphalt contains small quantities of sulfur, oxygen, nitrogen, and trace quantities of metals, such as vanadium, nickel, iron, magnesium, and calcium. The 90 to 95 percent hydrocarbon content renders asphalt susceptible to oxidative aging. In time this, together with other forms of aging, leads to changes in the chemical and molecular structure of the asphalt.

The viscoelastic character of asphalt results in varied behavior as loading and temperature change. At higher temperatures, there is more flow or plastic behavior, while at low temperatures and short duration loading, the binder tends to be stiff and elastic. At intermediate temperatures, it tends to act as a combination of the two.

The effects of time and temperature on asphalt are related; the behavior at high temperature over short time periods is equivalent to what occurs at lower temperatures and long duration. (A slowly applied load or a load applied over a long time period produces the same viscous deformation in asphalt at cold temperatures as a more quickly applied load at warm temperatures.) An example of this is where slow moving truck traffic on a steep grade cause more rutting than the same truck traffic on a relatively flat grade. This effect is also seen at intersections where the pavement is much more rutted, particularly at traffic lights where traffic slows or stops. This is often referred to as the time-temperature shift or superposition concept of asphalt. The following discussions on the high, low, and intermediate temperature behavior of asphalt provides a little more detailed description of these properties.

Because asphalt are composed of organic molecules, they react with oxygen from the environment. This reaction is called oxidation and it changes the structure and composition of asphalt molecules. In asphalt, the maltenes oxidize to form asphaltenes and other products which causes the asphalt to become more brittle, which is referred to as oxidative hardening or age hardening. Oxidative hardening usually happens at a relatively slow rate in a pavement, however, and the rate increases somewhat in warmer climates and during warmer seasons. Because of this hardening, old asphalt pavements are more susceptible to cracking. As asphalt ages and oxidizes, it becomes stiffer and the temperature at which it acts like an elastic solid increases with age and oxidation, which contributes to fatigue cracking and temperature cracking. Improperly compacted asphalt pavements with higher air void contents will experience accelerated oxidative hardening. Inadequate compaction leaves a higher percentage of interconnected air voids, which allows more air to penetrate into the asphalt mixture, leading to increased oxidation and hardening of the asphalt.

Because of the chemical complexities of asphalt, most existing specifications have been developed around physical property tests. All currently used specifications use penetration, viscosity, and ductility tests. These physical property tests are performed at standard test temperatures and the test results are used to determine if the material meets the specification criteria.

In 1987, the SHRP began a $50 million research effort to develop new procedures for defining and measuring the physical properties of asphalt, along with the concurrent development of new HMA mix design procedures. A major result of the binder-related research effort is the Superpave binder specification which utilizes all new test equipment and procedures to determine and define binder properties. A unique feature of the Superpave specification is that the specified performance criteria remain constant, but the temperature at which the criteria must be achieved changes for the various grades. As discussed, existing specifications set up a range of viscosities and penetration values at set temperatures, which represented laboratory, field, and plant mixing temperatures.

The Superpave tests measure physical properties that can be related directly to field performance by engineering principles. The Superpave binder tests are also conducted at temperatures that are encountered by in-service pavements to provide better binder performance for specific environmental regions. The new test procedures are noted in figure 3-1.3, along with some indication of binder performance predicted (Asphalt Institute SP-1). However, these test procedures are used to establish the mix design. Ultimately, the pavement structure, mixture design, and of paramount importance, the as-constructed mix properties, along with binder properties, will ultimately determine the performance of the pavement over its service life.

In addition to Superpave binder specifications, Superpave mix designs are being developed to help improve the performance of dense-graded HMA concrete pavements by improving the mix design procedures.

Finally, this module also covered the part mineral aggregates have in the construction and ultimate service of HMA pavement. One of the most variable properties of mineral aggregate is that of aggregate grading. Aggregate gradation is the distribution of particle sizes expressed as a percent of the total mass (weight). Gradation is determined by passing the material through a series of sieves stacked with progressively smaller openings and then weighing the material retained on each sieve. Grading of the mineral aggregate is still one of the more difficult properties to control in the construction of HMA pavement and, consequently, has considerable effect on the ultimate performance of the HMA pavement. There are also several basic gradings that are used to produce distinctly different HMA mixes such as Open-Graded Friction Courses and Stone Mastic Asphalt Pavements which were also described and discussed in this module.

6. REFERENCES

1. "Bituminous Products for Road Construction and Maintenance," Manual 2 July 1995 published by Sabita Ltd., P.O. Box 6946, Roggebaai 8012, South Africa.

2. The Asphalt Handbook Manual Series No. 4 (MS-4) 1989 Edition Published by the Asphalt Institute P.O. Box 14052, Lexington, KY 40512-4052.

3. Superpave Asphalt Institute Superpave Series No. 1 (SP-1), Asphalt Institute, Research Park Drive, Lexington, KY 40512-405.

4. Wada, Y. and Hirose, H., "Glass Transition Phenomena and Rheological Properties of Petroleum Asphalt," Journal of the Physical Society of Japan Vol. 15 No. 10 (1960).

5. Goodrich, Joseph L. "Asphalt Binder Rheology, Asphalt Concrete Rheology and Asphalt Concrete Mix Properties," Unpublished.

6. Peterson, J.C. "Chemical Composition of Asphalt as Related to Asphalt Durability - State of the Art," Presented at the 63 Meeting of the Transportation Research Board, Washington DC., January 1984.

7. Stroup-Gardiner, Mary, Newcomb, David E., "Polymer Literature Review," Research Report No. MN/RC - 95/27, Minnesota Department of Transportation, September 1995.

8. Private discussions with Dr. Joseph L Goodrich.

9. Bouldin, M.G., Collins, J.H., Berker, A., "Rheology and Microstructure of Polymer/Asphalt Blends," Rubber Chemistry and Technology, Vol. 64, No. 4, September - October 1991.

10. "The Shell Bitumen Handbook," published by Shell Bitumen UK, Riverside House, Guildford Street, Chertsey Surrey KT169AU 1990 ISBN-0-9516625-0-3.

11. “Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types,” Asphalt Institute Manual Series No. 2 (MS-2) Sixth Edition, Asphalt Institute, Research Park Drive, Lexington, KY 40512-4052.

12. “Washington State Pavement Design Guide Volume 2,” Washington State Department of Transportation, P.O. Box 169, Olympia WA, 98501.

13. “Open-Graded Friction Courses,” Technical Advisory United States Department of Transportation Federal Highway Administration T5040.31, December 26, 1990, Washington DC.

14. Smith, Harry A, “Performance Characteristics of Open-Graded Friction Courses,” National Cooperative Highway Research Program, Synthesis of Highway Practice 180, Transportation Research Board, Washington, DC. September 1992.

15. Huddleston, I.J., Zhou, H., Hicks, R.G., “Evaluation of Open -Graded Asphalt Concrete Mixtures Used in Oregon,” Transportation Research Record No. 1427, Transportation Research Board, National Academy Press, Washington, DC. 1993.

16. “Stone Mastic Asphalt (SMA) Mixture Design,” Federal Highway Administration, Publication No. FHWA-RD-92-006, McLean, Virginia 22101, March 1992.

17. Brown, D. “SMA Comes of Age,” Roads & Bridges Magazine, January 1997, Scranton Gillette Communications, Inc. Des Plaines, IL 60016-2282.

MODULE 3-2

JOINT AND CRACK SEALING

1. INSTRUCTIONAL OBJECTIVES

The sealing and resealing of joints and cracks in both portland cement concrete (PCC) and hot-mix asphalt (HMA) pavements is an important part of pavement maintenance and restoration that is often not adequately considered. If performed effectively and in a timely manner, joint and crack sealing can help reduce pavement deterioration and, thereby, prolong pavement life. This module will cover the sealing and resealing of cracks in HMA pavements. Module 4-2 will cover the sealing and resealing of joints and cracks in PCC pavements.

This module describes recommended procedures for crack sealing operations. Upon successful completion of this module, participants will be able to accomplish the following:

1. Identify the major factors that affect sealant performance.
2. Describe the procedures required to seal or reseal cracks in HMA pavements.
3. Identify the primary sealant types, appropriate specifications, and sealant properties.

2. INTRODUCTION

Crack sealing is one of the more commonly performed pavement maintenance activities. One objective of these activities is to reduce the amount of moisture that can infiltrate a pavement structure, thereby reducing moisture-related distresses, such as stripping, pumping of fines and increased fatigue cracking. Crack sealing is performed to extend the service life of the existing pavement or as a surface preparation treatment conducted prior to the construction of an overlay to extend the service life of the overlay.

A 1990 national survey of crack sealing practices for flexible HMA pavements found that 45 of 50 States sealed cracks.(1) Over 70 percent of the States that reported sealing cracks reported that crack sealing extended the pavement service life by at least 3 years. Crack sealing extends the service life of a pavement by eliminating or reducing the entrance of water into the pavement structure.

In 1990, the Strategic Highway Research Program (SHRP) initiated a nationwide study on the Long Term Pavement Performance (LTPP) studies. One of the Specific Pavement Studies (SPS-3) dealt with the preventive maintenance effectiveness of flexible pavements. The SPS-3 experimental plan consisted of constructing four typical maintenance treatments: crack seal, chip seal, slurry seal, and a thin HMA pavement overlay on similar adjacent pavement sections, and then comparing them to a control section with no maintenance treatments. Preliminary results of the study from surveys conducted in 1993 clearly indicate that the crack sealed sections, as well as the rest of the treatment section, are in better condition than the control sections where no sealing was done after 3 years.(2) It is too soon yet for the results of the experiment to indicate which of the treatments is more cost-effective. Of 12 observations and recommendations made in the 1994 report, the first two are specifically pertinent to preventive maintenance treatments and crack sealing in particular:

"The pavement sections on which preventive maintenance treatments have been applied have generally out-performed the associated control sections (i.e., sections that received no treatments)." [2]

"A specific treatment's performance is generally related to the condition of the pavement at the time the treatment was applied. Treatments applied to pavements in good condition have good results, treatments applied to pavements in poor condition have poor results, and treatments applied to pavements whose conditions are somewhere in the middle have mixed results, depending on the treatment type and traffic." [2]

3. DEFINITIONS

Sealing operations on flexible pavements address various forms of cracking that may occur, such as thermal cracking, reflection cracking, block cracking, and alligator cracking.[4] Crack sealing is most effective on transverse thermal and transverse reflection cracks.

Cracks in HMA pavements are irregular in dimension and direction. Generally, three repair methods for cracks in HMA pavements are recognized: crack sealing, crack filling, and crack repair. Only crack sealing and crack filling are addressed in this module. Although little distinction has been made in the past between crack sealing and crack filling, the purpose of each activity is different.[3] Crack sealing is a comprehensive operation, involving thorough crack preparation and placement of high-quality materials into or over candidate cracks. It is typically performed on working cracks that are not severely deteriorated.

The purpose of crack filling is to reduce the amount of water infiltrating the crack which can cause stripping or which can weaken the bound layers and cause a reduction in stiffness and strength in the unbound layers of the roadway. Crack filling involves limited crack preparation and placement of lower quality materials. It is normally conducted on cracks that experience little movement (less than 2.5 mm) and that are slightly or moderately deteriorated. Crack filling is intended as a short-term deterrent to pavement deterioration until more substantial rehabilitation can be performed.

Sealant Materials

There is a wide variety of sealants on the market today, each with its own inherent characteristics. The general categories used by the American Concrete Institute (ACI) to differentiate among sealing materials are:[5]

- Thermoplastic materials.
  - Hot-Applied.
  - Cold-Applied.

- Thermosetting materials.
  - Chemically-cured.
  - Solvent release.

Table 3-2.1 provides a summary of the different sealant materials by category, including applicable specifications and typical costs.

Table 3-2.1. Typical sealing and filling materials.

SEALANT/FILLER MATERIAL	EXAMPLE PRODUCT	APPLICABLE SPECIFICATION(S)	COST RANGE ($/kg)
Thermoplastic Materials			
Asphalt Cement	AC-10, AC-20	ASTM D 3381	0.11-0.33
Asphalt Emulsion	CRS-2, HFMS	ASTM D 244, ASTM D 977, ASTM D 2397	0.11-0.33
Polymer-Modified Asphalt Emulsion	Witco CRF, Hy-Grade Kold Flo	ASTM D 244, ASTM D 977, ASTM D 2397	0.88-1.22
Asphalt Rubber (Hot-Applied)	Koch 9000, Crafco AR2	ASTM D 5078	0.44-0.66
Fiberized Asphalt	Kapejo Bonifibers + AC Hercules FiberPave + AC	---	0.33-0.55
PVC Coal Tar	Crafco Superseal 444, Meadows Gardox	ASTM D 3406	1.44-2.10
Rubberized Asphalt	Koch 9005, Crafco 221, Meadows Hi-Spec	ASTM D 1190, D 3405 AASHTO M 173, M 301	0.44-1.10
Low Modulus Rubberized Asphalt	Crafco 231, Koch 9030, Meadows Sof-Seal	Mod. ASTM D 1190, D 3405 Mod. AASHTO M 173, M 301	0.77-1.44
Thermosetting Materials			
Polysulfide	Koch 9015	Fed. Spec. SS-S-200E	2.21-2.76
Polyurethane	Vulchem, Sikaflex, Burke U-Seal	Fed. Spec. SS-S-200E	6.08-7.18
Silicone	Dow 888, Mobay 960, Crafco RS	Fed. Spec. TT-S-001543A, TT-S-00230C	5.52-7.73

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. Many thermoplastic sealants are applied in a heated form, although some are diluted such that they can be installed without heat.

Hot-Applied

Among the more widely-used hot-applied thermoplastic sealants are asphalt cement, asphalt rubber, rubberized asphalt, fiberized asphalt, and polyvinyl chloride (PVC) coal tar. Asphalt cement has been used for many years as a sealant material with very limited success because of its poor elastic properties. Although inexpensive, it is not generally used for sealing medium- and high-volume roadways. Asphalt cement is generally graded by its viscosity or penetration properties.

Asphalt rubber has been very widely-used on highways, airfields, and city streets. It is a blend of asphalt and suspended, unmelted rubber particles that results in improved elasticity, greater cohesiveness, and an increased softening point. Essentially, the quality of the asphalt rubber varies with the quality of rubber incorporated. A new standard specification, ASTM D 5078, was recently released for asphalt rubber materials.

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. The resulting sealants possess a greater working range with respect to low temperature extensibility and resistance to high temperature softening and tracking. Most of the rubberized asphalt materials are governed by ASTM D 3405.

In recent years, softer grades of asphalt have been used in the rubberized asphalts to further improve low temperature extensibility. These materials, referred to as low modulus rubberized asphalt sealants, are used for crack sealing operations in many northern States because of their increased extensibility.

Polyester, polypropylene, and polyethylene fibers have also been added to asphalt in an effort to enhance sealant performance. However, extensibility and resistance to softening are only slightly improved and adhesion is reduced in relation to the original asphalt. Currently, no standard specifications exist for fiber-modified sealants.

Several agencies have used PVC coal tar sealant materials. These materials are fuel and jet blast resistant, have a high softening point, and bond well to concrete. Installation requires accurate temperature control, care in adding fresh sealant, and adherence to continuous heating limits. However, due to variable performance in the field and the potential health problems associated with coal tars, PVC sealants are not widely-used. These materials are governed by ASTM D 3406.

Cold-Applied

Cold-applied asphalt cutbacks and emulsions have been used as joint and crack sealant materials with limited success. Cutbacks consist of asphalts that have been thinned with light petroleum solvents. As the solvents evaporate, the asphalt cures, thus eliminating the heating requirements of many sealants. However, poor performance and environmental restrictions have substantially reduced the use of asphalt cutbacks. ASTM D 1850 is occasionally used to specify these sealant materials.

An asphalt emulsion is a mixture of small asphalt particles suspended in water and an emulsifying agent (such as soap). Such emulsions can be applied with or without heating to wet or dry joints. They maintain fair elastic properties, but are temperature-sensitive and prone to tracking.[6] Modifying agents such as rubber and polymer are added to some emulsions to improve performance. Occasionally, sand is added to the emulsion when filling wide cracks.

Thermosetting Sealant Materials

Thermosetting sealants are typically one or two component materials that set by the release of solvents or cure through a chemical reaction. Some of these sealants have shown potential for good performance, but they are also four to ten times more expensive (material cost) than standard rubberized asphalt. However, thermosetting sealants often are placed thinner and may have lower labor and equipment costs.

Chemically-Cured

Chemically-cured sealants are the predominant type of thermosetting materials used in highway sealing applications. They include: polysulfides, polyurethanes, silicones, and epoxies.

The performance of polysulfide and polyurethane sealants has been variable. While these materials seem to retain elasticity fairly well, their adhesive capabilities are questionable, particularly the polysulfides. In addition, most of these materials are two-component materials, which introduces an additional step in the sealing operation, thereby increasing sealing time and providing another source for error.

Silicone sealants are one-part, cold-applied materials that have been used in the paving industry since the 1970s. Their properties include good extensibility, resistance to weathering, and temperature susceptibility resistance. These sealants have bonding strength in combination with a low modulus that allows them to be placed thinner than the normal sealants.(7) Because of silicone's thin layer application and lower associated equipment costs, the ratio of in-place cost compared to rubberized asphalt is not nearly as high as the ratio of material cost (for a given volume).

Silicone sealants are available in self-leveling and non-self-leveling forms. The non-self-leveling silicone requires a separate tooling operation to press the sealant against the sidewall and to form a uniform recessed surface. Recently developed self-leveling silicone sealants can be placed in one step since they freely flow to fill the joint reservoir without tooling. Performance of silicone sealants is typically tied to joint cleanliness and tooling effectiveness. Many States, such as Georgia and Kentucky, have developed their own silicone specifications.

Solvent Release

Thermosetting sealant materials that cure through the release of solvent are included in this category. Examples of these materials include chlorosulfonated polyethylene and certain butyl and neoprene products.(5) However, because the extension-compression range of these materials does not exceed $\pm$ 7 percent, they are not used on pavements.

4. PURPOSE AND APPLICATION

In flexible pavements, non-sealed or poorly sealed cracks allow moisture and debris to enter the pavement structure, contributing to asphalt stripping, secondary cracking, cupping (depressed crack edges), spalling, and lipping (elevated crack edges). In addition, the presence of excess water in the pavement base or subgrade tends to reduce the compressive and shear strengths of the supporting materials immediately below and adjacent to the crack.(6) As a result, applied traffic loads in the vicinity of a crack create greater pavement deflections, additional cracking, cupping, and, eventually, potholes.

Flexible pavements exhibiting advanced stages of alligator cracking should not be treated by crack sealing. The presence of this distress indicates a structural deficiency in the pavement and structural solutions (e.g., a structural overlay) are required, but the surface can be temporarily sealed with a localized chip seal. However, flexible pavements displaying block cracking, a non-load-associated distress, can be sealed to prevent further deterioration.

Most types of longitudinal and transverse cracking (including reflection cracking) can be effectively sealed. Of course, the amount and severity of these cracks occurring on a flexible pavement will dictate whether crack sealing is a cost-effective alternative. If the pavement is badly deteriorated and exhibiting extensive longitudinal and/or transverse cracking, then it may be more suited to crack filling until more substantial rehabilitation work can be effected.

5. LIMITATIONS AND EFFECTIVENESS

Crack sealing is most effective when conducted on pavements exhibiting little structural deterioration. As noted previously, flexible pavements displaying extensive alligator cracking or severe crack deterioration are candidates for more rigorous rehabilitation alternatives, or temporary sealing with a chip seal.

Most agencies allow crack sealing operations on pavements throughout the year, although some limit it to the winter months, when the tranverse cracks will be open the widest. However, this latter practice places the sealant material in a state of compression during most of the year, so that tracking and pull-out failures may occur during the summer months.

Joint resealing should be performed when the existing sealant material is no longer performing its intended function. The optimum time to perform joint resealing is in the spring or the fall when moderate installation temperatures are prevalent.

Many sealant failures have been attributed to poor or inadequate preparation of the joint/crack and to poor material handling procedures. (See references 1, 4, 8, and 9.) Proper preparation of the joint or crack is essential to ensure that the sealant material can effectively bond to the sidewalls. If latent, dust, water, or other debris remains on the sidewalls, a good bond will not be achieved.

Many of the newer sealant materials are sensitive to heating and application temperatures, and indeed narrower temperature ranges are being recommended by manufacturers.[8] The use of supplementary temperature monitoring devices are recommended so that the sealant temperature can be closely observed. Underheating the material results in poor bonding, whereas overheating of the material destroys its ductile properties.[8]

Increased roughness and irritating tire noise may develop on flexible pavements that have been extensively sealed. This is particularly true where the sealing material has been allowed to build up over transverse cracks. These types of installations decrease the quality of the service the pavement provides the user and may actually shorten the effective service life of the pavement.

In the past, the effectiveness of joint sealing has been questioned by some agencies. For example, one agency contends that the purported benefits derived from joint sealing do not offset the costs of sealing and resealing operations.[10] Under experiments SPS-3 and SPS-4 of the SHRP program, the effect of sealing activities on pavement performance is being examined. While this debate may never be completely resolved, those efforts should go a long way toward identifying whether sealing activities are effective and under what conditions they should be applied. Nonetheless, the overwhelming majority of States' experiences support the contention that sealing cracks and resealing joints is a meaningful rehabilitation activity, within the constraints discussed elsewhere in this module.

6. DESIGN CONSIDERATIONS

The following factors should be addressed when planning crack-sealing or crack filling operations:

- Climate conditions.
  - At time of installation.
  - General.

- Highway classification.
- Traffic level and percent trucks.
- Crack characteristics and density.
- Materials.
- Material placement configurations.
- Procedures and equipment.
- Safety.

The planning processes should concentrate on selecting the optimum material and placement configuration, and determining the procedures and equipment to use based on the existing and future roadway conditions.

In the past, little distinction was made between crack filling and crack sealing. The purpose and functions of each of these specific treatments must be clearly understood so that the most cost-effective procedure is used. The two different procedures are defined in SHRP-H-348 as:

- Crack Sealing. The placement of specialized materials either above or into working cracks using unique configurations to prevent the intrusion of water and incompressibles into the crack.

- Crack Filling. The placement of materials into non-working cracks to substantially reduce infiltration of water and to reinforce the adjacent pavement.

Working refers to cracks that experience horizontal and/or vertical movement equal to or greater than 2.5 mm, and non-working refers to movement less than 2.5 mm either from load or annual temperature changes.

Normally, working cracks with limited edge deterioration should be sealed, while non-working cracks with moderate to no edge deterioration should be filled. Whether a crack is working or non-working can generally be determined by the type of crack. The most common types of cracks that are generally characterized as working cracks are transverse thermal cracks and reflective cracks. For optimum performance, sealant placed in working cracks must adhere to the crack sidewall and flex as the crack opens and closes. The demands for adhesion and flexure are not as great for non-working cracks.

Several different material placement configurations have been used for treating cracks in HMA pavements. Figure 3-2.1 illustrates a few of the more common configurations in which crack sealants and crack fillers are placed. In the past, materials were typically placed flush or recessed in refaced crack reservoirs (figure 3-2.1a and figure 3-2.1b) or, in the case of non-refaced cracks, flush or capped (figure 3-2.1c and figure 3-2.1d). In recent years, the concept of a controlled cap has become popular. This concept, referred to as overbanding, can be used on both non-refaced cracks and refaced cracks, as depicted in figure 3-2.1e and figure 3-2.1f. The overband appears to provide fairly good performance, particularly when used with rubberized asphalt and in areas where damage from snow plows is unlikely. Several agencies have reported good performance with the overbanding configuration.[8,16,17] However, many States have experienced problems with sealant ridging or bleeding through a new overlay if the sealant was applied too thickly and/or too soon before constructing the overlay.[2] The overbanding configuration should not be used if the pavement being crack sealed will be overlaid in the near future.

Sealant Properties

Critical sealant properties that significantly affect the performance of the sealant material include the following:

- 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 incompressibles 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 incompressibles 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.

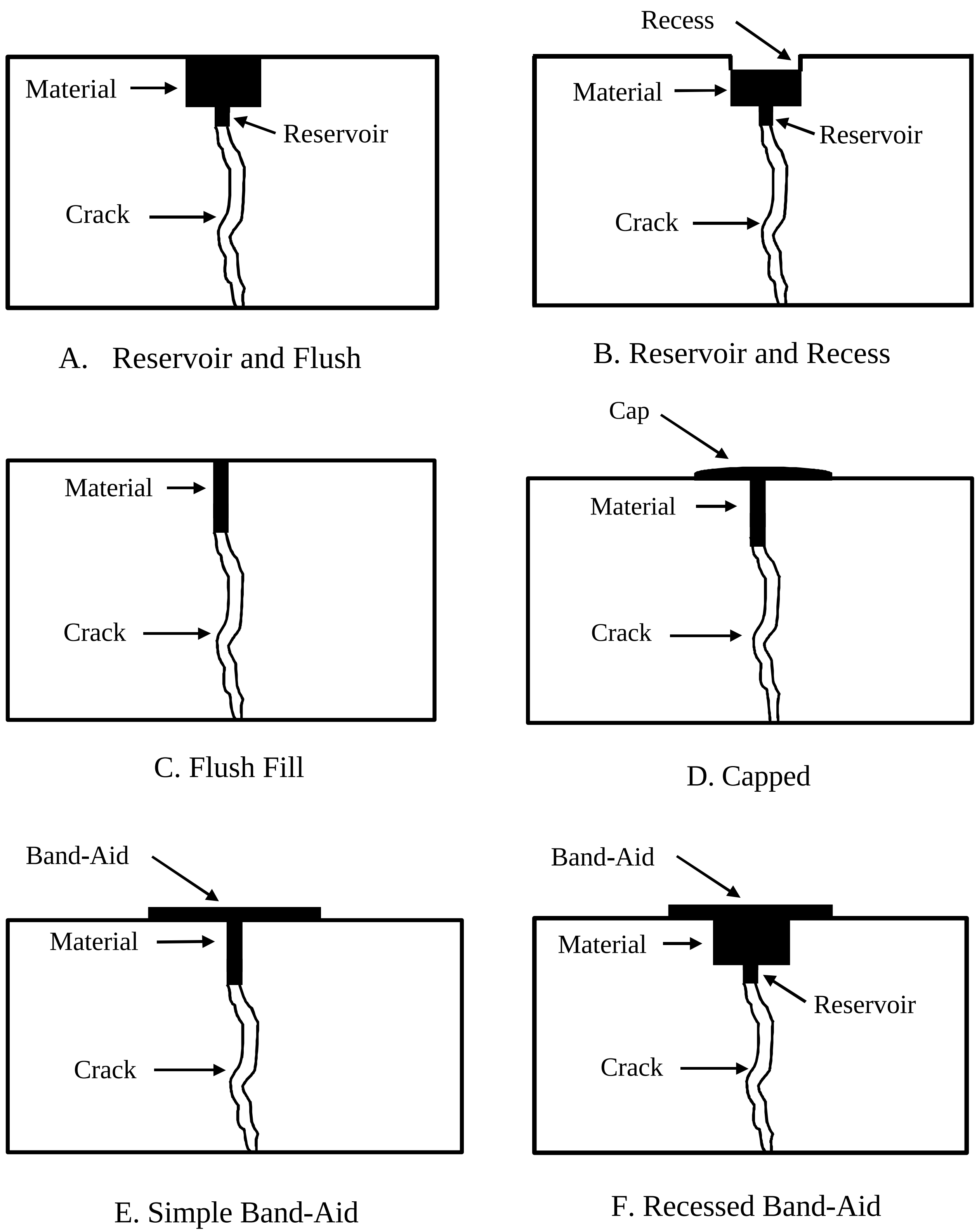


Figure 3-2.1. Illustration of various sealant configurations.

Typically, higher-quality sealants are specified for joint resealing operations than for crack sealing operations. However, it may be more cost-effective to use lower quality sealants in joints or cracks that experience little movement or that will be overlaid or otherwise rehabilitated in the near future.

References 8 and 18 provide information on attempts to correlate laboratory sealant properties to the performance of the joint or crack sealant material in the field.

7. PAVEMENT SURVEYS

For a pavement to be considered as a candidate for joint or crack sealing, several pieces of information should be collected first. This includes the type of pavement, its overall condition, the condition of the joints, and the type, amount, and width of the cracking that is displayed.

Flexible Pavements

For flexible pavements, the first piece of information needed is the type, amount, and condition of cracking occurring in the pavement. If alligator cracking has developed over a significant portion of the pavement, crack sealing or filling is not an effective activity. Flexible pavements with block cracking and most types of linear cracking are better candidates for these operations.

Additional information needed to determine the suitability of crack sealing includes the approximate linear amount of cracks to be sealed, the pavement cross section (to determine whether cracks are reflective or not), the longest and shortest crack spacing (to determine crack movement limits), and the approximate crack widths (or range of crack widths).

Generally, flexible pavement cracks less than 3 mm are not recommended for crack sealing. Most cracks between 3 mm and 25 mm are recommended for crack sealing, provided that limited secondary cracking exists. Cracks greater than 25 mm wide should be patched using standard patching materials and procedures. From a timing standpoint, crack sealing should be done shortly after the working cracks have developed.

8. COST CONSIDERATIONS

While all material types are used in new concrete construction, concrete joint resealing and crack sealing operations generally employ hot-applied thermoplastic materials and chemically-cured thermosetting materials. Maintenance materials for cracks in flexible pavements are normally hot and cold applied thermoplastics. Table 3-6.1, presented earlier, shows various sealant types and typical cost ranges. It should be noted that these costs are material costs only; labor, traffic control, and equipment costs are not included. When making cost comparisons, the total installation cost and the anticipated life of the sealant material must be considered.

9. CONSTRUCTION SEQUENCE

After the most appropriate sealant 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.

Crack Sealing

The sealing of cracks in flexible pavements is a comprehensive procedure requiring preparation of the crack prior to the installation of the sealant material. Steps in the sealing of HMA cracks are:

- Crack refacing (optional, depending upon agency specifications).
- Crack cleaning.
- Sealant installation.

Crack Refacing

The creation of a reservoir for cracks in flexible pavements continues to be a controversial issue. Some agencies believe that the limited benefits gained from this operation are not enough to offset its additional cost. Others cite the benefits of establishing a properly-dimensioned reservoir. Much of the experience here is based on a mix of working and non-working cracks. Distinguishing between working and non-working cracks and the need to crack seal or crack fill should help to more clearly quantify the relative benefits of refacing. Refacing is not required when crack filling non-working cracks. Refacing should be considered when crack sealing working cracks.

In any event, if the cracks are to be refaced, the reservoirs should be designed to accommodate the strain resulting from predicted crack movement. Crack refacing may be accomplished through the use of routers or diamond-bladed saws, with the use of routers more common.

Modern crack routers come in two fundamental designs: rotary impact and vertical spindle. A rotary impact router consists of multiple cutter blades mounted about horizontal shafts that are spaced along the periphery of a circular cutting head. The cutting head rotates about a primary horizontal shaft. As a result, the cutter blades experience two types of motion: circular and translational.

Vertical spindle routers, similar to wood routers, consist of a clustered cutting head that rotates about a vertical shaft to remove pavement material to the desired depth and width. To maintain the router position over the crack, a skilled operator is required. If old sealant is present in the crack, it may be smeared on the crack face by this routing operation.

Rotary impact routers are much more productive than vertical spindle routers; however, their tendency to spall and fracture the pavement surround is correspondingly greater. The damage that rotary impact routers can do to asphalt pavements is not nearly as critical as the damage they cause on rigid pavements. While they may cause new fractures, rotary impact routers assist in removing any pre-existing damage or loose material from the edge of the crack.

Diamond-bladed saws are occasionally used in asphalt crack refacing operations. As in concrete crack sealing, crack saws with small diameter blades and lightweight two or three wheel designs are more successful in following the irregular crack patterns. Yet, compared to routers, they are not as maneuverable and, therefore, have a greater tendency to miss cracks when used by inexperienced operators. In instances where experienced operators are available, diamond-bladed saws can provide good results.

Crack Cleaning

The most critical step in the HMA crack sealing operation is the cleaning of the crack. The sidewalls of the crack must be free of all foreign matter that would interfere with sealant bonding. Crack cleaning in HMA pavements is typically performed in one of three fashions:

- Airblasting with compressed air.
- Wire brushing followed by airblasting.
- Hot airblasting using a hot compressed air (HCA) lance.

Compressed air units having oil and water filtering devices generally do an adequate job of cleaning debris from cracks. Wire brushing is occasionally necessary to help in the removal of any sticky laitance left behind on the crack sidewalls by refacing equipment.

The use of the hot compressed-air (HCA) lance has been found to be not only an effective means of cleaning and drying a crack prior to sealing, but also a way of extending the conditions under which the crack sealing operations take place.[(4)] An HCA lance produces a stream of hot, compressed air that can be directed at the crack to remove debris, eliminate moisture, and warm the crack sidewalls to enhance the bond between the sidewalls and the sealant material. HCA lances that are capable of producing 1650 ^{o}C air at a velocity of about 800 m/s have produced good results and are recommended. However, in using the HCA lance, it is important that the HMA pavement not be scorched or burned with the unit.

Sealant Installation

The materials used for sealing cracks in HMA pavements are primarily hot or cold applied thermoplastics. Recently, however, a specially formulated silicone has been introduced for sealing applications in HMA pavement. Originally developed for sealing longitudinal PCC lane-asphalt shoulder joints, the silicone has undergone some experimentation in sealing working transverse cracks in HMA pavements.

The sealant installation and finishing phases are normally conducted in close sequence. Squeegees used to finish the material should follow within 0.6 to 0.9m of the sealant applicator wand. This trailing distance may need to be increased for thinner sealants or when deep cracks are encountered that necessitate a second sealant application.

The thermoplastic sealing materials generally come prepackaged in 18.1- to 22.7-kg blocks that can simply be placed in the melter for heating. As mentioned previously, care must be taken to follow manufacturers' recommendations with regard to safe heating temperature, recommended placement temperature, and prolonged heating limitations. A supplemental thermometer is highly recommended for verifying material heating temperatures taken from the melter gauges.

In some instances, modifiers are purchased from manufacturers and must be added on-site to the base asphalt cement. This is often the case with fiber and crumb rubber materials. Manufacturers generally provide a recommended percentage for incorporating the modifiers, given the grade of asphalt and type of application.

Flexible Pavement Crack Filling

As previously mentioned, the flexible pavement crack filling operation is intended to simply fill existing cracks in HMA pavements with a low-quality material to deter pavement deterioration until more substantial rehabilitation work can be done. As such, little, if any, crack preparation work is conducted, although rapid cleaning with compressed air may occasionally be performed.

Cold-applied and unmodified asphalts are primarily considered for crack filling operations. These materials are often applied using cornucopia pour pots, which are hand-held, conical containers that use gravity-flow principles for application of the material to the crack.

After application of the filler materials, a blotter material may be placed to prevent tracking or pick up of materials by traffic. Two of the more popular blotting materials are sand and stone chips.

A strong case can be made for using the higher quality hot-applied thermoplastic materials for crack sealing, considering that the cost of the sealant is by far one of the lesser costs in any crack filling operation, compared to the much higher costs of labor, equipment and user delays.

10. EQUIPMENT

A brief description of the equipment used in joint and crack sealing operations is described in this section. Unless stated otherwise, all of the equipment described here is suited to either joint or crack sealing operations.

Equipment for Crack Refacing

The equipment used for refacing operations consists of the following:

- Diamond-Bladed Saw. These are 26- 46-kW water-cooled saws equipped 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.

- Vertical Spindle Routers. Vertical spindle routers, like wood routers, consist of a clustered cutting head which rotates about a vertical shaft to remove concrete to the desired depth and width. To maintain the router position over the crack, a skilled operator is required. If old sealant is present in the crack, it may be smeared on the crack face by this routing operation.

- Rotary Impact Router. A rotary impact router consists of multiple cutter blades mounted about horizontal shafts that are spaced along the periphery of a circular cutting head. The cutting head rotates about a primary horizontal shaft. As a result, the cutter blades experience two types of motion: circular and translational. Rotary impact routers are not recommended for concrete crack refacing, as they tend to spall the reservoir edges. For removal of sealant in PCC joints, routers are slower and have the potential for spalling joint edges.

Equipment for Crack Cleaning

Obtaining clean, dry sidewalls prior to sealant placement is critical to the performance of the sealant installation. Equipment used for the cleaning operation includes the following:

- 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 or crack, but are not as effective as sandblasting at removing laitance. As a minimum, compressed air units should have a blast pressure of 690 kPa and a blast volume of 4.3 m^3/min.

- Wire Brushes. Wire brushes are mechanical, power-driven devices used in conjunction with compressed air to remove debris and loosened fragments in HMA pavement cracks.

- HCA Lance. A hot compressed air lance is often used to deliver a high velocity stream of hot air to the crack that removes most of the debris, dries water in the crack, and promotes bonding by heating the area surrounding the crack. HCA lances that produce 1650 ºC air at a velocity of about 800 m/s are highly recommended, although extreme care must be exercised to avoid burning the asphalt.

Equipment for Joint/Crack Sealant Placement

- Melters. Hot-poured thermoplastic materials must be 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 manner of heating.

- 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 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 and crack. Another way of applying a sealant material is using a cornucopia pour pot, which is a hand-held, conical-shaped pot used to apply unheated or partially-heated emulsions into cracks.

- Squeegee. Squeegees are finishing tools, with either a U- or V-shaped rubber blade, used to strike-off or shape excess material applied to a joint or crack.

11. SUMMARY

This module presents information on joint and crack sealing in rigid and flexible pavements. The need for sealing operations is discussed, including guidelines for identifying candidate projects. Various sealant materials that are available are presented, along with their properties, applicable specifications, and typical costs.

Important design considerations for joint and crack sealing are described. These include the anticipated joint/crack movements, the shape factor (W:D) of the sealant material as installed, the sealant configuration, and the properties of the sealant.

Procedures for the following sealing operations are described:

- PCC transverse joint resealing.
- PCC crack sealing.
- PCC longitudinal joint resealing.
- Asphalt concrete (AC) crack sealing.
- AC crack filling.

With the exception of the crack filling operation, which is essentially the placement of filler material in a crack with little, if any preparation, the steps for each of these operations include: refacing, cleaning, and installation of the new sealant material. Adequate cleaning of the sidewalls of the joint or crack is especially important to promote bonding of the sealant material.

Guidelines for joint and crack sealing operations are provided in references 20, 21, and 22.

12. REFERENCES

1. Eaton, R. A., Ashcraft, J., "State-of-the-Art Survey of Flexible Pavement Crack Sealing Procedures in the United States" CRREL Report 92-18, United States Army Corps of Engineers Cold Regions Research & Engineering Laboratory, September 1992.

2. Raza, H., "Summary Report - 1993 Field Evaluations of SPS-3 and SPS-4 Test Sites" FHWA Report No FHWA-SA-94-078, Federal Highway Administration Washington, D. C., October 1994.

3. Smith, K. L., Romine, R., " Materials and Procedures for Sealing and Filling Cracks in Asphalt-Surfaced Pavements", SHRP- H-348 Asphalt Pavement Repair Manuals of Practice, Strategic Highway Research Program, Washington, DC, 1993.

4. Rossman, R. H., H. G. Tufty, L. Nicholas, and M. C. Belangie, "Value Engineering Study of the Repair of Transverse Cracking in Asphalt Concrete Pavements," FHWA-TS-89-010, Federal Highway Administration, May 1990.

5. "Guide to Joint Sealants for Concrete Structures," ACI 504R-90, American Concrete Institute, 1990.

6. Chehovits, J. and M. Manning, "Materials and Methods for Sealing Cracks in Asphalt Concrete Pavements," <u>Transportation Research Record 990</u>, Transportation Research Board, 1984.

7. Zimmer, T. R., S. H. Carpenter, and M. I. Darter, "Field Performance of a Low-Modulus Silicone Highway Joint Sealant," <u>Transportation Research Record No. 990</u>, Transportation Research Board, 1984.

8. Belangie, M. C. and D. I. Anderson, "Crack Sealing Methods and Materials for Flexible Pavements," FHWA/UT-85/1, Utah Department of Transportation, May 1985 (appendix revised 1987).

9. Blais, E. J., "Value Engineering Study of Crack and Joint Sealing," FHWA-TS-84-221, Federal Highway Administration, December 1984.

10. Shober, S. F., “Portland Cement Concrete Pavement Performance as Influenced by Sealed and Unsealed Contraction Joints,” Transportation Research Record 1083, Transportation Research Board, 1986.

11. Spells, S. and J. M. Klosowski, “The Importance of Sealant Modulus to Performance of Concrete Highway Joint Sealants Under Vertical Shear,” Transportation Research Record 1041, Transportation Research Board, 1985.

12. Darter, M. I., “Design of Zero-Maintenance Plain Jointed Concrete Pavement, Volume I Õ Development of Design Procedures,” FHWA-RD-77-111, Federal Highway Administration, June 1977.

13. Wolters, R. O., “Sealing Cracks in Bituminous Pavements,” Report No. 185, Minnesota Department of Highways, 1973.

14. Barksdale, R. D. and R. G. Hicks, “Improved Pavement-Shoulder Joint Design,” NCHRP Report 202, Transportation Research Board, 1979.

15. Tons, E., “A Theoretical Approach to Design of a Road Joint Seal,” Highway Research Board Bulletin 229, Highway Research Board, 1959.

16. Chong, G. J. and W. A. Phang, “Improved Preventive Maintenance: Sealing Cracks in Flexible Pavements in Cold Regions,” Transportation Research Record 1205, Transportation Research Board, 1988.

17. Turgeon, C. M., “Evaluation of Materials and Methods for Bituminous Pavement Crack Sealing and Filling,” Final Report, Investigation 9LRR660, Minnesota Department of Transportation, June 1989.

18. Cook, J. P., F. E. Weisgerber, and I. A. Minkarah, “Evaluation of Joint and Crack Sealants,” FHWA/0H-91/007, Ohio Department of Transportation, January 1990.

19. “Rigid Pavement Design for Airports, Chapter 7 Õ Standard Practices for Sealing Joints and Cracks in Airfield Pavements,” Air Force Manual 88-6, January 1983.

20. “Pavement Rehabilitation Manual,” FHWA-ED-88-025, Federal Highway Administration, September 1985 (Supplemented April 1986, July 1987, March 1988, February 1989, October 1990).

21. Darter, M. I., E. J. Barenberg, and W. A. Yrjanson, “Joint Repair Methods for Portland Cement Concrete Pavements,” NCHRP 281, Transportation Research Board, 1985.

22. “Guide Procedures for Concrete Pavement 4R Operations,” AASHTO-AGC-ARTBA Joint Committee, Subcommittee on New Highway Materials, Task Force 23, 1985.

MODULE 3-3

PATCHING WITH BITUMINOUS MIXTURES

1. INSTRUCTIONAL OBJECTIVES

This module describes the properties needed in a bituminous patching mixture for both pavement maintenance and rehabilitation. The techniques for constructing acceptable patches are covered for both maintenance and rehabilitation. This module emphasizes the use of bituminous patching materials for flexible pavements, although bituminous materials can be used on rigid pavements in emergency situations. Permanent repairs of concrete pavements should be made with cementitious materials.

The participants will be able to accomplish the following upon successful completion of this module:

1. List the desirable properties of a bituminous patching mixture.
2. Differentiate between HMA materials and cold-mix, stockpiled patching mixtures on the basis of quality, aggregate, and binder.
3. Describe the conditions that require patching and differentiate between temporary, semi-permanent patching, and localized reconstruction.
4. List the steps necessary to accomplish patching as part of a 4R project.

2. INTRODUCTION

Bituminous patching in a rehabilitation project requires that consideration be given to the pavement structure, the material used in the patching operation, and the procedures used in construction of the patch. Various materials have been used for bituminous patching, with a wide range of results. There are important considerations that must be considered when determining the appropriate materials to use in different applications.

Bituminous patches can be either full or partial depth. Partial-depth repairs usually involve removing the surface layer and replacing it with hot-mix asphalt concrete.(1) Full-depth repairs involve removal of the complete pavement down to the subgrade or to an intermediate layer that is intact.(1) Most pothole repairs on flexible pavements are full-depth.

Bituminous patching materials are mixtures composed primarily of bituminous binders and aggregates. The two main types of bituminous patching materials available are:

- Materials that are mixed and stockpiled for a period of time before use, known as “cold mixes.” These materials may be heated prior to placement in some instances.

- Materials that are mixed hot and compacted in the hole while still hot, referred to as hot-mix asphalt concrete (HMA), or hot-mix.

3. DEFINITIONS

Cold Mixes

Cold-mix patching materials have traditionally been considered for maintenance as a temporary solution to be used during colder weather when hot-mix materials are unavailable and when there is insufficient time for more rigorous patching procedures. Since they are not intended to be a permanent solution, they are not recommended for use in a permanent rehabilitation project. Temporary and semi-permanent patching with cold-mix materials is performed as a stop-gap measure until higher-quality materials can be placed using recommended procedures or until a rehabilitation can be performed.

Cold mixes are a combination of aggregate and either of two asphalt-based binders: cutbacks or emulsions. Chemical modifiers can be added to the binder material, whose purpose is to enhance certain characteristics of the mix, most notably stripping resistance characteristics. Some patching materials incorporate polypropylene or polyester fibers into the material to improve stability.

Binders

Cutbacks are a blend of asphalt cement and a solvent such as gasoline, kerosene, or diesel fuel. The solvent makes the asphalt cement fluid for mixing, and then evaporates upon application, leaving the asphalt cement as the residue coating the aggregate. Because of more recent environmental restrictions on the use of cutbacks, they are not as widely-used as they once were.

Among cutbacks, some of the most commonly used liquid binders for stockpiled patching mixtures are medium-curing (MC) cutbacks. The MC binders keep a patching mixture workable for long periods of time and, in addition, have a fairly hard residue for good stability. Both the MC-250 and MC-800 grades are used often. The thicker grade (MC-800) can be used in the fall and spring, but the thinner grade (MC-250) is normally desired for winter use. Slow-curing cutbacks are sometimes used, but do not have as hard a residue.

Emulsions are a blend of asphalt cement, water, and an emulsifying agent. The emulsifying process breaks the asphalt cement into minute droplets that are suspended in the water by the chemical action of the emulsifying agent. Water makes the asphalt cement fluid for mixing, and then evaporates upon application. The environmental advantage of emulsions is that they do not release hydrocarbons into the atmosphere. Some emulsions, such as high-float emulsions (HFE), also contain solvents.

Emulsions may either be anionic (bearing a negative charge) or cationic (bearing a positive charge), depending upon the type of emulsifying agent used. The emulsion should be compatible with the charge of the mineral aggregate being used in the patching mix. For example, most siliceous aggregates (sandstone, quartzite) are negatively charged and require a cationic emulsion, whereas some limestone aggregates are positively charged and require an anionic emulsion.[(1)]

Medium-curing and slow-curing asphalt emulsions and cationic emulsions are commonly used. However, many agencies do not like these liquid binders, as the stockpiled mixes seem to be stiff and quite difficult to handle. On the other hand, patching mixtures made with HFE seem to have good workability after being stored. HFE materials allow the retention of much thicker films of residual asphalt on the aggregate.[(2)] These materials will improve in workability as they are manipulated and will have few problems with drainage during storage due to the thicker films of residual asphalt. The softer grades, HFE-150 and HFE-300, are used for patching mixtures. The use of high-float emulsions is increasing as the binder of choice for emulsion mixes.

Some agencies make their own liquid binders for patching mixtures. Normally an AC-10 asphalt cement is thinned with an SC-250 or possibly a special petroleum distillate, such as kerosene or stove oil. These liquid binders have strong residues and the resulting patching mixture has good workability at fairly low temperatures (-7 °C to -9 °C). The amount of thinner used can be varied for the actual conditions encountered. Diesel should not be used in these conditions as it produces a very soft and slow curing binder.

Aggregates

The key to success with cold patching mixtures lies in the aggregate gradation and quality. Table 3-3.1 contains several aggregate gradations that have been studied in the past. These gradations have all been used with some success over the years. It is recommended that the gradation used in a stockpiled, cold mixture be an open gradation with very little fines. This will provide a mixture with a large amount of voids and will allow space for the volatiles in the binder to cure and allow for a more thorough set of the material. Crushed aggregates are absolutely essential due to the stability generated by the aggregate-aggregate interfaces.

Table 3-3.1. Aggregate gradations used for cold-mix patching.[2,4]

Sieve Size (mm)	**PADOT Cold Mix 485**		**CTDOT Cold Mix Class 14**		**NCHRP 64 Intermed. "C"**		**ASTM D3515 Open Mixtures**	
	Percent Passing		Percent Passing		Percent Passing		Percent Passing	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
19			90	100				
12.5			70	100		100		100
9.5		100	60	82	90	100	85	100
4.75	85	100	40	65	20	40	40	70
2.36	10	40	28	50	15	30	10	35
1.18	10				10	25	5	25
0.60					5	15		
0.30			6	26	0	10	0	12
0.075	0	2	2	8	0	5		

A one-sized, crushed aggregate gradation having 100 percent passing the 9.5 mm sieve and 100 percent retained above the 1.18 mm sieve has been proposed by Kandahl.[3] Initial field trials indicate that the mix has good storability and workability, and produces a patch with good performance when placed properly.

In the past few years, proprietary cold-mix materials have been developed that exhibit good performance. These materials are mixed in a plant for better uniformity, and use binders that have specially designed modifiers to enhance their performance. Tight controls on the aggregates, especially gradation and cleanliness, also promote good performance.

The majority of the proprietary cold mixes on the market today use an emulsified asphalt as the base binder, and are modified with different chemicals to promote stripping resistance, stability, and cohesion. Three widely-used proprietary materials are:

- Sylvax UPM.
- QPR 2000.
- Perma Patch.

All three of these materials can be produced at a plant using locally available aggregate and truckloads of the specially-formulated binder. Commonly, the companies dispensing the binder will design the mix proportions using local aggregate and run tests on the finished product to check on the quality of the mix. The costs of these materials is higher than conventional cold mixes, but the increases in patch life over conventional materials seems to be sufficient to reduce "repeat patching" and may help to bring down the life-cycle cost of the overall operation.

Several agencies have attempted to develop non-proprietary cold mix materials with the same enhanced performance. Three examples are:(2)

- PennDOT 485.
- PennDOT 486.
- HFMS-2 with styrelf.

The two PennDOT materials are mixed using emulsions and anti-stripping additives; the 486 material also has polyester fibers added for additional stability and cohesion. The HFMS-2 with styrelf was produced and approved in Oklahoma, and uses a high-float, medium-set emulsion modified with styrene butadiene to improve stability.(2)

Hot-Mix Asphalt

HMA is the traditional paving material used to construct flexible pavements and overlays of PCC pavements. Asphalt cement is heated and mixed into heated aggregate at a central plant. Since no curing process is required, the problem of improper curing is avoided. HMA provides the highest quality and most consistent product, it is the recommended material for major maintenance patching projects requiring longer life and rehabilitation projects.

Since a stiff binder with good durability is desired, asphalt cements are the binders usually used. The grades most frequently used are:

- AC-20 (60 to 70 penetration).
- AC-10 (85 to 100 penetration).
- AC-5 (120 to 150 penetration).

The grade used is normally the same grade used in asphalt paving concrete, although some agencies use a stiffer grade in patches to improve stability. The aggregates used must be of the same quality and gradation as those used in normal hot-mix production.

While HMA is the preferred repair material, its expense and availability often limits its application. In the northern part of the United States, most hot-mix plants close down in winter so that agencies are unable to obtain HMA at the height of the patching season. Furthermore, the storability of HMA also limits its more widespread use.

4. PURPOSE AND APPLICATION

Patching of flexible pavements is generally performed for one of several reasons. These include any or all of the following:

- To repair localized distress.
- To improve motorist safety.
- To reduce the pavement roughness.
- To reduce the rate of pavement deterioration.
- To repair a pavement prior to overlay for improved support.

Pothole Repair

Potholes in flexible pavements are associated either with a localized soft base or subgrade materials or from moisture access through transverse or longitudinal reflection or temperature cracks. The localized failure pothole initiates in a location that is weaker than the surrounding area. This may result from a number of causes, such as a loss of sufficient load carrying support caused by the presence of water in the foundation, localized soft materials not considered in the pavement design, moisture in the pavement structure combined with a large number of freeze-thaw cycles, or the variability of the original construction materials. Whatever the cause, the pavement section exhibits higher deflections under load than other sections, as shown in figure 3-3.1a and figure 3-3.1b.

Large numbers of heavy traffic loadings on these weak areas produce high deflections, resulting in very rapid fatigue cracking. These cracks allow water to infiltrate, softening the lower layer more, thus accelerating the rate of cracking, as shown in figure 3-3.1c and figure 3-3.1d. Eventually the fatigue cracking becomes so severe that traffic removes chunks of the asphalt from the weakened area.

When a pavement has deteriorated to the stage where potholes develop, the damage is more widespread than is visible at the surface and the pavement should be evaluated thoroughly to determine the actual extent of its deterioration. As most pavement fatigue cracking usually starts from the bottom of the pavement and progresses to the surface, there is much more fatigue cracked pavement than can be seen on the surface. Generally, pothole patching is only a temporary repair, as the surrounding pavement will continue to deteriorate, unless that area is also incorporated in the repair. Pavements with badly deteriorated sections of substantial length may require more extensive rehabilitation or reconstruction, rather than temporary patching.

Potholes also develop at reflective cracks in HMA overlays of rigid pavements or cement-stabilized bases. Their development is related primarily to the presence of moisture in the HMA layer or at the HMA/PCC interface. Reflection cracks in an HMA overlay of a rigid pavement expose the HMA material to water. Water works to deteriorate the reflection crack by destroying the integrity of the HMA material. In northern climates, water freezing in the cracks further deteriorates the HMA overlay. Loss of integrity near the crack makes it much easier for the overlay to lose bond and break into pieces under continued traffic.

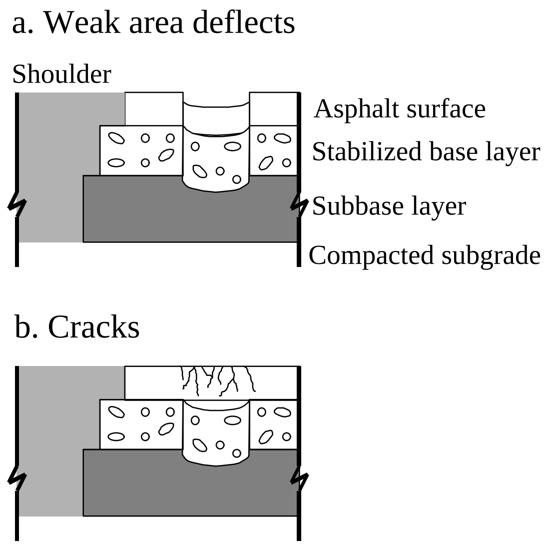


Figure 3-3.1. Pothole formation and deterioration in an HMA pavement.(5)

5. LIMITATIONS AND EFFECTIVENESS

Localized distresses such as potholes are not the only reason bituminous patching may be needed. As mentioned above, when potholes are seen, there may be more extensive damage within the pavement. In flexible pavements, large areas may develop fatigue cracking that indicate a structural inadequacy. If these areas are not patched properly accounting for what caused the distress, resurfacing will prove to be a waste of time and money, as the overlay will continue to deteriorate very rapidly in these areas because they already exhibit poor support. This form of patching is more appropriately termed "localized reconstruction." If the overlay thickness can be decreased or the service life of the overlay can be increased, then the cost-effectiveness of the pavement repair can be determined. If the costs of the repair exceed the resulting benefits of reduced overlay thickness or extended service life of the overlay, then the repair is not warranted.

6. DESIGN CONSIDERATIONS

Patching Material Mix Design

Bituminous patching mixtures must develop certain properties in order to perform well. These properties are also necessary in other types of bituminous mixtures and typically are found in HMA. For all patching mixtures, the following are important:

- Stability. Bituminous patching materials must possess adequate stability to resist shoving and rutting under repeated loads.

- Adhesiveness. Depending upon the construction procedure and the size of patch, the materials must be sticky enough to adhere well to the sides and bottom of the hole.

- Resistance to Stripping. The bituminous binder and aggregate combination must resist stripping in the presence of moisture.

- Durability. Patching mixtures must resist deterioration and disintegration due to traffic loadings and local climatic conditions.

For cold mixtures only, the following additional properties are required:

- Workability. Cold mixtures must be easily handled, shoveled, and compacted. The type of binder and the temperature are the main factors that influence workability. Mixtures with hard binders at low temperatures cannot be handled easily and should be avoided.

- Storability. Cold mix patching mixtures must be able to be stored for a long period of time exposed to weather and still retain their workability. Drainage of the asphalt binder from the stockpile, in addition to being an environmental problem, can also pose problems with poor quality cold mixes.

All patching mixtures should possess the above properties to provide for a good patching material. If a normal HMA has problems with stability, stripping, durability, or cracking, it will not perform as well as a patching mixture. Patching mixtures are commonly subjected to more severe conditions than normal HMA materials in flexible pavements, because they are placed in locations known to have poor support and typical high moisture areas.

Hot-mix patching materials should be subjected to the same standard tests as conducted on HMA for new construction. This includes such items as Marshall stability, Marshall flow and voids in the mineral aggregate (VMA). For cold mixes, there are several test methods which address their specific properties. Some of these procedures include the rolling sieve test, developed in Ontario, and the Texas boiling test.[6,7] Procedures have been developed to quantify workability and storability in cold mixtures, such as the pocket penetrometer test developed by Penn State and the blade penetrometer developed in Ontario.[2,6]

Pavement Surveys

The principal need for patching in HMA pavements is to repair localized areas of distress. Areas with fatigue cracking can be visually quantified from the distress survey conducted during the evaluation, but the quantity of visible distress is not always indicative of the extent of the deterioration. Figure 3-3.2 shows the relationship between surface deflection measured with an nondestructive testing (NDT) device and areas with visible fatigue cracking. In the figure, areas with visible fatigue cracking are candidates for patching. However, the plot also shows areas with high deflections that do not show visible fatigue cracking, because fatigue cracking begins at the bottom of the asphalt concrete and the cracks have not reached the surface.

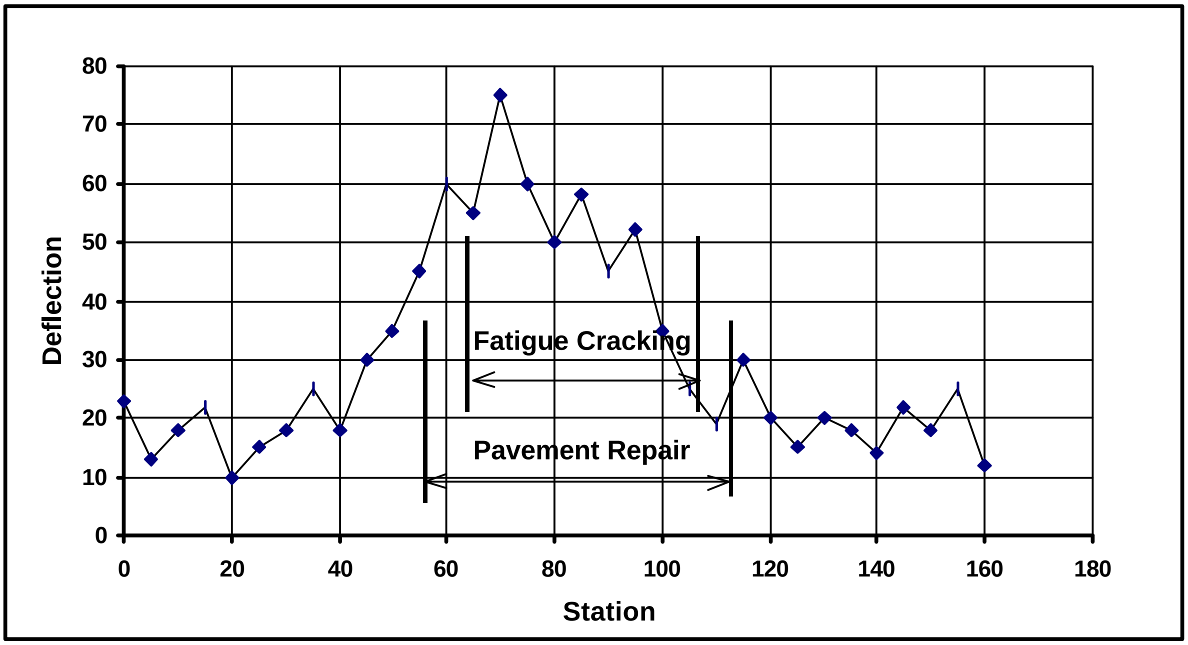


Figure 3-3.2. Deflections profile for an HMA pavement.

The selection of areas for patching prior to overlaying is an economic decision. Visible cracking of medium- to high-severity should be replaced to maintain a high level of integrity in the underlying asphalt layer. The areas with poor support should be replaced if it can be shown that patching these areas will reduce the thickness of overlay required (by repairing areas with high deflections, the mean deflection measured on the pavement is reduced, lowering the thickness of the material needed). Some combination of pavement area patched and overlay thickness can be determined that will provide the lowest cost overall. This is best determined with specific detailed deflection surveys on typical sections of potholes and fatigue cracking so the general extent of the fatigued week areas can be estimated and comparative estimates made.

7. COST CONSIDERATIONS

For Maintenance Applications

Hot-Mix Asphalt Concrete

Hot-laid patching mixtures are produced in hot-mix plants or hot-boxes. They contain quality aggregate, are accurately proportioned, contain hot, dry aggregate well coated with bitumen binder, and are compacted to high-density while hot. The workability of the hot-laid patching mixture depends upon its temperature. The mix must still be hot when it is compacted in the hole. This is the highest-quality type of bituminous patching mixture and should be used on all rehabilitation projects, if possible.

When patching with HMA, it is usually necessary to have a hot-box of some sort to keep the material hot and workable between patches. The distance between patches and the size of the patches will also factor into what type of equipment is needed. If there are a number of large patches in close proximity, then a hot-box may not be necessary to keep the material workable until it is unloaded. If the distance between the patches is greater, the material will need to be heated to prevent cooling and stiffening of the mix.

The cost of hot-mix asphalt varies greatly throughout the United States. Typical costs of HMA material is between $27.50 per metric ton and $44 per metric ton. However, costs of the actual patching operation will have to consider the equipment needed, the manpower required, and the productivity of the operation.

Cold Mixes

Cold mixes can generally be stored for a period of time between mixing and the time that they are used. Depending upon the type of binder, these patching mixtures may be stored for as long as six to 12 months. They are stockpiled so that they are available during the colder winter months.

Stockpiled patching mixtures are produced and used in a variety of ways. They can be produced in a plant (cold or hot) or they may be produced in the field where mixing is done by blading the cold aggregate and binder back and forth on the ground. After mixing and storing, the material may be taken directly from the stockpile, placed, and compacted in the hole. The stockpiled patching mixture may be run through a portable heater just before it is used to improve workability and compaction. Some agencies currently use a "hot-box" mounted in a pick-up truck bed that uses engine exhaust as a heat source.

The quality of cold mixtures can vary considerably. The lowest-quality patching mixture results when the aggregate and liquid binder are mixed by blade at the project site. The aggregate may be poorly-graded and neither hot nor dry, and thus, may be poorly coated by the binder. These patching materials often have stripping problems. After storage, if placed cold, they may be difficult to handle and compact and can have a very short life. Poor quality mixtures should not be used in any pavement repair, be it maintenance or rehabilitation. Plant run cold mixes are much higher quality, and are produced and used successfully all over the country.

The costs for these materials ranges from approximately $33 per metric ton for the PennDOT 485 to between $72 per metric ton and $88 per metric ton for the UPM. These prices reflect only the cost of the material, and do not include other costs, such as shipping, labor, or placement. It has been demonstrated that the majority of the costs associated with patching a pothole comes from the labor needed to perform the patching, especially when the patches do not last and need to be repatched. Estimates of costs to place material have been estimated at anywhere from $126 per metric ton to $374 per metric ton, depending on the crew size, equipment, and procedures used.[8]

Rehabilitation Pavement Repair

On projects where there is a large variability in subgrade soil types and deposition sequence compounded with unique natural or manmade drainage patterns, there will also be a large variation in pavement support. Obviously, the pavement will usually begin fatigue cracking in those locations with the weakest pavement support first. The optimum pavement management strategy favors the selection of projects as they start to fatigue crack, but before they are fully fatigue-cracked. This set of conditions leads to the need for some form of pavement repair as part of most pavement overlay project design.

The simplest way to determine the need for pavement repair under these conditions is to design the overlay based on, or including, the weakest sections in the design thickness computations, and then design it excluding the weakest sections. If there is a significant variation in the subgrade support and limited amount of fully fatigued pavement along the project, then it is not uncommon for the pavement thickness designs to differ by 50 mm or more. If there is not a significant variation in subgrade support and or the amount of fully-fatigued pavement increases, then the difference in the two design thickness become less as the cost for pavement repair becomes greater. There is obviously some point of diminishing returns where it is not as cost-effective to repair the pavement. This is shown conceptually (not real data) in figure 3-3.3 where the cost to construct the overlay decreases as the amount and cost of pavement repair is increased. There is some limiting range where it is not cost-effective to repair all damaged pavement. The point of diminishing returns changes with each project depending upon the amount and type of distress encountered.

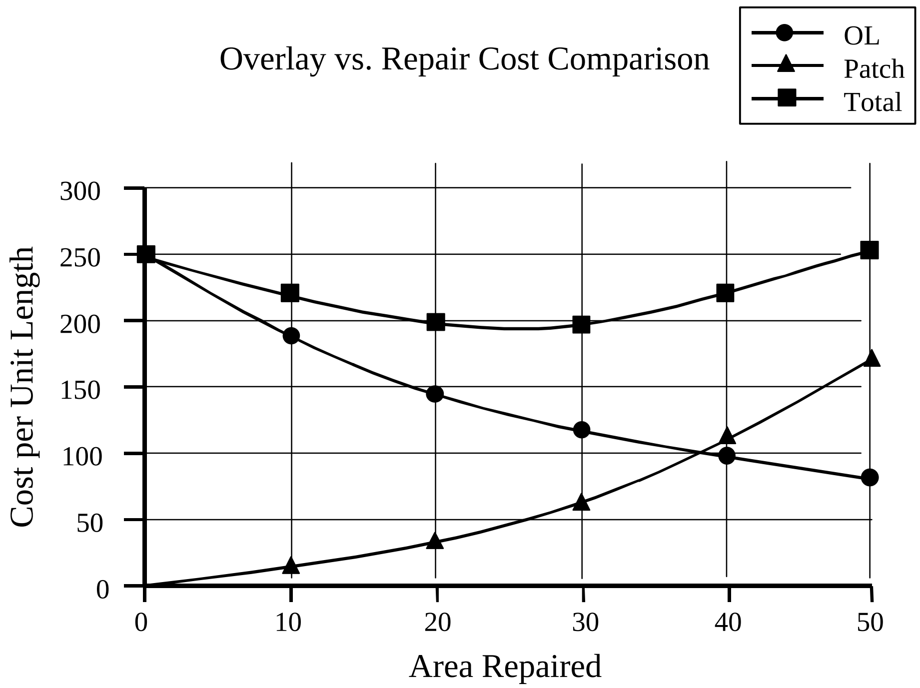


Figure 3-3.3. Example of pavement repair cost comparison.

While it is not practical to conduct a detailed cost estimate for varying repair levels as implied by figure 3-3.3 on projects where there is a significant difference in overlay thickness, it is practical to make some simple comparisons between a basic levels of pavement repair for the weakest sections and the reduced overlay costs.

7. CONSTRUCTION

There may be wintertime maintenance instances where HMA patching has to be performed using the "throw and go" approach. This occurs when there are patches to be placed during adverse weather or when patching must be performed in the presence of high traffic volumes. This procedure generally includes the shoveling of cold mix material into a pothole and leaving it for traffic to compact. This procedure is repeated in an attempt to patch as many potholes as quickly as possible. Although the throw-and-go procedure is quite common, it should never be used in a situation where the possibility exists for performing a more rigorous patching operation. Recent studies have indicated that better quality, State-specified and proprietary materials are able to provide quality patches even when placed in potholes filled with water, as long as time is taken to compact the material with truck tires, prior to allowing traffic on the patch.[2,9]

A life-cycle cost analysis of HMA patching procedures used in Pennsylvania found that throw-and-go procedures cost approximately <u>three times</u> more than semi-permanent patching procedures when considering life-cycle costs.[9] In that study, patching procedures were identified for the construction of a semi-permanent patch, which apply to maintenance patching as well and pavement repair prior to overlaying a pavement. The procedures are:

- Mark patch boundaries.
- Cut boundaries.
- Clean and repair foundation.
- Apply tack coat.
- Fill the hole with the patching material.
- Compact the patch.
- Cleanup.

These seven steps have been thoroughly described in a report prepared for the Pennsylvania Department of Transportation.[5] They are briefly discussed here because of their importance in the overall performance of the patch.

Mark Patch Boundaries

The boundaries must be marked to include all deteriorated areas. The area adjacent to the marked area should be free of cracks and appear solid. These boundaries can be changed if cutting and removal expose more deterioration than was first expected. The patch boundaries should be straight, but the patch area need not be rectangular. The straight sides will provide a firmer edge so that greater compaction can be achieved.

Cut Boundaries

The deteriorated material in the patch area must be removed, leaving sound vertical sides around the patch. Removal should begin in the center of the marked area and work out toward the patch boundaries. Removal should be to a depth where sound, dry material is encountered. Full-depth saw-cutting is not recommended, because it produces too smooth of a face to promote good bonding. Partial-depth saw cuts and breaking out with a jackhammer leaves the rough face needed for bonding and interlock with the patch material. The smooth cut at the surface provides a good riding surface and an edge that can be easily sealed. In order to speed up the patching operation, the saw may be omitted in lieu of a jackhammer equipped with a "clay spade." The clay spade is a flat chisel approximately 150 mm wide, and can be used to obtain acceptable results.

Clean and Repair Foundation

A thorough cleaning must be performed to provide a surface to which the tack and patch material can readily bond. Compressed air should be used to remove all loose particles, particularly in the corners. If the material remaining in the bottom of the open patch is wet, loose, soft, or disturbed, it must be reworked, dried out and recompacted, or otherwise removed. Again, this is "localized reconstruction." If high deflections are present and not corrected, the patch will deteriorate rapidly. Density levels and moisture contents should be monitored closely to ensure that an adequate structure is obtained.

Apply Tack Coat

A tack coat is important to the long-term performance of a patch placed with HMA. The tack coat "wets" the existing material to promote bonding with the patch mix. Emulsions, cutbacks, and synthetic resins all provide suitable results when properly used as tack coats. Tack coats should be low-pressure sprayed or brushed onto the patch area to obtain a uniform thin coating along the edges of the prepared hole. It is important that excessive tack not be used, as this can tend to collect in the bottom of the hole and reduce the effectiveness of the patch. When cold mix is used, the bottom of the patch should never be tacked unless the base of the patch is PCC. The solvents in the tack can soften the cold mix, increase shoving, and promote stripping.

Placing Material

Patching material can be placed in 100 mm lifts with good results. Thicker patches should be placed in lifts as determined by local experience and specifications, usually 50 to 75 mm. Patching material should be placed with a shovel working from one side of the patch to the other. Patch material should never be thrown or raked into the hole, as this could result in segregation. Enough material should be placed so that after compaction, the center of the patch is approximately 6 mm higher than the surrounding pavement.

Compact the Patch

Compaction is the most critical procedure in obtaining a good patch. HMA patching materials must be compacted before they cool. An undercompacted mix will ravel, shove out of the patch, or compact excessively and leave a depression. The compaction process and equipment should match the size of the hole being compacted. In general, the material should first be pinched into the hole by rolling the edges of the mixture. Next, the center of the patch should be rolled, moving outward toward the edges with each succeeding pass. The roller must rest completely on the patch mix and not on the old pavement. This will require rolling transversely across the pavement if rutting is present. Hand tamping should not be used as it does not provide adequate compactive effort.(1)

Cleanup

The pavement should be swept clean, and then the edge of the patch may be sealed using any standard asphalt product. Synthetic resins should not be used because they remain tacky and will be picked up by vehicle wheels. An asphalt material can be poured along the patch edges and brushed lightly. Then, clean sand or screenings should be placed over the asphalt product. Road dust or large stones should not be used as the blotting material for the edge seal.

8. EQUIPMENT

Patching Equipment

An effective pothole repair requires the use of proper equipment. Equipment for a conventional patching operation can be classified into four groups:(10)

- Cleaning, tacking, and sealing equipment. Included in this category are brooms, brushes, spray cans (for tack and seal), rags, and torches (for drying). An air compressor with a sprayer is useful in blowing debris from the hole and also for drying.

- Hole preparation equipment. A jackhammer with a chisel point is the best tool for squaring a hole and establishing vertical faces. Some jackhammers are hydraulically-driven while others run off a portable air compressor.

- Compaction equipment. Vibrating plate or vibrating roller compactors are very effective because they can "work" the mix while it is hot. Other effective compactors are static or nonvibrating steel wheel rollers (for large potholes).

- Mixing equipment. Equipment is needed to mix, heat, or recycle the patching material. These include recyclers, stockpile heaters, and hot boxes.

Automated Patching Equipment for Maintenance Patching

Recent developments in pothole repair have produced a series of automated or semi-automated patching equipment. Spray injection devices use a crushed aggregate and asphalt binder that are blown into a pothole simultaneously to produce a cold mix material. Three examples of this type of device are:

- Rosco RA-200.
- Durapatcher.
- Wildcat Roadpatcher.

All three pieces of equipment use an emulsified asphalt in most instances, which is heated within the unit to improve coating and compaction of the patches. All three machines follow the same basic procedure, outlined below:

- Blow debris from hole.
- Spray bottom of hole and surrounding pavement with binder to act as a tack coat.
- Blow aggregate and binder into hole to create patching material.
- Place a layer of cover aggregate to prevent tracking.

Most manufacturers of this equipment do not require compaction of the material, believing that the force of the material as it is blown into the hole provides sufficient compactive effort. A steel-wheeled roller will improve the surface roughness of spray injection patches. Rubber-tired rollers should be avoided for two to three days depending on the set time of the binder, but may also be used for added compaction.

While the spray injection devices "mix" the cold mix material in a different manner, the same steps should be taken to ensure that the aggregate and binder are compatible, just as with any other cold mix material.

Other automated equipment augers premixed material into the pothole to speed the placement process. Preparation of the pothole and compaction of the material after placement will still be required to ensure long-lasting patches.

All of the proprietary and State-specified materials discussed in this section are currently being evaluated under the Strategic Highway Research Program (SHRP) H-106 project. The spray injection devices listed are also being evaluated under SHRP H-106. All of the materials and devices are being compared to determine which materials and procedures offer the best performance.

9. SUMMARY

The use of both HMA and stockpiled, cold-mix bituminous patching mixtures to repair HMA pavements are discussed in this module. The properties needed, the materials required, and the construction techniques used for placing these mixtures are discussed.

- The properties needed in a bituminous patching mixture are stability, adhesiveness, stripping resistance, durability, and workability.

- HMA should be considered as the primary material for permanent patching projects, because they are of higher quality, use well-graded aggregate instead of open-graded aggregate, and contain hard asphalt cement instead of liquid cutbacks and emulsions.

- Two methods may be used to make a patching mixture workable:
  - Heating the mixture.
  - Using liquid binders.

Heating lowers the viscosity of the binder so that hot-laid patching mixtures are workable and can be compacted by the same procedures used for hot-mix asphalt concrete pavements. Liquid binders have inherently low viscosity to retain workability. Stockpiled mixtures are less stable and less durable than the HMA. However, newer mixes with open-graded, crushed aggregate, and with compatible, nonstripping binders, have shown improved performance, as have several widely available proprietary cold mix materials.

Procedures for constructing a semi-permanent patch are outlined. These include:

- Mark patch boundaries.
- Cut boundaries.
- Clean and repair foundation.
- Apply tack coat.
- Fill the hole with the patching material.
- Compact the patch.
- Cleanup.

Equipment used in patching operations is presented, including recent developments in automated patching equipment.

10. REFERENCES

1. Roberts, F.L., P.S. Kandhal, E.R. Brown, D.Y. Lee, and T.W. Kennedy, "Hot-Mix Asphalt Materials, Mixture Design, and Construction," National Asphalt Pavement Association, 1991.

2. Anderson, D.A., H.R. Thomas, Z. Siddiqui and D.D. Krivohlavek, "More Effective Cold, Wet-Weather Patching Materials for Asphalt Pavements," FHWA-RD-88-001, Federal Highway Administration, 1988.

3. Kandahl, P.S. and D.B. Mellott, "Rational Approach to Design of Bituminous Stockpile Patching Mixtures," Transportation Research Record No. 821, Transportation Research Board, 1981.

4. Herrin, M., "Bituminous Patching Mixtures," NCHRP Synthesis of Highway Practice No. 64, Transportation Research Board, 1979.

5. Anderson, D.A., H.R. Thomas, and W.P. Kilareski, "Pothole Repair Management: An Instructional Guide," PTI Report No. 8202, Pennsylvania Transportation Institute, 1982.

6. Tam, K.K. and D.F. Lynch, "New Methods for Testing Workability and Cohesion of Cold Patching Material," Ontario Ministry of Transportation, 1987.

7. Lee, K.W. and M.I. Al-Jarallah, "Utilization of Texas Boiling Test to Evaluate Effectiveness of Antistripping Additives in Saudi Arabia," Transportation Research Record No. 1096, Transportation Research Board 1986.

8. Anderson, D.A., H.R. Thomas, and Z. Siddiqui, "A Comprehensive Analysis of Pothole Repair Strategies for Flexible and Rigid Base Pavements," PTI Report No. 8618, Pennsylvania Transportation Institute, 1986.

9. Thomas, H.R. and D.A. Anderson, "Pothole Repair: You Can't Afford Not To Do It Right," Transportation Research Record No. 1102, Transportation Research Board, 1986.

10. Eaton, R.A., E.A. Wright, and W. E. Mongeon, "The Engineer's Pothole Repair Guide," Cold Regions Technical Digest No. 84-1, Cold Regions Research and Engineering Laboratory, United States Army Corps of Engineers, March 1984.

MODULE 3-4

COLD MILLING

1. INSTRUCTIONAL OBJECTIVES

This module describes the basic techniques for pavement surface restoration. The participant shall be able to accomplish the following upon successful completion of this module:

1. Describe cold milling and its basic objective.
2. List the major reasons for cold milling hot-mix asphalt (HMA) pavement surfaces.
3. Describe equipment and construction problems encountered in typical cold milling operations.

2. INTRODUCTION

Diamond-grinding, and cold milling are the most basic forms of surface restoration by surface removal that are used to correct a variety of surface distresses. Each of the techniques address a specific pavement shortcoming and commonly may be used in conjunction with other pavement restoration techniques as part of a comprehensive pavement restoration program. In certain circumstances, it may be justified to use one of these techniques as the sole restoration technique in a pavement rehabilitation project. Only cold milling will be covered in any detail in this module. Diamond-grinding and grooving will be covered in module 4, which deals with rigid pavements.

3. DEFINITIONS

Cold milling is generally performed only on flexible pavements with HMA surfaces, although it is being increasingly used on rigid pavements. Cold milling operations use drum-mounted carbide steel cutting bits to chip off the surface of the pavement. This technique is often used to correct rutting, to maintain curb lines, and in preparation of the pavement for an overlay.

4. PURPOSE AND PROJECT SELECTION

Cold Milling

Cold milling uses carbide-tipped cutters or bits, mounted in holders on a revolving drum, to chip away the pavement surface. Cold milling operations can be conducted over the entire lane width and project length, as in the case of rut removal, or on small areas for use in partial-depth repairs.

Cold milling can be used for mass removal of the asphalt surface, (50- to 100-mm) such as removing a layer of asphalt concrete pavement over a rigid pavement or removing a layer of rutted or raveling asphalt concrete pavement to recycle or replace in kind or as preparation for a new overlay. Cold milling can also be used to level and profile a pavement surface to correct minor roughness and rutting. Leveling and profiling existing pavement surfaces to provide a new wearing surface requires a finer surface texture than is required for mass removal. The finer surface texture is achieved using specially made cold milling drums that have two to three times as many carbide bits through the use of either additional bits on double or triple warp drums, or forked bits with two or three bits on single warp drums.

Conventional cold milling has been used very successfully for removing asphalt concrete surfacing and is often used to remove surface rutting.[1] It is possible to remove 75 to 100 mm of asphalt concrete in a single pass. Cold milling has also been used successfully on rigid pavements to provide a surface for bonding a concrete overlay, and for removing an asphalt overlay that has deteriorated.[8] Special care and equipment are needed for cold milling to be used for final surface texturing of rigid pavements because standard equipment and processes can produce too rough a surface and may cause excessive microcracking of the pavement surface and excessive spalling of the transverse joints.[9]

Major uses for cold milling include the following:

- Removal of rutting on HMA pavement.
- Restoration of curb line on HMA pavements.
- Restoration of cross slope for drainage or correction of drainage inlet cover problem on HMA pavements.
- Restoration of surface friction on HMA pavements.
- Removal of an HMA overlay on a rigid pavement.
- Provision of a roughened, clean surface for bonding a PCC overlay to an existing rigid pavement, although secondary cleaning of the surface is still required.
- Removal of HMA material in conjunction with asphalt concrete recycling.
- Removal of HMA and PCC material when conducting partial-depth repairs.

Cold milling is most often used to remove material prior to placement of an overlay. Cold milling creates a clean, roughened surface for bonding and eliminates the need to raise drainage structures and other utilities to the level of the new pavement. Cold milling can also be used to remove HMA material for use in recycling (see module 3-7).

5. LIMITATIONS AND EFFECTIVENESS

Cold Milling

Cold-milling provides a surface with good frictional characteristics, with improved macrotexture, and a better drainage profile. The use of milling machines, modified with the multihead blocks, offers the potential to obtain this improvement with a much finer surface texture, resulting in far less tire noise and a smoother ride. In time, traffic will wear away the milled pattern and skid resistance will depend solely on the fractured aggregate surface. If the aggregate is susceptible to polishing, loss of skid resistance will occur in a relatively short time. An investigation of the aggregate should precede the milling operation if the milled surface will be subjected to traffic for an extended period of time.

The surface texture produced by any milling operation is a function of the carbide bit spacing, the rotational speed of the drum, the bit quality and wear, and the speed at which the milling machine is advanced over the surface. Increasing the number of carbide bits, increasing the rotational speed of the drum, and using slower advance speeds, produces a smoother surface texture.

6. DESIGN CONSIDERATIONS

Assessing Resulting Surface Profile

The quality of diamond-grinding and cold milling work can be assessed through measurement of the pavement roughness after grinding. The same ride quality standards are used for grinding projects as for new rigid pavement construction.[13] Profile traces obtained prior to grinding can be compared to profiles obtained after grinding to determine and document improvements in the ride quality.

The most commonly used profile measuring device for grinding operations is the California profilograph. Many agencies currently use this device for acceptance testing on new rigid pavements and flexible pavements, and these specifications can be applied on grinding projects as well. Typical acceptance values for the California profilograph are as follows:

- 0.16 km increments: 0.19 m/km maximum.
- 1.6 km increments: 0.16 m/km average for job.
- 1.6 km increments: 0.11 m/km maximum.

The higher numbers historically have been used more in the eastern part of North America, while the lower numbers have been used in the western regions.

Other devices, such as the the Rainhart profilograph, Mays Ridemeter, BPR Roughometer, and the PCA Roadmeter, have also been used for acceptance testing. More recently non-contact profilers such as the K. J. Law Profilometer, South Dakota Profiler, RST Laser Profiler, and others have also been used for acceptance testing The test equipment to be used in acceptance testing should be the same as that used in the initial evaluation and should be specified along with procedures to be followed in acceptance testing. The acceptance criteria for any of these devices should be the same for projects that have been ground as they would be for new construction. Whenever a roughness measuring device is used, especially the road response types, particular attention should be paid to proper calibration.

Transverse slope is also important, and should be specified to obtain the desired grade. Generally, a transverse slope of 1.7 percent, or 6.4 mm over 3.66 m, is recommended. Grinding limits and transitions or stop lines at bridges and ramps should be clearly marked on the plans.

Each agency should carefully evaluate its pavements and equipment in arriving at reasonable acceptance standards. It must be remembered that the level of smoothness required has a great effect on the cost of the grinding operation. The setting of unrealistic levels that require extensive removal or additional grinding will cause a large increase in the bid cost of grinding.

Skid Resistance

Pavement skid resistance is improved through milling by the restoration of the cross-slope and the increased surface macrotexture. Proper cross-slope facilitates transverse drainage and reduces the potential for hydroplaning, especially in cases where studded tire wear has produced “ruts” in the concrete pavement. The increased macrotexture initially provides high skid numbers, but this improvement may be temporary, particularly if the pavement contains aggregate susceptible to polishing.[4] Skid resistance is generally measured using either a standard ribbed tire (ASTM E 501) or a standard smooth tire (ASTM E 524).

Pavement Removal and Surface Profiling

Cold milling operations can be used as a method of removing existing pavement layers prior to an overlay or to correct surface irregularities such as rutting or corrugations. In the first instance, milling should be considered as a means of maintaining curb height requirements after placement of an overlay or to provide material to be recycled and used as the overlay material. Information regarding overhead clearances and existing curb heights should be reviewed prior to determining the extent of cold milling to be done. Information on the material characteristics and its ability to be recycled would be needed if cold milling were being considered as part of a recycling operation. More information on recycling can be found in module 3-6 and module 3-7.

When milling is being used to correct rutting or corrugation, the amount of the material to be milled should be estimated. In instances where rutting and corrugation recur in the same areas, the integrity of the underlying support should be investigated so that the cause of the problem can be corrected, rather than addressing only the visible distress.

7. COST CONSIDERATIONS

Cold Milling Costs

The cost of cold milling is highly dependent on the material to be cold milled and the depth of removal. Typical costs for cold milling of asphalt cement pavements range from \$0.60/m^2 to \$3.60/m^2. The costs of milling PCC pavements are somewhat higher than those figures due to its hardness.

Diamond-Grinding Costs

The cost of diamond-grinding in asphalt, which is seldom done, is a little higher than cold milling costs. Diamond-grinding of PCC pavements ranges from \$3.00/m^2 to \$10.80/m^2.

8. CONSTRUCTION

Cold Milling

This technology has been used on both asphalt concrete and PCC pavements. For asphalt concrete pavements, the use of multi-head blocks may prove to be a cost-effective method for removing ruts and increasing skid resistance without producing a surface prone to excessive tire noise.

Cold milled surfaces have been opened to traffic directly without using an overlay. One milled asphalt concrete surface has been in service for over eight years on I-57 in Illinois. This pavement, an HMA/PCC composite, was milled to remove excessive rutting. There were some complaints by motorists due to the high tire noise initially produced, but as the milled surface has worn down and the noise level dropped, complaints have subsided. A similar project on SR 5 in western Washington was also milled to remove excess rutting but the contract required controls on the surface texture which resulted in the use of a triple tooth milling drum on the project. This project which provided a finer smoother surface texture than normal grinding, is over 6 years old and has generated no motorist complaints.

Cold milling can be conducted over small areas, as when used as the primary means of material removal for partial-depth patching. Cold milling can also be used on pavement that passes beneath bridges, and near curbs and drainage structures prior to overlaying, providing clearance and eliminating the need for expensive grade changes.

Cold milling has been gaining wider acceptance as a rehabilitation method for rigid pavements. One problem encountered in both Oregon and Iowa was the spalling of transverse joints and cracks. It is believed that this problem may be eliminated by filling the joints and cracks with a cementitious material prior to the milling operation.[7] Although a number of projects have been conducted in Iowa, Illinois, Oregon, Puerto Rico, and Washington, the long-term effectiveness of this technique for PCC has not been established.

9. EQUIPMENT

Cold Milling Equipment

Cold milling equipment uses carbide bits mounted on a revolving drum to break up and remove the surface material. Drum widths vary from as little as 0.3 m to as long as 3.6 m. The carbide bits must be continually maintained and frequently replaced to provide a uniform texture with no ridges or low spots. This is critical if the pavement is not going to receive an overlay. Positive, definitive grade control is also an essential part of a cold milling operation.(8)

Traditionally, a single carbide bit is mounted on a block, which is then bolted to the revolving drum. This results in a conventional bit spacing of approximately 15 mm. As the drum revolves and advances forward, the pavement material is impacted and is chipped away, producing a rough texture that is adequate for a riding surface.

Newer carbide cutting blocks are available that mount two to three carbide cutting bits on a single block (2-head and 3-head blocks). When attached to a conventional cold milling drum, the number of cutting bits is increased by a factor of two to three, and the spacing between bits is significantly reduced (to approximately 5 mm for a 3-head block). Drums modified in this manner produce a much smoother texture, more suitable for use as a riding surface. Some newer cold milling drums are being built that have a steeper angle to the wrap on which the bits are mounted which produces the same increase in carbide bits using single bit blocks.

In Oregon, cold milling equipment modified with the three-head blocks was used to remove ruts created in PCC pavement by studded tire wear.(19) The Oregon DOT evaluated microcracking in PCC cores obtained from pavement sections that were cold milled and diamond ground. A petrographic analysis conducted according to ASTM C 856-83 revealed that the limited amount of microcracking observed was more prevalent in those cores subjected to diamond-grinding than those abraded by the three-head carbide bits.(19) The resulting surface, although rougher than that produced by diamond-grinding, was still considered an acceptable riding surface.

Small milling machines are currently available which are specifically designed for partial-depth patching. These machines have cutting head widths from 0.3 to 0.9 m, and are highly maneuverable, making them ideal for use as a primary means of material removal. They have been used on both HMA and PCC for partial-depth pavement repairs and have increased the productivity of these repair operations.

Equipment must be inspected frequently to ensure all cutting bits are functioning properly and that worn bits are replaced. This is particularly critical if the pavement is not going to receive an overlay, as worn cutting bits will produce a surface texture characterized by ridges and low spots. Cold milling equipment can remove up to 75- to 100-mm of asphalt concrete with adequate grade control.

10. PROCEDURES

Cold Milling Procedures

Cold milling is generally conducted longitudinally along the pavement profile. It is recommended that failed pavement areas be patched prior to cold milling, as cracking becomes difficult to locate on the milled surface. The forward speed of the machine, the rotational velocity of the rotating drum, the spacing of the carbide bits, and the grade control of the cutting head should be closely controlled to produce a uniform texture throughout the project. The longitudinal profile should be held to the same tolerance as new construction.

11. SUMMARY

Diamond-grinding, and cold milling are two basic forms of surface restoration by surface removal that have been used successfully to correct a variety of rigid and flexible pavment surface distresses. The appropriate application of these techniques can result in a very cost-effective extension of pavement life.

Diamond-grinding uses closely-spaced diamond saw blades to remove a thin layer of material from a PCC surface, resulting in a very smooth surface. It is primarily used to remove faulting and pavement rutting caused by studded tire wear. Diamond-grinding can also improve skid resistance by increasing the macrotexture of the surface and correcting deficiencies in pavement drainage, although this effect may be temporary if the aggregate is susceptible to polishing. Diamond-grinding is typically used in conjunction with other concrete pavement restoration (CPR) techniques.

Cold milling uses carbide teeth cutting bits mounted on a revolving drum to chip away the surface material, producing a highly textured surface. Primarily used on HMA pavements, cold milling equipment has recently been modified for limited use on rigid pavements. Cold milling provides an excellent tool for asphalt concrete surface removal prior to placement of an overlay and when conducting partial-depth patching.

12. REFERENCES

1. Van Deusen, C., "Cold Planing of Asphalt Pavements," Proceedings, Association of Asphalt Paving Technologists, Volume 48, 1979.

2. Neal, B.F., and J.H. Woodstrom, "Evaluation of Cold Planers for Grinding PCC Pavements," FHWA-CA-TL-78-15, California Department of Transportation, 1978.

3. "Pavement Rehabilitation Manual, Volume II, Treatment Selection," New York State Department of Transportation, Materials Bureau, April 1991.

4. Henry, J.J., and K. Satio, "Skid-Resistance Measurements with Blank and Ribbed Test Tires and Their Relationship to Pavement Texture," Transportation Research Record 946, Transportation Research Board, 1983.

5. "Texturing of PCC Pavement Surfaces," Technical Advisory T 5140.10, Federal Highway Administration, September 1979.

6. "Pavement Texturing by Milling," Final Report for Iowa Highway Research Board Project HR-283, January 1987.

7. "Microscopical Examination of Pavement Cores for Microcracks Conducted for Oregon Department of Transportation," Construction Technologies Laboratory, Inc., March 1990.

8. "Pavement Rehabilitation Manual," Federal Highway Administration, FHWA-ED-88-025, September 1985 (Supplemented April 1986, July 1987, March 1988, February 1989, October 1990)

MODULE 3-5

SURFACE REHABILITATION TECHNIQUES

1. INSTRUCTIONAL OBJECTIVES

The use of various surface-applied rehabilitation techniques, such as chip seals, slurry seals, and friction seals, have been well-established methods of pavement rehabilitation for asphalt pavements of all classes of highways, from low-volume roads to interstate highways. The participants shall be able to accomplish the following upon successful completion of this module:

1. Identify and differentiate the major types of surface rehabilitation techniques.
2. Demonstrate the design principles required for the successful application of chip seals and open-graded friction courses.
3. Describe the construction sequences and describe the equipment operating characteristics for the various surface rehabilitation techniques.

2. INTRODUCTION

Surface rehabilitation techniques have long been used for asphalt pavement maintenance and rehabilitation procedures. Historically, they have been used primarily on low-volume roads in rural areas. More recently, they have been used on higher-volume streets and roads, successfully extending pavement life at lower expense. For some time, certain surface rehabilitation techniques have also been used in Europe to extend the performance of portland cement concrete (PCC) pavements and several highway agencies now employ these surfacing techniques in the United States on asphalt concrete pavements. Because of these newer applications, and to provide better understanding of past practices that still have utility surface rehabilitation techniques are discussed in this module to provide the engineer with an understanding of what these applications can reasonably be expected to do for a pavement. Surface rehabilitation techniques with relatively long-term performance may be considered for Federal 4R funding.

3. DEFINITIONS

Surface rehabilitation techniques include any application to a roadway surface of a layer of asphalt or asphaltic mixture with or without aggregate. Such mixtures are commonly in the range of eight to 25 mm thick. Their purpose is to seal the pavement and to improve or protect the surface characteristics of the roadway, but they generally do not provide any structural enhancement from increased thickness.

Types of Surface Rehabilitation Techniques

Surface rehabilitation techniques are classified by their composition which may be either solely asphalt, or a combination of asphalt, aggregate, modifiers, and by their use. Typical types of surface rehabilitation techniques are briefly described in this section.

Fog Seal

Fog seals are very light applications of an emulsion to the pavement surface with no aggregate. These applications seal the surface and provide a small amount of rejuvenation, depending on the type of emulsion used and the condition of the existing pavement surface.

Sand Seal

A sand seal usually consists of a spray application of a rapid-set emulsion with a light covering of sand or screenings. This application serves the same function as a fog seal, but provides better surface friction. However, the surface appearance of a sand seal does not provide the delineation that a fog seal does. A sand seal is typically 2- to 5-mm thick.

Asphalt Chip Seal

An asphalt chip seal, also referred to as a seal coat or a bituminous surface treatment, consists of sequential applications of asphalt and stone chips, applied either singly or in layers, to build up a structure that can approach 25 mm thick. This application is the traditional seal coating done by local agency maintenance crews and contractors. It serves as the surfacing for many miles of low-volume unpaved roads or as the wearing surface for many miles of paved roads. Many agencies apply multiple surface treatments to produce a thickness on the order of 10 to 25 mm.

Rubberized Asphalt Chip Seal

A rubberized asphalt chip seal is a special type of chip seal in which rubber (ground-rubber tires) is blended with the asphalt cement. This application has been used both as a SAM (stress-absorbing membrane) and a SAMI (stress-absorbing membrane interlayer) to help reduce reflection cracking, but it has also been used without overlays. The ground rubber adds stiffness and resiliency to the asphalt, and also improves bonding with the aggregate. The added stiffness and resiliency may also enable the seal to "bridge" existing cracks better.

Slurry Seal

A slurry seal consists of a diluted emulsion mixed with fine aggregate in a special mixer on the job site, and then squeegeed onto the pavement surface. It is effective in sealing surface cracks, waterproofing the pavement surface, and improving skid resistance at speeds below about 64 km/h. Different types of slurry seal are used that differ by the size of aggregate used. The thickness of the slurry seal is generally less than 9.5 mm.

Microsurfacing

Developed in Europe, microsurfacing is a term used to describe the application of a polymer-modified slurry seal, with latex rubber being the most commonly used polymer. Microsurfacing materials consist of asphalt and latex mixed with aggregate, fillers, and other additives and is a modification of the slurry and sand seal.[(3)] It has been used as a wearing surface and for rut-filling. Ralumac[(7)] is probably the most widely known example of this process in the United States.

Cape Seal

This application is a combination of both a chip seal and a slurry seal. For paved roads, the chip seal is applied first and, between four and 10 days later, the slurry seal is applied. For unsurfaced roads, an application of penetration oil (MC-70 or SC-70) is applied first as a prime coat. Then, about two days later, the chip seal is applied, and about two weeks after that, the slurry seal is applied after most of the cutter stock has cured out of the cutback. The advantage of the cape seal is that a thicker and longer-lasting surface is obtained, and it can be used on higher volume roads.[(1)] The cape seal also has a smoother, more pleasing appearance, and it is more resistant to damage from snowplowing.

Sandwich Seal

A sandwich seal consists of an application of a one-layer course of aggregate particles, followed by an application of emulsion (although asphalt cement can also be used), followed by a second course of smaller aggregate to fill the voids. The term “sandwich” is derived from the fact that the asphalt application is placed between the two layers of aggregate. Typical sandwich seals are between five and 20 mm thick.

Open-Graded Friction Course

Open-graded friction courses (OGFC), also called plant mix seals or popcorn mixes, are porous surface mixes produced with large amounts of voids (minimum 15 percent) that allow water to drain rapidly through the mix and flow to the side of the road. These mixes are used to improve the friction properties of the surface and also reduce the tire spray and hydroplaning potential, thereby reducing wet weather accidents. The OGFC also tends to provide a quieter riding surface, as it produces lower tire noise. Typical thicknesses of OGFC are 25 to 50 mm.(1) An excellent summary of the performance of OGFC is found in reference 2 and reference 19.

Functions of Surface Rehabilitation Techniques

The functions of surface rehabilitation techniques can be summarized as follows:

- Provide a new wearing surface.
- Seal cracks in the surface.
- Waterproof the surface.
- Improve pavement surface friction and surface drainage.
- Slow pavement weathering and aging.
- Improve the surface appearance.
- Provide visual delineation as between the mainline pavement and the shoulder.

There are many different ways in which an application can classify as a surface rehabilitation technique. However, it need not meet all of these requirements to qualify and probably will not. By meeting selected requirements, surface rehabilitation techniques may extend the service life of a pavement and reduce required maintenance expenditures.

All of the surface rehabilitation techniques provide a new wearing surface. However, those applications that include aggregate, especially the larger-sized aggregate, are often applied specifically to improve the wearing characteristics of the pavement. A chip seal surface treatment provides a new layer of aggregate exposed to the traffic, which can furnish better durability and wear characteristics than the original surface. In general, this application improves the surface friction. OGFC and rubberized chip seals will also improve the wearing characteristics of the pavement. The aggregate to be used should be tested using the Los Angeles Abrasion Test and the Sulfate Soundness Test to ensure that it has satisfactory durability.

Surface rehabilitation techniques provide a large amount of asphalt material that can cover and seal small cracks. All types of surface treatments will seal these cracks for a short period of time; success in this application is dependent on the size of the cracks, their movement, and the ability of the asphalt to penetrate the cracks. Chip sealing provides reasonably long-lasting crack sealing of nonworking cracks, depending on the extent and severity for the cracking. A double chip seal may be required to provide reasonable service-life where there is more severe cracking.

All surface rehabilitation techniques also serve to waterproof the pavement; the crack sealing and waterproofing aspects of these treatments restrict moisture infiltration and reduce the rate of pavement deterioration.

The use of surface rehabilitation techniques is also an effective means of improving surface friction. For example, the aggregate in a standard surface treatment directly increases the surface friction of a pavement by increasing the roughness of its macrotexture. To accomplish this, the aggregate used in the surface treatment must have good inherent friction characteristics, and the design and construction of the surface treatment must be controlled to ensure the level of surface friction will remain high following construction. An OGFC provides an additional contribution to improving surface friction in that the wet weather accident potential of a pavement is reduced since water is quickly removed from the pavement surface, thereby reducing the possibility of hydroplaning.

Some asphalt concrete pavements exhibit weathering or aging of the surface in the form of raveling without any other defects. On these pavements, the application of fog seals, slurry seals, or chip seals may be an effective solution in areas where oxidation and hardening of the asphalt has occurred. The asphalt application adds softer asphalt material to the oxidized surface of the pavement and retards future increased hardening of the original asphalt surface. The extra material provided by the asphalt reduces the voids on the surface of the pavement and deters the entry of water and air, which would cause further hardening of the asphalt. Properly timed treatments such as fog seals can add two to four years to the service life of these pavement at minimal additional cost.

Surface rehabilitation techniques also provide a marked improvement of a pavement's physical appearance. Pavements with excessive patching or crack sealing may appear quite unattractive. A surface treatment is a simple, effective means of covering these irregularities and restoring a uniform appearance.

Surface rehabilitation techniques also serve to clearly delineate between treated and nontreated areas. When a treatment is applied to a pavement and not to the shoulder, the distinct difference in the visual appearance of the shoulder and the mainline pavement is an aid to motorists. Studies have shown that when this distinction exists, drivers avoid driving on the lane/shoulder joint. This helps increase the life of the pavement. Providing a different appearance and texture for the lane and the shoulder is a simple, but effective safety enhancement.

4. PURPOSE AND APPLICATION

The selection of a surface rehabilitation treatment is typically driven by a single need, such as the need to improve surface friction or to waterproof the pavement surface. However, as noted above, surface rehabilitation techniques can serve many functions. A review of the functions described in the previous section suggests that these treatments, while perhaps applied for one reason, also serve secondary or tertiary needs.

With this in mind, the primary purposes for the use of the various surface rehabilitation treatments are the following:

- Fog Seals are primarily used to seal the surface of asphalt concrete pavements that have begun to ravel from the hardening of the asphalt cement near the pavement surface from age and oxidation. In sealing the surface and reducing the rate of further hardening of the asphalt cement, fog seals have been shown to have prolonged the effective service life of these pavements several years.[18] They have also been used to prevent raveling of chip seals and to seal asphalt concrete pavement paved late in the fall or with excessively high void content. On higher volume routes they have been used to prevent raveling of open-graded friction seals and to maintain shoulders. [1]

- Sand Seals fill an intermediate use between fog seals and full chip seals. They are often used where an asphalt concrete pavement has raveled to the extent where there is quite a bit of fine aggregate missing from the surface. A sand seal is used instead of a fog seal to fill in the lost fine aggregate. They are also used as a pavement preparation treatment to provide a more uniform surface prior to constructing a chip seal and to seal low severity fatigue cracks before constructing an overlay.[19,20]

- Asphalt Chip Seals have been used for many years, on lower volume roads (less than 2,000 vehicles per day), as the wearing surface on untreated granular roadbeds. The Asphalt Institutes Basic Asphalt Emulsion Handbook[7] lists the following uses of chip seals:

  - To provide a low-cost, all-weather surface for light to medium traffic.

  - To provide a waterproof layer to prevent the intrusion of moisture into the underlaying courses.

  - To provide a skid-resistant surface. Pavements that have become slippery because of bleeding or wear and polishing of surface aggregates may be treated with sharp, hard aggregate to restore skid resistance.

  - To give new life to a dry, weathered surface. A pavement that has become weathered to the point where raveling might occur can be restored to useful service by application of a single-surface or multiple-surface treatments.

  - To provide a temporary cover for a new base-course that is to be carried through a winter or for planned stage construction. The surface treatment makes an excellent temporary surface until the final asphalt courses are placed.

  - To salvage old pavements that have deteriorated because of aging, shrinkage cracking, or stress cracking. Although the surface treatment has little or no structural strength, it can serve as an adequate stop-gap measure until a more permanent upgrading can be funded and completed.

  - To define shoulders so they won't be mistaken as traffic lanes.

  - To provide rumble strips for safety.

Chip seals have also been used on higher volume roads (over 10,000 ADT) because of their ability to waterproof the surface, provide low severity crack sealing, and improve surface friction. The possibility of loose chips and traffic disruptions, however, has limited the use of chip seals on higher volume facilities.[1]

- Slurry Seals are used most effectively on pavements where the primary problem is excessive oxidation and hardening of the existing asphalt. They are used for sealing minor surface cracks and voids and retarding surface raveling, particularly on city streets where there are special layer depth constraints in curb and gutter sections. They have been used to improve surface friction characteristics, and also to delineate different pavement surface areas.[1] They have also been used to fill and stabilize open-graded mixes that are raveling excessively.

- Microsurfacing is an enhanced slurry sealing system that is used for much of the same purposes that slurry seals are used, but may provide larger aggregate size and with more construction and curing control may be placed in greater thicknesses.[1] With these attributes they are commonly used for rut filling, minor leveling, and restoration of skid resistant surfaces as well as providing a slurry seal system that can be used on higher volume roadways.

- A Cape Seal, which consists of a chip seal covered with a slurry seal, provides a highly flexible surface treatment like a chip seal but the application of the slurry eliminates any loose cover stones and provides a dense smoother surface that provides a longer service life on higher volume roadways.

- Sandwich Seal which is an application of larger aggregate followed with an application of emulsion and then an application of smaller aggregate provides a little more leveling than a standard chip seal and longer service life than a single chip seal. The service life is more in line with a double chip seal.

- Open-Graded Friction Course have large interconnected voids that allows water to drain through the mix laterally to the side of the road. This rapid removal of water at the pavement surface eliminates the layer of water that can form between the tire and the pavement surface, thus reducing the potential for hydroplaning. It also reduces tire spray and improves visibility in rain compared to visibility in rain on normal dense mixes. Open-Graded Friction Courses also reduce roadway noise depending on the air voids and the maximum stone size at the surface. Results from various studies show that a noise reduction of 3 to 6 decibels can be achieved by using OGFCs.[1]

Traditionally, most surface rehabilitation techniques have been applied on low-volume roadways. Among other factors, this may have been due to an uncertainty about the long-term benefits of surface rehabilitation treatments on high-volume roadways and a fear of vehicular damage, caused by stones dislodged and propelled by traffic. However, recent innovations in this technology have seen some agencies using surface rehabilitation techniques on high-volume roadways. For example, in 1989, a chip seal project was constructed on US 169 in Tulsa, Oklahoma with no complaints of vehicle damage.[4] An excellent discussion of the use of chip seals on high-volume roadways is found in reference 5. In addition, there is a good synthesis of highway practice on OGFC in reference 19.

5. LIMITATIONS AND EFFECTIVENESS

While surface rehabilitation techniques are an effective means of addressing surficial and nonstructural defects in a pavement, it is important to realize that they provide no structural benefit to the existing pavement. That is, the existing pavement is not strengthened or structurally improved through the application of a surface treatments. Therefore, an asphalt pavement exhibiting structural distresses (such as alligator cracking) would not structurally benefit from the application of a surface treatment, although the surface treatment may reduce the rate of deterioration of the pavement by sealing the cracks and reducing water infiltration. In this sense, a surface treatment can be considered to make an indirect contribution to the structural adequacy of the pavement.

Limitations of specific surface rehabilitation techniques include the following:

Worn surfaces should be leveled before applying a chip seal. However, a study in Pennsylvania indicated that the construction and performance of chip seals were not influenced by rut depths of up to 25 mm.[6]

- While OGFC are an effective means of improving the surface drainage of a pavement, there appear to be several disadvantages to their use, including delamination (when placed directly over PCC pavements); rapid formation of reflective cracking and raveling; a higher potential for stripping in the underlying pavement; increased maintenance; and problems with ice control and salting operations. The OGFC is also susceptible to gouging , particularly from snowplow blades, and disintegration from fuel spillage due to the increased void space.

- A slurry seal is an excellent surfacing to seal oxidized and slightly raveling surfaces. A slurry seal will not last as long as a chip seal if the pavement surface is cracked and exhibits excessive deflections under traffic.

- Fog seals, consisting of the spray application of an asphalt emulsion, are being used more frequently as a maintenance activity in the upkeep of shoulders. Because shoulders do not receive the same level of traffic as the mainline pavement, the asphalt concrete is not kneaded and the surface is not sealed as tightly as the mainline pavement. This leads to increased oxidation and hardening of the asphalt cement in the shoulder, resulting in premature raveling and loss of aggregate. With the loss of aggregate, water can penetrate more readily and the shoulder fails more rapidly than the mainline pavement. The extra asphalt material provided by the fog seal counteracts the increased aging and maintains the seal on the surface prolonging the life of the shoulder.

6. DESIGN CONSIDERATIONS

Both chip seals and OGFC require a design procedure to be followed that results in the selection of the proper types and amounts of aggregate and binder. Application rates are provided for fog seals and sand seals, while the design of slurry seals and microsurfacing are typically governed by the material's manufacturer.

Asphalt Chip Seal

There are two components to be considered in the design of a chip seal: asphalt material and aggregate. Unfortunately, there can be a large variation in the properties of these two materials and the results from these variations can be complex and variable. In addition, there are a number of different materials that can be used interchangeably. The materials are not, however, necessarily interchangeable on a one-to-one basis. If the differences are not noted, the design of the surface treatment will be incorrect, and poorer results may likely result. The general design considerations are similar for all types of surface treatments, and are discussed in more detail in this section. Each agency must evaluate the available procedures to ensure they will work with local materials under the prevailing environmental conditions. The considerations listed below should be addressed in the implemented design procedure.

- Existing pavement.
    - Structural adequacy for future traffic.
    - Physical condition of the surface.

- Asphalt type.
  - Asphalt cement.
  - Emulsified asphalt.
  - Cutback asphalt (more limited use due to environmental restrictions).

- Aggregate.
  - Type.
  - Gradation.
  - Quality.
  - Angularity and shape.

- Quantity selection.
  - Residual asphalt content.
  - Aggregate application rate.

- Local conditions and experience.

- General environment.
  - Temperatures before and after placement.
  - Moisture conditions during and after placement (humidity, rain).

Asphalt Type

All asphalt materials such as asphalt cement, cutback asphalt, and emulsified asphalt have been used successfully in the design and construction of surface treatments in the different regions of the country. The selection of one type over the other can not be recommended.

Asphalt Materials

- Asphalt Cement is used to construct chip seals in regions that have very hot weather. The asphalt cement is shot at high temperatures where it flows well and accepts chips readily. A fairly warm pavement surface is required so the asphalt does not cool off too quickly before placement of the chips.

- Cutback Asphalt are blends of asphalt cement and solvents that make the asphalt cement fluid for spraying or mixing then evaporate leaving the base asphalt cement in place to bind the rock. The solvents used are; gasoline for rapid curing, kerosene for medium curing, and diesel fuel for slow curing cutbacks. The rapid cure cutbacks are normally used for single-sized chip seals, while the medium cure cutbacks are used for graded chip seals.

- Emulsified Asphalt is an emulsion of very small asphalt cement particles held in suspension in water with the use of an emulsifying agent. Like cutback asphalts, emulsified asphalts come in rapid, medium, and slow-setting grades for different uses. The rapid, medium, and slow setting grades are developed through the use of different emulsifying agents and the addition of some solvents. The rapid setting emulsions are used mostly for chip sealing, while the medium and slow setting grades are used for the construction of emulsion mixes or recycling and fog or tack seals. The emulsified asphalt “sets” or “breaks” when the asphalt particles precipitate or fall out of the water suspension and coat the aggregates. The emulsion turns color from brown to black during this processes. For rapid setting emulsions, this processes starts on contact with the chips. For medium setting emulsions, this processes starts some time after mixing with the aggregate, depending upon the emulsion and the amount of fines in the aggregate. Slow setting emulsions are very stable and normally break with the evaporation of the water.

The use of cutback asphalts, once commonly used in chip seals, has declined considerably because of environmental restrictions on hydrocarbon emissions from evaporating solvents in specific regions around the country.

Initially, the use of emulsions in place of cutbacks created some problems in the construction of surface treatments because of the unfamiliarity with the use of emulsions. However, these problems have diminished as more agencies have gained experience with emulsions. The amount of emulsion used in constructing a chip seal needs to be higher than the amount of cutback normally used, because of the larger percentage of water present in an emulsion compared to the percentage of solvent present in a cutback. Placement of the chips, right after application of the emulsion, is also much more critical than for cutbacks. If the chips are applied after the emulsion starts to break, the small asphalt particles precipitate on to the old pavement surface rather than the chips, resulting in poor embedment of the chips in the residual asphalt.

The amount of asphalt material applied affects the performance of the chip seal. The function of the asphalt material is to seal the surface and to hold the aggregate in place under traffic. The amount of asphalt material needed is a function of the binder type selected, the surface conditions, and the properties of the aggregate used. The asphalt material must fill the voids between aggregate particles to such a depth that the aggregate will not be displaced under moving wheels. A well-designed chip seal will have a resulting aggregate embedment of 50 to 70 percent, meaning that 50 to 70 percent of the aggregate is held by the binder while 30 to 50 percent is exposed above the binder layer.[1]

Aggregate

The voids in a layer of aggregate vary with the gradation of the aggregate and its maximum size. This is shown in figure 3-5.1, in which the void space in each application is calculated. The smaller the aggregate size, the smaller the void space remaining to be filled by asphalt. Thus, finer gradations require lower asphalt application rates and more control over the application. A variation of plus or minus 0.23 L/m^2 will cause either more bleeding or more rock loss on chip seals using smaller-sized chips compared to chip seals using larger-size chips.

In addition to the maximum aggregate size, the aggregate gradation is important. Generally a one-sized chip is best for seal coat work. However, adequate sources to produce a one-sized stone may not be readily available in some locations and a graded material may need to be used based on the cost of the material. One study in Pennsylvania indicated no differences in performance between chip seals placed using a single-sized stone and chip seals using a graded aggregate.[6] However, the use of dense-graded aggregates for chip seals usually are harder to construct well, and the use of one-sized stone is preferred whenever possible.

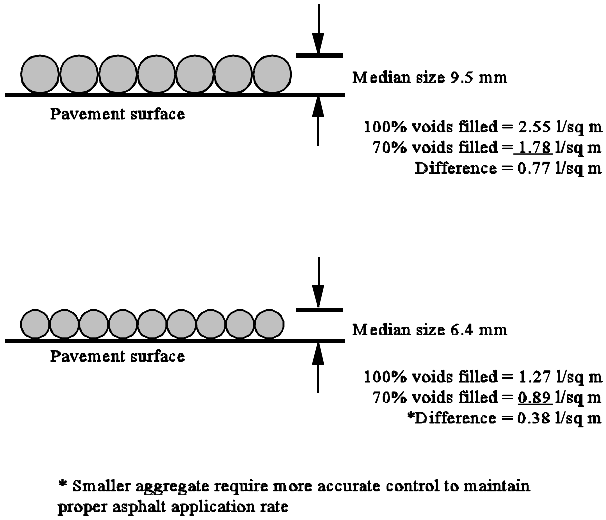


Figure 3-5.1. Voids in surface treatments made with different sized aggregates.

The fines in a graded material will fill the voids in the surface treatment. This reduces the amount of asphalt material required to fill the voids to the prescribed height necessary to hold the aggregate. Gradations with a large amount of fines should not be used because the fines reduce the effective amount of asphalt material to such a low amount that the asphalt is not capable of bonding to the aggregate and maintaining surface sealing integrity. Filling the voids with fines may also result in bleeding during hot periods as the fines tend to combine with the asphalt and act as part of the binder, thus increasing the effective asphalt content when the pavement surface is hot.

Typical gradations for chip seals are shown in table 3-5.1 for several maximum sizes.(7) These are not one-sized materials, but have a gradation curve that must be considered in the design. The design value for the aggregate is generally a function of the median size of the aggregate material, as shown on the graph in figure 3-5.2. For most aggregates meeting their specification values, the median size will not change appreciably with normal variability of the materials. However, problems develop when the aggregate contains large amounts of dust or fines. The gradation and cleanliness must be checked periodically during construction to ensure that a large amount of fines are not allowed to accumulate in the stockpiled material.

The shape of the aggregate is important in the design of the chip seal. This is one reason that the aggregate to be used should be checked before the project begins. If the particles are roughly cubical or round, a given amount of asphalt will coat them to a certain depth, as shown in figure 3-5.3. However, if the aggregate particles are elongated and thin, the same amount of asphalt may more deeply embed or completely submerge the aggregate when it is seated by rolling on its flat side, as also shown in figure 3-5.3.

Quantity Selection

Residual Asphalt Content

When calculating application rates for emulsified asphalts or cutbacks, the total amount of asphalt material to be applied is determined by the residual factor. The residual factor is the percentage of asphalt cement remaining after the other additives (water or solvent) have evaporated.

Table 3-5.1. Typical aggregate for use in chip seals (adapted from reference 7).

Size No.	**Nominal Size**	**Amounts Finer than Each Laboratory Sieve, Weight Percent**						
		25 mm	19 mm	13 mm	10 mm	(No. 4) 4.75 mm	(No. 8) 2.36 mm	(No. 16) 1.18 mm
6	19 mm to 10 mm	100	90 to 100	20 to 55	0 to 15	0 to 5		
67	19 mm to (No. 4) 4.75 mm	100	90 to 100		20 to 55	0 to 10	0 to 5	
68	19 mm to (No. 8) 2.36 mm	100	90 to 100		30 to 65	5 to 25	0 to 10	0 to 5
7	13 mm to (No. 4) 4.75 mm		100	90 to 100	40 to 70	0 to 15	0 to 5	
78	13 mm to (No. 8) 2.36 mm		100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5
8	10 mm to (No. 8) 2.36 mm			100	85 to 100	10 to 30	0 to 10	0 to 5

When asphalt cement is used to construct the chip seal, the residual factor is 1.0 because there are no additives present to evaporate, and all the material applied is effective in holding the aggregate in place. When a cutback is used, there is a certain amount of solvent that will evaporate from the cutback, leaving a reduced amount of asphalt cement. For typical cutbacks used in chip sealing, this residual factor ranges from 65 percent to near 80 percent. In an emulsion, the water will evaporate along with the small amount of solvent allowed in some formulations. For most emulsions, the residual factor ranges from 55 to 65 percent.

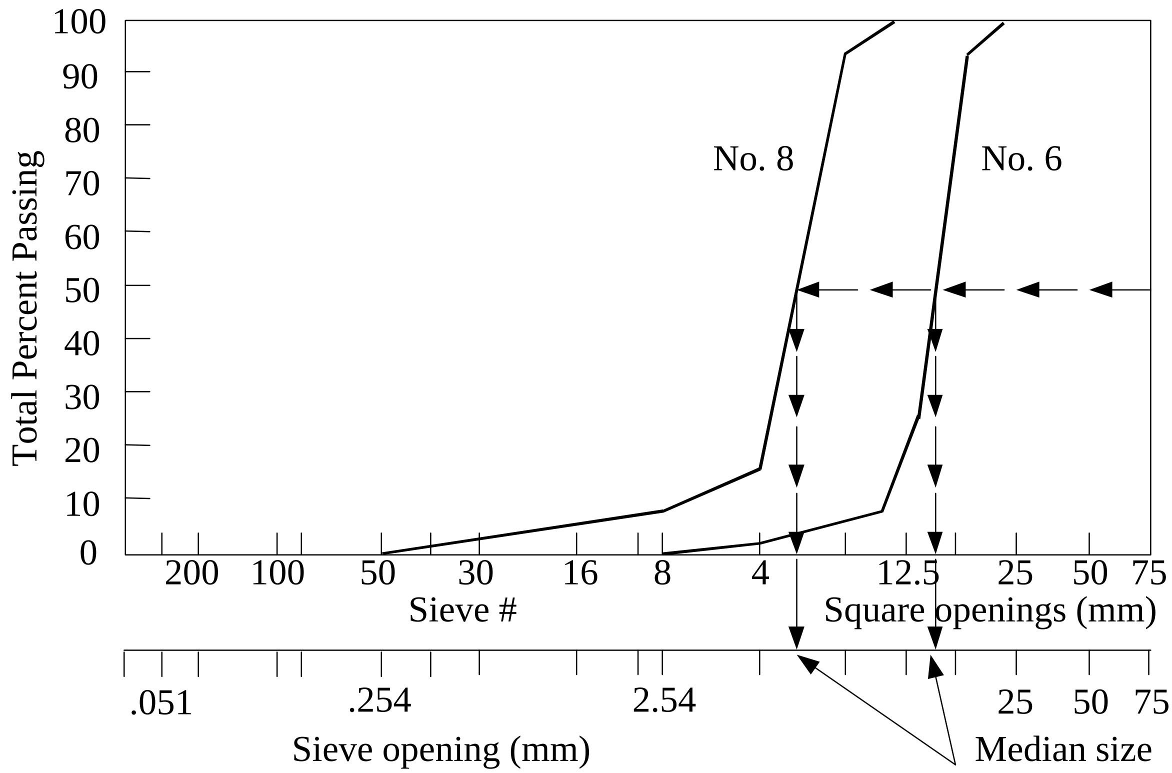


Figure 3-5.2. Typical gradations with median sizes indicated.

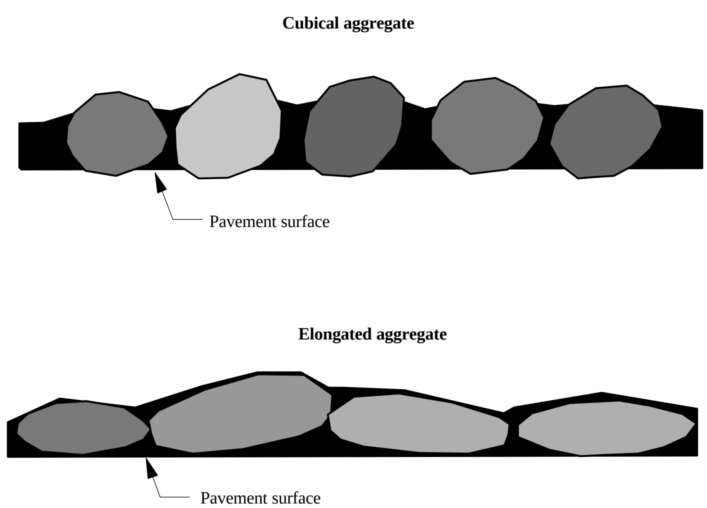


Figure 3-5.3. Difference in coating of cubical and elongated aggregate for same amount of asphalt.

It should be apparent that there is a large difference in the amount of material remaining after evaporation when similar amounts of cutback or asphalt emulsion are applied initially. The design procedure used must recognize these differences. With any given aggregate size, there will be a difference between the necessary application rate for a cutback and the application rate for an emulsion to get the same residual asphalt and chip embedment. Emulsions generally have a lower residual ratio and, thus require a higher application rate to get the same residual.

The aggregate and asphalt material properties together dictate the amount of residual asphalt required. The gradation of the aggregate has a large effect on the amount of asphalt, as does the aggregate's cleanliness. The initial design, for estimation purposes, may be made using the specification gradation for the aggregate. However, before construction, the aggregate should be sampled from the stockpiles and subjected to laboratory testing to determine the actual gradation, and the design should be refined accordingly. Proper asphalt application is the single most important variable in a successful chip seal application.

Aggregate Application Rate

The amount of aggregate is specified in kg/m^2, and, for design purposes, is assumed to contain a certain amount of embedment when rolled and seated by traffic over time. The asphalt must cover 50 to 70 percent of the depth of the aggregate to hold it properly.[18] The values are based on embedment of the aggregate particles after rolling. Additional seating and alignment of the chips will occur over time from traffic. This will increases the embedment of the chips.

The general range in chip embedment in the asphalt is depicted in figure 3-5.4, which shows in a very general way how the chips are embedded during placement, rolling, and latter compaction by traffic. If asphalt is added to completely fill the voids in the aggregate after rolling, there will be too much asphalt after traffic has compacted the aggregate, resulting in bleeding or flushing. For more detailed design equations to determine aggregate and asphalt cement quantities, the Asphalt Institute publications should be consulted.[9]

Light traffic will produce very little additional compaction after construction and the amount of asphalt needed to embed the aggregate after rolling will be higher. In this case the amount of residual asphalt should produce about 70 percent embedment of the stone after construction. On the other hand, heavy traffic will provide much more compaction after construction, and the amount of asphalt required to embed the chips should be less. In this latter case, the amount of asphalt should be applied to produce about 50 percent embedment of the stone after construction.

Typical aggregate and asphalt application rates for single and double chip seals are provided in table 3-5.2 and table 3-5.3, respectively.[7] These values are for information only; actual project values will vary depending upon local conditions.

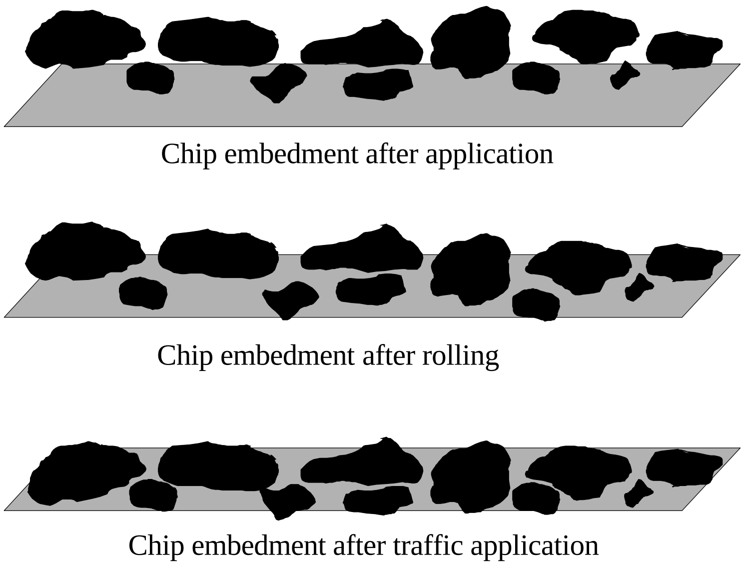


Figure 3-5.4. Chip embedment with construction sequences.

Table 3-5.2. Typical quantities of asphalt and aggregate for single chip seals.[7]

Nominal Size of Aggregate	**Size No.**	**Quantity of Aggregate, kg/m²**	**Quantity of Asphalt, l/m²**	**Type and Grade of Asphalt**
19 mm to 9.5 mm	6	21-27	1.8-2.3	RS-2, CRS-2
12.7 mm to (No. 4) 4.75 mm	7	14-16	1.4-2.0	RS-1, RS-2 CRS-1, CRS-2
9.5 mm to (No. 4) 2.36 mm	8	11-14	0.90-1.6	RS-1, RS-2 CRS-1, CRS-2
4.75 mm (No. 4) to 1.18 mm (No. 16)	9	8-11	0.70-0.90	RS-1, MS-1 CRS-1, HFMS-1
Sand	AASHTO M-6	5-8	0.45-0.70	RS-1, MS-1 CRS-1, HFMS-1

Notes: 1. The lower application rates of asphalt shown in the above table should be used for aggregate gradations on the fine side of the specified limits. The higher application rates should be used for aggregate having gradations on the coarse side of the specified limits.

2. The mass (weight) of aggregate shown in the table is based on aggregate with a specific gravity of 2.65. In case the specific gravity of the aggregate used is lower than 2.55 or higher

than 2.75, the amount shown in the table above should be multiplied by the ratio that the bulk specific gravity of the aggregate used bears to 2.65.

3. It is important to adjust the asphalt content for the condition of the road, increasing it if the road is absorbent, badly cracked, or coarse, and decreasing it if the road is “fat” with flushed asphalt.

Pavement Texture		Correction (l/m²)
Black, flushed asphalt		-0.05 to -0.27
Smooth, nonporous		0.00
Absorbent	C slightly porous, oxidized	0.14
	C slightly pocked, porous, oxidized	0.27
	C badly pocked, porous, oxidized	0.41

It is important to recognize the need for the asphalt quantity corrections noted at the bottom of table 3-5.2. These values are only suggestions and should be modified to reflect local experience and conditions. They are needed because the physical condition of the surface influences the amount of asphalt material needed in the chip seal. If the surface is flushed or bleeding, the amount of asphalt should be reduced to compensate for the excess already present. If the surface is oxidized and very porous, the amount used should be increased because the surface will absorb some of the asphalt, effectively reducing the asphalt available to coat the aggregate in the chip seal. When an emulsion has been applied at too low a rate, and the chip embedment is less than 50 percent, additional asphalt can be added by applying a fog seal to provide adequate embedment and minimize chip loss. The fog seal should consist of an SS-1 or CSS-1 diluted 50 percent to aid the flow of the emulsion around the chips.

Table 3-5.3. Typical quantities of asphalt and aggregate for double chip seals.[7]

	Nominal Size of Aggregate	Size No.	Quantity of Aggregate, kg/m²	Quantity of Asphalt, l/m²
12.7 mm Thick 1st Application 2nd Application	9.5 mm to (No. 8) 2.36 mm 4.75 mm (No. 4) to 1.18 mm (No. 16)	8 9	13.6-19.0 5.4-8.1	0.91-1.36 0.30-0.40
15.9 mm Thick 1st Application 2nd Application	12.7 mm to 4.75 mm 4.75 mm to 1.18 mm	7 9	16.3-21.7 8.1-10.9	1.36-1.81 1.81-2.27
19 mm Thick 1st Application 2nd Application	19 mm to 9.5 mm 9.5 mm to 2.36 mm (No.8)	6 8	21.7-24.4 10.9-13.6	1.59-2.27 2.27-2.72

Open-Graded Friction Courses

Similarly to the design of chip seals, the design of OGFC must consider both the aggregate and binder components. OGFC mixes are produced at an asphalt mix facility in the same way as conventional hot-mixed asphalt concrete, with the primary difference between the two being the gradation of the aggregate. There is also a significant difference in appearance [OGFC appears too rich in comparison to hot-mix asphalt (HMC) concrete] and in the mixing and placement temperature (OGFC placed at normal hot-mix, temperatures will experience drainage of the asphalt cement, causing thinner asphalt film thicknesses and early raveling of the mix).

Materials for OGFC

Aggregate

For best performance of an OGFC, the coarse aggregate should be completely crushed (more than a single-crushed face), have good abrasion resistance, and have a high resistance to polishing. Traprock and slag are particularly well-suited aggregates.

The drainage characteristics of an OGFC will not function properly unless a large percentage of voids are present in the mix. Since the amount of air voids are controlled primarily by the gradation of the aggregate, a special gap gradation of the aggregate is needed, with a large reduction in the amount of aggregate in the middle fractions. However, a certain amount of fine aggregate is needed in an OGFC mix. These fines provide a necessary keying action for the larger aggregate and increase the overall strength of the mix. Mineral filler is also needed to bulk the asphalt films and increase the stability of the mix. Usually a minimum of two to five percent material passing the 75 µm is used. Figure 3-5.5 illustrates a typical gradation for OGFC.[10]

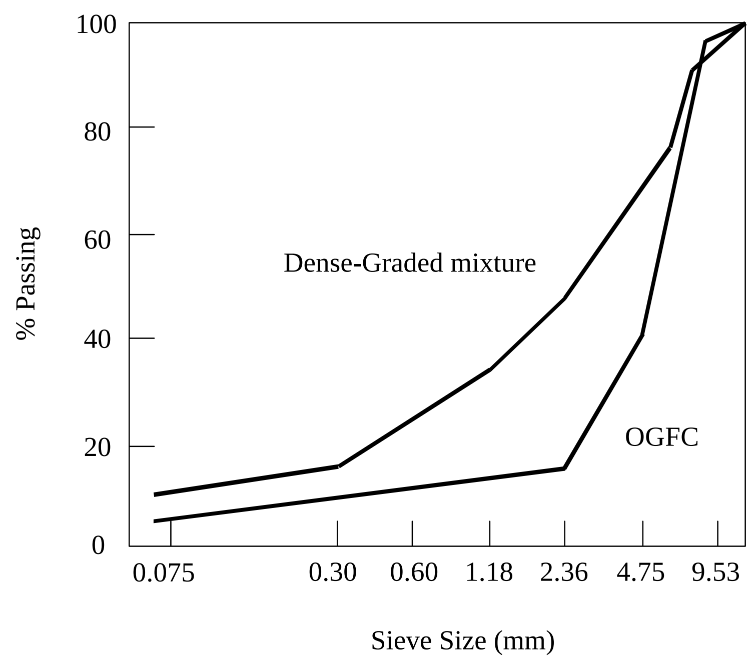


Figure 3-5.5. Typical gradations for open-graded friction courses.[10]

The recommended gradation for a 12.7 mm maximum size aggregate used in an OGFC is given below:[10]

Sieve Size	Percent Passing
13 mm	100
10 mm	95 - 100
4.75 mm (No. 4)	30 - 50
2.36 mm (No. 8)	5 - 15
75 µm (No. 200)	2- 5

Binder

Asphalt cement grades AC-10, AC-20, AC-40 (85-100, 60-70, and 40-50 pen, respectively) and AR-4000 and AR-8000 have been recommended as binders for OGFC.[10,11] Some agencies prefer the softer grades, believing that the binder will harden rapidly because the air and water circulate freely through the mix. Other agencies, even some of those in the northern parts of the United States, prefer the harder grade, AC-20 (85-100 pen), especially when the OGFC is to be used in high traffic volume locations to minimize scuffing and tearing of the new mat.

The harder binders have greater stiffness and produce a thicker film on the aggregate. In Europe, modified binders have been used to help maintain thicker films and still provide the softer base asphalt properties desired.[2] The use of modified binders has increased in North America for the same reasons. Rubber/asphalt binders have also been used quite successfully in OGFC. Because of the high void content of this type of construction, it is essential that there be thicker films of asphalt on the aggregate. This thicker film is required to produce a stable and mix to minimize early raveling of the OGFC.

Some agencies have found that an OGFC are prone to asphalt stripping problems and perform much better when the mix contains an anti-stripping agent. Since an OGFC is subjected to greater adverse effects of water and air during its life than a normal mix, the anti-stripping agent should reduce raveling, and strengthen binder adhesion. Usually a heat-stable, anti-stripping agent is added to the asphalt at the mixing plant at a rate indicated from stripping tests.

Design Procedures

Various procedures have been used to design an OGFC, including an FHWA procedure (reference 8) and a procedure developed by the Asphalt Institute.[7] The FHWA procedure is more commonly used, and consists of the following five steps:[10]

- The asphalt content is initially selected, based upon the kerosene-equivalent value of the coarse aggregate.

- After compacting and determining the vibrated unit weight of the trial mix, voids in the coarse aggregate are computed.

- The amount of fine aggregate needed [percent passing 2.36 mm (no. 8) sieve] is computed, so that the resulting compacted mixture will have a design percent air void of at least 15 percent.

- The optimum mix temperature is selected to produce a viscosity of 0.0008 m/s^2. Aggregate and binder are mixed at this temperature and checked to determine that the excess binder does not drain off the aggregate.

- The resistance of the OGFC to the effects of water is determined by a modified immersion-compression test. After 4 days of immersion in water, the retained strength should be not less than 50 percent. When the mix does not meet this water resistance requirement, anti-stripping agents should be added to the binder and the test rerun to ensure compliance with the 50 percent criterion.

The FHWA procedure determines the optimum binder content solely as a function of the coarse aggregate absorption. A study by one agency suggests that the design procedure must be modified to account for the effects of binder viscosity and mixture modifications (e.g., anti-stripping agents) on the identification of the optimum binder content.[12]

Slurry Seal

Depending upon the application, there are three different grades of slurry seals that can be used (see table 3-5.4).[7,13] Type I is used on lower volume roadways for maximum crack penetration and also makes an excellent pretreatment for an overlay or chip seal. Type II, the most commonly used variety, is coarser than Type I and can be used on moderate to heavy volume roadways to seal the surface and improve surface friction. Type III is used on pavements where surface irregularities require a thicker sealer with a larger aggregate. It is also recommended for use as a first course in a multi-course application and to restore surface friction.

Microsurfacing

Microsurfacing consists of a mixture of polymer-modified emulsified asphalt, mineral aggregate, mineral filler, water, and additives. Most State specifications require that the mixture include 82 to 90 percent aggregate. The amounts of other constituents as a percentage of the dry aggregate generally are:[1,8]

- 1.5 percent to 3.0 percent portland cement as a mineral filler.
- 5.5 percent to 9.5 percent residual asphalt.[8]

Table 3-5.4. Types of slurry seals.[7, 3]

	TYPE OF SLURRY		
	I	**II**	**III**
General Usage	Crack filling and fine seal	General Seal, medium-textured surfaces	1st or 2nd application, two-course slurry, highly-textured surfaces
Gradation			
Sieve Size	Percent Passing	Percent Passing	Percent Passing
9.5 mm	100	100	100
4.75 mm	100	90-100	70-90
2.36 mm	90-100	65-90	45-70
1.18 mm	65-90	45-70	28-50
600 µm	40-65	30-50	19-34
300 µm	25-42	18-30	12-25
150 µm	15-30	10-21	7-18
75 µm	10-20	5-15	5-15
Residual Asphalt Content, % Weight of Dry Aggregate	10-16	7.5-13.5	6.5-12
Application Rate, kg/m^2	3.3-5.4	5.4-8.1	8.1

Various gradations have been used in microsurfacing, typically containing less than 10- to 13-mm top size aggregate.[3,14] Aggregate gradations used by different States normally follow ISSA recommendations for slurry types II and III with minor variations. Trial mix designs should be formulated to determine the optimum combination of the component materials.

Fog Seals

A slow-setting asphalt emulsion is typically used for fog seals. It is diluted with water in varying proportions, but in most cases a one to one dilution is used.[7] Typical application rates for fog seals range from 0.45 l/m^2 to 0.68 l/m^2, although exact quantities are determined by the pavement surface texture, the pavement dryness, and the amount of cracking.[7]

Sand Seals

Sand seals are a spray application of asphalt emulsion followed by a light covering of fine aggregate, such as clean sand. Typical application rates for the rapid set emulsion generally used for sand seals vary between 0.68 l/m^2 and 0.91 l/m^2, while the aggregate application rate varies between 5.4 kg/m^2 and 8.1 kg/m^2.[7]

7. PAVEMENT SURVEYS

If the existing pavement is not structurally adequate to carry the projected traffic for the next three to five years, surface rehabilitation techniques such as OGFC, slurry seals, and microsurfacing should not be considered. Similarly, pavements with structural problems, resulting from poor drainage or an unstable base are not candidates for these surface rehabilitation techniques. These deficiencies should be noted during the survey and evaluation phase of project development and the pavement scheduled for more comprehensive rehabilitation. The more flexible surface rehabilitation techniques such as single-chip and multiple-chip seals can be used over structurally deficient pavement sections within limits and provide reasonable service life.

The engineer also needs to consider the presence and width of cracks in the pavement. If cracks are too wide or are experiencing excessive movement, they will rapidly reflect through any of the surface rehabilitation treatments.

As mentioned under the discussion of chip seals, the surface texture of the pavement must be considered, since it will affect the amount of asphalt material required in the treatment. The concepts discussed for adjusting the amount of asphalt for chip seals also can be applied to other surface rehabilitation treatments.

8. COST CONSIDERATIONS

While typical costs for the various surface rehabilitation techniques will depend on many factors, including the type of binder, type of aggregate, and the application rates, some typical costs for several of the surface rehabilitation treatments mentioned are provided below:[1]

- Chip Seal — \$0.90/m^2
- Double Chip Seal — \$1.35/m^2
- OGFC — \$2.45/m^2

- Rubberized Chip Seal — \$1.80/m^2
- Slurry Seal — \$0.84 to \$1.14/m^2
- Microsurfacing — \$1.05 to \$2.00/m^2
- Fog Seal — \$0.12 to \$0.17/m^2
- Sand Seal — \$0.30 to \$0.48/m^2
- Cape Seal — \$1.70 to \$2.40/m^2
- Sandwich Seal — \$1.20 to \$1.30/m^2

9. CONSTRUCTION CONSIDERATIONS

Typical Construction Problems

Several agencies have reported problems in the construction of one or more of these surface rehabilitation techniques. However, many of these problems could have easily been avoided through improved construction techniques and reasonable quality control processes.

Chip Seals

The more common problems associated with chip sealing are streaking of the asphalt, excessive aggregate loss, and bleeding. Possible causes for these problems are listed below.[20]

Chip Seal Problems and Causes

Loss of Cover Aggregate	Streaking	Bleeding
Emulsion application rate too low Open porous surface Near empty distributor (<380 l) Dry dusty aggregate Excess fines in aggregate Delay in applying aggregate Delay in rolling aggregate Use of steel wheel roller Roller speed to fast (>2.2 m/s) Cool weather Cool pavement surface (>15.5° C) Wet pavement surface Rain during or after application Late season application Fast traffic too soon	Spray bar height incorrect Spray bar rising as load lightens Incorrect nozzle angle Incorrect nozzle size Worn nozzles Incorrect pump speed Pump pressure too high Pump pressure too low Asphalt emulsion too cool Near empty distributor (<380 l) Nonuniform aggregate application	Asphalt emulsion rate too high Loss of cove aggregate Aggregate application too low Asphalt rich existing surface Excess asphalt emulsion in ruts

Careful monitoring and quality control procedures during the construction and application of the surface rehabilitation technique can eliminate these problems.

Slurry Seals and Microsurfacing

The more common problems associated with slurry seals and microsurfacing are flushing, raveling, and delamination or pot holes. Possible causes for these problems are listed below.[8]

Slurry Problems and Causes

Flushing	Raveling	Delamination & Pot Holes
Early opening to traffic Excess binder in mixture Excess water in mixture Hot weather Single application in deep ruts	Deficient asphalt content Inadequate aggregate gradation Application to thin Emulsion quality Insufficient water	Inadequate surface preparation - patching - cleaning Placed over dry porous surface Placed on PCC pavement with-out tack

Open-Graded Friction Courses

OGFC usually fail by raveling. Early failure from premature raveling is usually caused from stripping of the asphalt cement from the aggregate or early oxidation and hardening of the asphalt cement due to construction or design procedures that resulted in thin asphalt cement films.[19] In some cases, premature raveling has been associated with too light or no application of tack.

OGFC have also been shown to accelerate the stripping of underlying pavement when placed over existing pavements which are prone to stripping. [19] The existing pavement should be checked for moisture sensitivity in the rehabilitation design procedure. OGFC should not be used over existing pavements that are moisture sensitive.

Key Inspection Points

The construction or application of a surface rehabilitation technique may begin only after the materials and equipment have been thoroughly inspected. The project must be closely monitored and controlled to ensure that a quality treatment is constructed. The key inspection points during construction include the following:

- Surface preparation.

- Calibration of the application equipment.
    - Asphalt distributor.
    - Aggregate spreader.

- Control and determination of quantity during application.

- Timeliness of the construction sequence (for treatments with aggregate).
    - Application of asphalt.
    - Application of aggregate.
    - Initiation and completion of rolling.
    - Sweeping after construction.

- Continual quality control of the materials used.

Surface Preparation

The surface must be free of loose debris and dirt and must not have been disturbed by construction traffic. A wet surface will not allow proper adhesion of asphalt cement or cutback-based surface rehabilitation treatments. However, some minor surface moisture (not wet) is permissible for asphalt emulsion applications. For a chip seal placed as a surface on a newly constructed pavement a fog seal should be applied to the pavement to minimize chip loss. A prime coat should be applied prior to application of the chip seal over a untreated surface.

Calibration and Monitoring of Application Equipment

Some of the surface rehabilitation treatments employ only the application of asphalt, while others require the application of both asphalt and aggregate. In either case, since the asphalt and aggregate application rates are the most critical factors governing the performance of the surface rehabilitation technique, maximum effort must be put forth to ensure that both the distributor and the chip spreader are properly calibrated.[6] Calibration procedures are described in the next section. After calibration, the application rates of the asphalt emulsion and aggregate must be closely monitored and continually checked during actual construction to ensure that the correct quantities of asphalt and aggregate are being applied. The actual yield quantity for each aggregate truck load and continuous asphalt emulsion shot should be computed and recorded, continuously throughout the project.

Use of Precoated Aggregate

Precoated aggregate can be used in seal coats and surface treatments. The use of precoated aggregates is an effective means of reducing aggregate loss.[16] It can be particularly effective when using aggregate, which can not be washed effectively and/or when using a rubber asphalt or hot-applied asphalt cement binder.

10. EQUIPMENT FOR SURFACE REHABILITATION TREATMENTS

Asphalt Distributor

The asphalt distributor is the first and perhaps the most critical element in the construction of chip seals. Many of the problems associated with the construction of chip seals can be traced back to improper asphalt application. The distributor consists of a truck-mounted insulated tank with a system of spray bars and nozzles at the back of the tank to apply a uniform application of asphalt. The tank has a heater and a circulation system that heats and circulates the asphalt in the tank and a supply system that transports the asphalt from the tank to the spray bar. The amount of asphalt that is transported to the spray bar and applied to the road is controlled through a valve system, an asphalt pump with pump speed or pressure controls and the nozzle size. These are tied together with a distance and speed measuring system to set application rates.

Before a job is begun, the equipment should be calibrated to ensure that the manufacturer's settings provide the quantities of material specified in the manufacture’s literature. The contractor should be required to demonstrate that this has been done. It should never be assumed that the published settings for pump speed and vehicle speed are going to provide the application rate (l/min) stated in the equipment manual. The actual output will vary with equipment age and type of material applied and its viscosity (manufacture and temperature dependent). For calibration, the distributor nozzles may be placed over a pan of known size, and the asphalt to be used should be sprayed into the pan for a specified time. The amount of asphalt collected in the pan should be measured and compared with the amount predicted by the specified setting. Any differences should be noted, recorded, and used to adjust the settings in the field.

This procedure ensures that the equipment is functioning properly; however, it does not ensure that the required amount of asphalt will be sprayed onto the pavement. A field calibration must be performed and checked at regular intervals. An initial section of roadway should be designated as a test strip prior to construction. The distributor should be weighed full and the proper pump settings made. The asphalt should then be applied to the premeasured area at the pump settings to provide the desire coverage and the actual yield computed and compared to that required. If the amounts do not agree, the settings on the distributor should be adjusted, and another test strip placed if a large adjustment is called for. The chips should be placed and rolled as planned for normal construction.

A pad of cotton (0.61- by 0.61-m) or some other appropriate material may be placed at different locations within the test strip to determine the coverage both longitudinally and transversely on the pavement. The pads should be weighed before and immediately after the distributor passes over them, to prevent evaporation of solvent from altering the weights on the pads. The weight of asphalt on the pad and the area of the pad are used to compute the coverage of asphalt in l/m^2.

An alternative means of determining the asphalt application rate is by using the procedure outlined in ASTM D 2995. However, this procedure is tedious and more time is required to obtain the test results.[17]

Other variables to be observed during the test strip construction include the spray bar height and the nozzle settings. The height of the spray bars should be set such that the required amount of asphalt is applied in a uniform manner without streaking; the height of the spray bar controls the amount of overlap, as illustrated in figure 3-5.6. The best results are usually achieved with double coverage, although triple coverage can sometimes be used for spray bars with nozzles spaced at 100 mm intervals.[7] Spray fans set at a higher height setting may be susceptible to wind effects.

The angle of the nozzle will control the uniformity of the spray pattern, as indicated in figure 3-5.7. These should be monitored during the test strip, and continually during construction. Generally, the angle should be set between 15° and 30°.

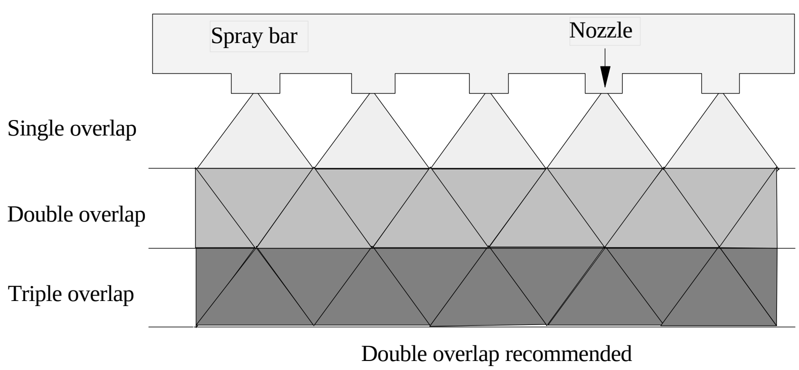


Figure 3-5.6. Spray bar height to establish proper overlap.

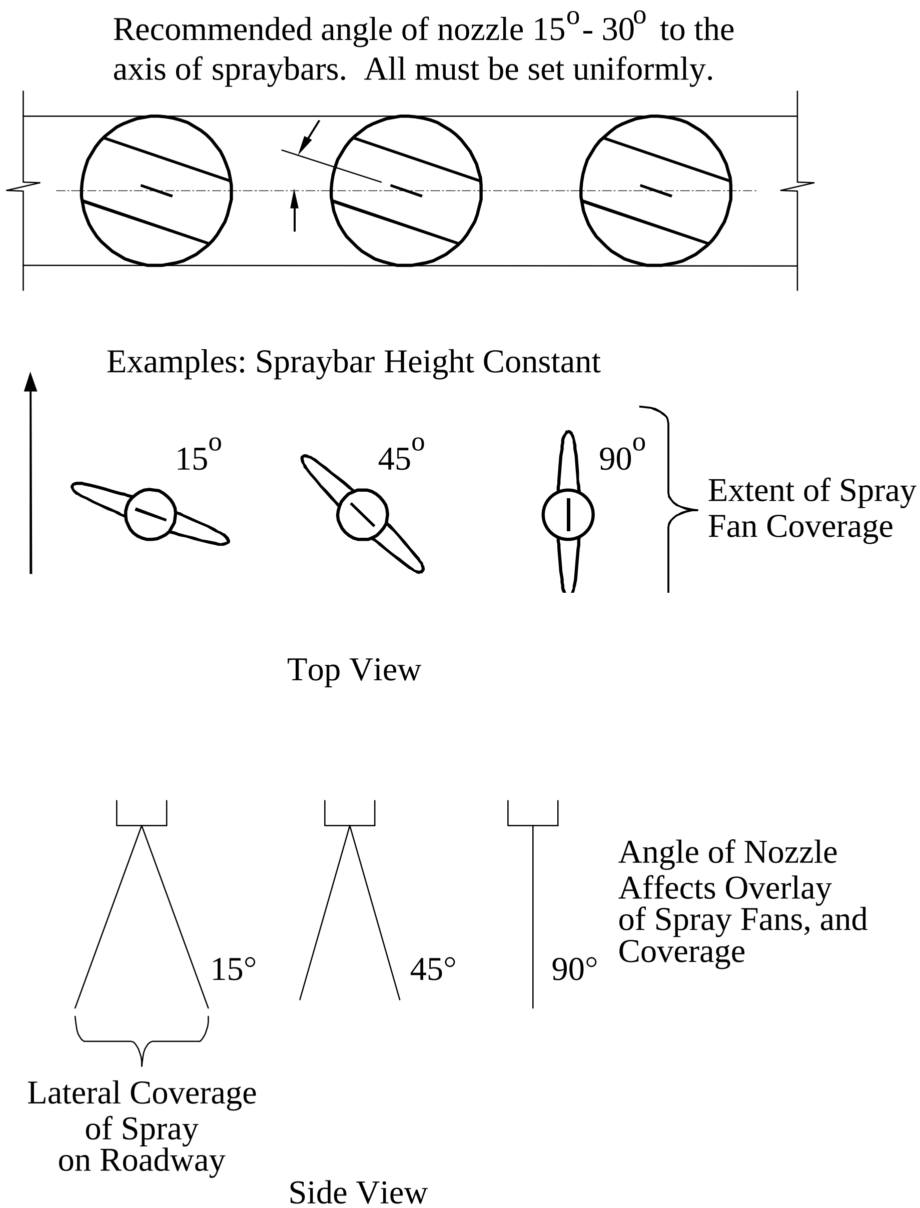


Figure 3-5.7. Angle of spray nozzle and angle of overlap.

Aggregate Spreader

The aggregate spreader is the second most important equipment used in constructing chip seals. Aggregate spreaders vary from simple vane types that attach to truck tail gates to very efficient self-propelled types. Tail gate type equipment should only be used for small isolated work like maintenance patching. A modern self-propelled mechanical aggregate spreader should be used for all chip seals used on pavement surface rehabilitation projects. A self-propelled mechanical spreader pneumatic tiredmotorized unit with a hopper on the front where the chips are dumped which transported to the back where a specialized gate system drops the chips uniformly across the pavement. This equipment also includes a screen on the hopper to reject oversized rock, individually controlled gates that allow varying rock application rates across the pavement, and a system using sloped screens that can separate out the larger chips and drop them ahead of the smaller chips.

The aggregate spreader should be calibrated in a manner similar to the distributor during construction of the test strip. Pans of a specified size should be placed in front of the spreader at regular intervals across the pavement. The aggregate placed on the pans by the spreader should be weighed to calculate the application rate (kg/m^2) as follows:

$$RATE = (WT)/(L \times W) \qquad (3\text{-}5.1)$$

where:

WT	=	weight of aggregate on pan, kg
L	=	length of pan, m^2
W	=	width of pan, m^2

The actual application rate is computed and compared to that required. If the amounts do not agree, the settings on the chip spreader should be adjusted. If a significant adjustment is required, another test strip should be constructed and the processes repeated. Chip embedment should also be checked at this time to confirm design application rates.

Special Equipment

Slurry Seal Equipment

The machine used for production and application of a slurry seal is a self-contained, continuous-flow mixing unit.(7) It contains a traveling mixing plant for the proportioning and mixing of the slurry seal that, after mixing, is spread on the pavement surface with an attached spreader box. The machine is capable of accurately delivering to the mixing chamber the predetermined amounts of aggregate, mineral filler (if required), water, and asphalt emulsion.(7) A schematic drawing of a typical slurry seal mixer/spreader is shown in figure 3-5.8.(7)

Microsurfacing Equipment

Microsurfacing equipment generally consists of latex-modified asphalt emulsion, aggregate, and additives being transported in a traveling mixing plant similar to, but slightly larger than, a regular slurry seal machine with a more powerful and faster mixer and greater control on material and feed rates.(3) Several different adjustable spreader boxes are available that can be used, for either full-width paving 2.4- to 4.3-m or for rut filling 1.5- to 1.8-m.(14)

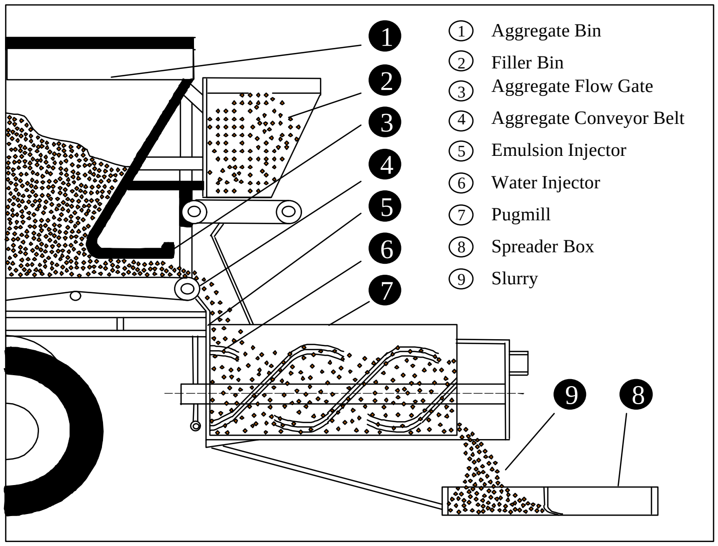


Figure 3-5.8. Schematic drawing of typical slurry seal mixer/spreader.[7]

11. CONSTRUCTION PROCEDURES

Construction and application procedures are provided for the more common surface rehabilitation treatments. Included are chip seal construction, OGFC construction, slurry seal construction, microsurfacing construction, fog seal applications, and sand seal applications.

Chip Seal Construction

The construction sequence for most chip seal projects is as follows:

- Calibration of asphalt distributor.
- Calibration of aggregate spreader.
- Cleaning of the existing pavement surface.
- Application of asphalt at desired rate.
- Application of aggregate at desired rate (prior to emulsion or cutback breaking).
- Rolling of aggregate (immediately after aggregate spreader).
- Curing of binder (will vary depending on effects of temperature and relative humidity on breaking of emulsion or cutback materials).
- Brooming of loose aggregate (after binder is cured, with time frame ranging from 15 hours to 24 hours).

The existing pavement surface should be dry, unless asphalt emulsions are used in the chip seal. The work should be conducted in warm weather, and should not be attempted when air temperatures are below 16^{E}C.

Rolling begins immediately after the aggregate has been spread. Typically, 4.5 metric ton pneumatic-tired rollers are used, although rollers with weights of 2.7 metric tons to 7.3 metric tons are also used. Steel-wheeled rollers are not generally used because they may crush the aggregate and have a tendency to bridge over low spots so that no compaction occurs in those areas.

A sufficient number of rollers must be available to ensure full coverage of the application before the asphalt sets or hardens. Once the asphalt begins to harden, the aggregate cannot be adequately seated into the asphalt. Failure to achieve this seating can lead to the aggregate being pulled out by traffic. The maximum amount of rolling should be determined by economics, while the minimum amount should be no fewer than three passes. One study indicated that one pass of an 7.3 metric ton roller was sufficient for proper seating of the aggregate.[6]

The total operation of asphalt application, chip spreading, and rolling, must always be completed as quickly as possible once it begins. If delays occur, the asphalt will set or cure and this will prevent the rolling from effectively seating the aggregate, which will cause chip loss. Ideally, the emulsion should begin to break just after the first roller pass has been made.

Typically, the seal coated surface can be opened to traffic in about two hours. Earlier opening times may be possible if vehicle speeds are limited to below 40 km/hr.

Newly-constructed chip seals that are starting to loose aggregate because of inadequate embedment from low asphalt application rates can be corrected somewhat with an application of a fog seal of CSS1 diluted 50 percent with water.

It should be noted that chip seals can be constructed using asphalt cement, and in fact some contend that the use of asphalt cement results in the best performing seal coat in warm climates. However, asphalt cement can only be used on hot days (greater than 27^5C) during the summer, and requires the immediate application of chips and rolling before the asphalt cement cools.

OGFC Construction

The construction procedures for an OGFC are different than those for dense-graded asphalt concrete. Butt joints should be used in lieu of slope or lap joints because of the mixes' resistance to lateral movement. The rolling should consist of one pass or two passes of a medium weight (7.3 metric ton to 9.1 metric ton) static steel-wheeled roller. Compaction should follow immediately behind the paving operation.

The OGFC should be placed only on a structurally sound surface, free of ruts and a large amount of cracking. Construction should only take place during good weather, when the temperature of the underlying pavement will be at least 15°C.

A tack coat is needed prior to the placement of the OGFC, regardless of the surface on which the OGFC is placed. Strict gradation control is required to ensure that adequate air voids are obtained. The most critical element is the temperature control in the mixture. The viscosity of the asphalt cement binder must not be allowed to drop below the point where the binder runs off the aggregate and reduces the coating thickness. This necessitates lower mix temperatures than those typically used with other hot mixes in the range of 110°C to 120°C which produce an asphalt viscosity of 800 centistokes.

The laydown of the OGFC uses conventional asphalt paving equipment. The paving operation can proceed at a faster pace than conventional paving operations, although excessive speeds should be avoided to prevent tearing of the mix.[10]

The rolling of the OGFC is generally limited to one or two passes of an 7.3 metric ton to 9.1 metric ton static, steel wheel roller. Pneumatic rollers should not be used because they tend to track the asphalt and they close the voids excessively.[15] It is important that the rolling operations begin as soon as possible after placement because the OGFC loses heat quickly. The OGFC may be opened to traffic as soon as it has cooled.

Because of the high cost of OGFC, the thickness of the OGFC layer is usually kept to a minimum.(1) A 16- to 19-mm thick layer usually can be placed satisfactorily. Thinner layers may produce raveling and wearing away of the mix, leaving bare spots in the surface.

Slurry Seal Construction

The same surface preparation must be performed for a slurry seal as for the other types of surface rehabilitation treatments. The surface must be clean of dust, and a light tack coat of similar emulsion should be applied to worn or oxidized surfaces. For newer surfaces, a light spray of water to dampen the surface may be sufficient. The emulsion and aggregate are mixed in a self-powered vehicle and spread over the pavement surface to a thickness of 3.2- to 6.4-mm.(1)

It is generally not necessary to roll a slurry seal except in areas that have been subjected to abrasion from steering or breaking forces.(1) Pneumatic rollers (4.5 metric ton) may be used for this compaction effort, but they must not be allowed on the slurry until the emulsion has broken.

Traffic should be kept off the pavement until the seal is sufficiently cured. Typically, this will range from 2 hours in warm weather to six to 12 hours in cooler weather.

Microsurfacing Construction

Microsurfacing operations should only be performed in situations where the road surface temperature is 10 °C and rising, and there is no fog or rain expected.(1) Tack coats are generally not required prior to microsurfacing, unless the surface is dry and raveled or PCC. If rut filling is performed, there should be at least one layer placed full-width over the filled ruts to provide uniform surface friction over the entire lane.

The material is placed using a modified-traveling plant mix. For surface retexturing, an adjustable width spreader box is used; for rut-filling activities, the favored method of placement is to use a drag box only slightly wider than the rut.(14) Ruts greater than 25 mm deep should be filled in multiple passes to avoid flushing

Application rates for high volume roads range from 8 to 19 kg/m, depending on the layer thicknesses required.(1) The thickness of a 8 kg/m^2 rate would correspond to a 6.4 mm layer, while a 16 kg/m^2 rate would correspond to a 12.5 mm layer, when applied in a single pass.(1)

Rolling is generally not required for either surface retexturing or rut-filling, although areas not to be exposed to traffic should be rolled with a 9.1 metric ton pneumatic tire roller.(1) The pavement may generally be opened to traffic in about 1 hour.(1)

Fog Seal Application

Fog seals are applied using an asphalt distributor. It is important that excessive amounts of asphalt not be applied to the pavement, which could result in pickup by vehicles and perhaps cause a slippery surface.(7) Traffic may be allowed on the fog seal in about two to four hours after most of the water has evaporated.

Allowing traffic on fog seals too soon may cause accidents from a temporally slick surface from the wet fog seal and vehicles may be covered with asphalt from tire spray.

Sand Seal Application

The placement of a sand seal begins with a spray application of asphalt emulsion. This is then followed with the application of the sand (or other screenings). For maximum adhesion, the sand should be placed immediately following the application of the emulsion; however, for increased surface friction, the sand can be applied just as the emulsion starts to break.(1) Pneumatic rolling is desirable to embed the sand into the emulsion. The pavement may not be opened to traffic until the seal is set (typically about two hours).

12. SUMMARY

This module provides information on various surface rehabilitation techniques. The major topics covered in this module are summarized below:

- Surface rehabilitation treatments are placed for a variety of reasons, including:

  - Provide a new wearing surface.
  - Seal cracks in the surface.
  - Waterproof the surface.
  - Improve pavement surface friction and surface drainage.
  - Slow pavement weathering.
  - Improve the surface appearance.
  - Provide visual delineation between the mainline pavement and the shoulder.
  - Rejuvenate the top 6 mm of an AC surface.

- Many different materials fall into the category of surface rehabilitation techniques. These are summarized in table 3-5.5.

Due to their more widespread usage, this module focuses primarily on chip seals, OGFC, slurry seals, microsurfacing, fog seals, and sand seals.

- Material selection must be conducted prior to the job to ensure that the most appropriate materials have been selected, and to acquaint the engineers who will be on the job with the materials they will be using. This will assist them in noting any variations occurring during construction.

- Where applicable, the design of many surface rehabilitation treatments has been neglected to a large extent by practicing engineers in the field. If a surface rehabilitation treatment is going to be used to extend the service life of the pavement, careful consideration must be given to its proper design.

- The construction sequence is critical to obtain a pavement surface with all the components seated together in the correct manner. The use of emulsions requires more attention be given to this aspect.

Table 3-5.5. Types of surface rehabilitation treatments.

MATERIAL	PURPOSE
Chip Seal	Provide new wearing course Improve surface friction
Open-Graded Friction Course	Provide surface drainage Reduce hydroplaning Reduce tire spray Improve surface friction
Rubberized Asphalt Chip Seal	Bridge and seal cracks Delay reflection cracks
Slurry Seal	Seal pavement surface Retard surface raveling Improve surface friction
Microsurfacing	Level pavement surface Fill ruts Restore surface friction
Fog Seal	Seal pavement surface Rejuvenate oxidized asphalt Provide delineation
Sand Seal	Seal pavement surface Rejuvenate oxidized asphalt Provide delineation Improve surface friction
Road Oiling	Control blowing dust
Cape Seal	Seal pavement surface Improve surface friction
Sandwich Seal	Seal pavement surface Improve surface friction

13. REFERENCES

1. Raza, H., “An Overview of Surface Rehabilitation Techniques for Asphalt Pavements,” Federal Highway Administration, Office of Engineering and Office of Technology Applications, 1991.

2. “Porous Asphalt Pavements: An International Perspective 1990,” Transportation Research Record 1265, Transportation Research Board, 1990.

3. Pederson, C.M., “Microsurfacing with Natural Latex Modified Emulsion,” Report No. FHWA/OK 86(7), Oklahoma Department of Transportation, November 1986.

4. Shuler, S., “High-Traffic Chip-Seal Construction: The Tulsa Test Road,” Transportation Research Record 1300, Transportation Research Board, 1991.

5. Shuler, S., “Chip Seals for High Traffic Pavements,” Transportation Research Record 1259, Transportation Research Board, 1990.

6. Roque, R., D. Anderson, and M. Thompson, “Effect of Material, Design, and Construction Variables on Seal-Coat Performance,” Transportation Research Record 1300, Transportation Research Board, 1991.

7. “A Basic Emulsion Manual,” Manual Series No. 19, Second Edition, The Asphalt Institute, 1986.

8. Raza, H.,“State-of-the-Practice Design, Construction , and Performance of Microsurfacing” Federal Highway Administration, Office of Engineering and Office of Technology Applications, Report No. FHWA-SA-94-051, June 1994.

9. “Asphalt Technology and Construction Practices,” Educational Series No. 1, Second Edition, The Asphalt Institute, January 1983.

10. “Open-Graded Friction Courses for Highways,” Synthesis of Highway Practice 49, Transportation Research Board, 1978.

11. “Criteria for Use of Asphalt Friction Surfaces,” Synthesis of Highway Practice 104, Transportation Research Board, 1983.

12. Shuler, S. and D.I. Hanson, “Improving Durability of Open-Graded Friction Courses,” Transportation Research Record 1259, Transportation Research Board, 1990.

13. “Recommended Performance Guidelines for Emulsified Asphalt Slurry Seal Surfaces,” Leaflet A 105, International Slurry Seal Association, January 1986.

14. Maurer, D.A., “Ralumac Latex-Modified Bituminous Emulsion Mixtures: A Summary of Experience in Pennsylvania,” Report No. PA-86-042+82-22, Pennsylvania Department of Transportation, April 1987.

15. Roberts, F.L., P.S. Kandhal, E.R. Brown, D.Y. Lee, and T.W. Kennedy, “Hot-Mix Asphalt Materials, Mixture Design, and Construction,” National Asphalt Pavement Association, 1991.

16. Kandhal, P.S. and J.B. Motter, “Criteria for Accepting Precoated Aggregates for Seal Coats and Surface Treatments,” Transportation Research Record 1300, Transportation Research Board, 1991.

17. Roque, R., M. Thompson, and D. Anderson, “Bituminous Seal Coats: Design, Performance Measurements, and Performance Prediction,” Transportation Research Record 1300, Transportation Research Board, 1991.

18. Linden, R.N., Mahoney, J.P., Jackson, N.C., “The Effect of Compaction on Asphalt Concrete Performance” Presented at the Annual Transportation Research Board Meeting, January 1989, Washington DC.

19. Smith, H.A. “Performance Characteristics of Open-Graded Friction Courses A Synthesis of Highway Practice” NCHRP Synthesis 180, Transportation Research Board, Washington, DC, September 1992.

20. Training material from Ed Schlect past Northwest District Engineer for The Asphalt Institute.

MODULE 3-6

RECYCLING OVERVIEW

1. INSTRUCTIONAL OBJECTIVES

This module presents information on recycling of asphalt surfaced and portland cement concrete surfaced pavements. Asphalt pavement recycling techniques include hot in-place recycling, cold in-place recycling and hot central plant recycling. Recycled portland cement concrete (PCC) pavement has identified end uses in portland cement concrete, hot-mix asphalt (HMA), chip seals, and base courses. Upon completion of this module, the participant should be able to accomplish the following:

1. Identify the types of pavement recycling.
2. Describe recycling processes.
3. Describe equipment used for recycling.
4. Select appropriate recycling operation for different conditions of existing pavements.
5. Discuss mixture design methods that incorporate recycled pavement materials.
6. Describe benefits, costs and performance of recycling operations.

2. INTRODUCTION

The need to reuse or recycle existing pavement materials for the reconstruction and rehabilitation of asphalt and portland cement concrete pavements is of increasing importance. Recycling can help to optimize the use of available materials and energy supplies and to decrease the cost of maintaining highways, roads and streets in the United States.

Rehabilitation and maintenance of the highway and street transportation system in the United States is costly, time-consuming, material-intensive, and an ever-increasing burden on public agencies. The recycling of existing pavement materials for rehabilitation and maintenance purposes offers several advantages over the use of conventional materials and techniques. As the techniques for recycling improve and specification writing agencies and contractors become familiar with the various processes available, the use of recycling has demonstrated cost savings over the use of new materials for major maintenance and rehabilitation of pavements. The Federal Highway Administration (FHWA) estimates the pavement industry generated $105.5 million in savings using recycled materials in 1985 and that 34 States had accepted some form of asphalt recycling in their specifications by 1985.[28] More recent information on type and extent of pavement recycling is not available. Other major benefits of recycling are conservation of aggregates, binders and energy, as well as preservation of the environment and existing highway geometrics.

Recycling or reuse of existing pavement materials for pavement rehabilitation, reconstruction and maintenance is not a new concept; literature indicates that pavement recycling existed as early as 1915.[1] However, the quantity of pavement materials recycled from 1915 to 1975 is small in comparison to the amount of recycling that has taken place since 1975.

The engineering community's interest in recycling starting in 1975 was largely based on economics, with some interest in energy conservation. During the mid and late 1970s in the United States there were problems related to a) reduced funding for transportation facilities, b) materials supply, c) equipment availability, d) trained manpower availability, and e) energy awareness and availability. Recycling of existing pavement materials for construction, rehabilitation and maintenance purposes offered a partial solution to these problems. Specifically, recycling offered the following major potential benefits compared with conventional techniques:

- Reduced costs.
- Preservation of existing pavement geometrics.
- Conservation of aggregates and binders.
- Preservation of the environment.
- Energy conservation.

Because recycling appeared promising from a wide variety of viewpoints, a number of agencies, including the National Cooperative Highway Research Program (NCHRP),[2,3] Federal Highway Administration (FHWA), (see references 4, 5, 6, 7, 8, 9, and 10), Corps of Engineers (for the Air Force),[11] and United States Navy[12] sponsored recycling research and implementation studies.

Associations and institutes also contributed to the development of recycling in the United States. These groups include The Asphalt Institute,[13] National Asphalt Pavement Association (NAPA),[14,15] Portland Cement Association (PCA),[16] Pacific Coast user-Producer Group on Asphalt Specifications,[17] American Society for Testing and Materials,[18] American Concrete Pavement Association (ACPA), Asphalt Emulsion Manufacturers Association (AEMA) and the Asphalt Recycling and Reclaiming Association (ARRA).

Early research, development, and implementation efforts led to the categorization of four types of pavement recycling:

- Surface recycling.
- Cold recycling.
- Hot recycling.
- Portland cement concrete pavement recycling.

These forms of recycling are addressed in a comprehensive manner in several publications. (See references 2, 3, 10, 19, 20, 21, 22, and 23.)

In the past 20 years there has been increasing emphasis on the need to reduce pavement rehabilitation costs and to conserve energy. Because of these twin emphases, many public agencies have reexamined and recognized the great value of recycling techniques. Recycling is considered capable of not only producing cost and energy savings, but also reducing the demand for asphalt during supply interruptions.

Common practice for many years has been to waste the old pavements removed before pavement reconstruction. In recent years, the use of pavement grinding to restore pavement ride quality, remove corrugations, smooth faulted joints, etc. has produced additional quantities of waste asphalt and concrete pavement materials. A comparatively recent development has been to treat salvaged pavements as materials having economic value.

Contractors and plant operators in some parts of the country have routinely blended small quantities (approximately 5 percent) of waste asphalt pavements into mixes produced for commercial and private work. In some States, contractors bidding on recycling work are required to purchase or submit price reductions for waste asphalt pavements removed during construction.

Recycling portland cement concrete has not held the interest afforded the recycling of asphalt pavement. Techniques for removing reinforcing steel, etc. need to be developed to reduce handling costs, for example. However, interest is developing in using recycled concrete pavements as a source of aggregates for both bituminous and concrete mixtures. State highway and local agencies have reported the use of crushed concrete pavements in recycled mixtures.

The significance of these developments is that many public agencies no longer consider recycling an experimental process. Many public agencies routinely permit recycling alternates in their standard specifications, as well as in special provisions. Production equipment is now available to permit the effective use of recycling and cost benefits of 10 percent or more have been reported.

3. DEFINITIONS

Pavement recycling operations can be classified by the two types of paving material recycled: asphalt and portland cement concrete. These categories can be broken down further according to the particular procedure used. Figure 3-6.1 illustrates the framework in which the types of pavement recycling are defined.

Pavement Recycling. The reuse of material from in-place pavements which are processed to provide quality paving materials suitable for use in new construction or in the rehabilitation of pavements. It is applicable to asphalt concrete pavements, portland cement concrete pavements, other pavement surfaces, such as chip seals and slurry seals, and to roads and streets that are unpaved or that have asphalt, portland cement or pozzolanic base courses. Definitions of the types of recycling follows.[26,27]

Asphalt Pavement Recycling. The reuse of an existing asphalt pavement by employing one of three recycling procedures: surface recycling, cold-mix recycling, or hot-mix recycling.

Asphalt Pavement Surface Recycling. The reworking in-place of the surface of an asphalt pavement to a depth of less than about 50 mm by any of the suitable machinery available. This operation is a single or multistep process that may involve the use of added materials, including aggregate, modifiers or asphalt mixtures (virgin or recycled).

Cold-Mix Asphalt Pavement Recycling. The reuse of untreated base materials and/or asphalt concrete pavement that is either processed in-place or at a central plant with the addition of asphalt emulsions, cutbacks, portland cement, lime and/or other materials as required to achieve desired mix quality, followed by placement and compaction.

Hot-Mix Asphalt Pavement Recycling. The removal of more than the top 25 mm of an asphalt pavement with or without removal of underlying pavement layers (e.g., untreated base materials) that is processed by sizing, heating and mixing in a central plant with additional components such as aggregate bitumen or recycling agents; then relaid and compacted according to standard specifications for conventional hot mixtures (e.g., asphalt concrete base, binder, and asphalt concrete leveling or surface course).

Portland Cement Concrete Pavement Recycling. The reuse of existing portland cement concrete pavement by processing into aggregate and sand sizes, then used in place of or, in some instances, with additions of conventional aggregates and sand into a new portland cement, or asphalt concrete mixture, or used as aggregate for a stabilized or unstabilized base.

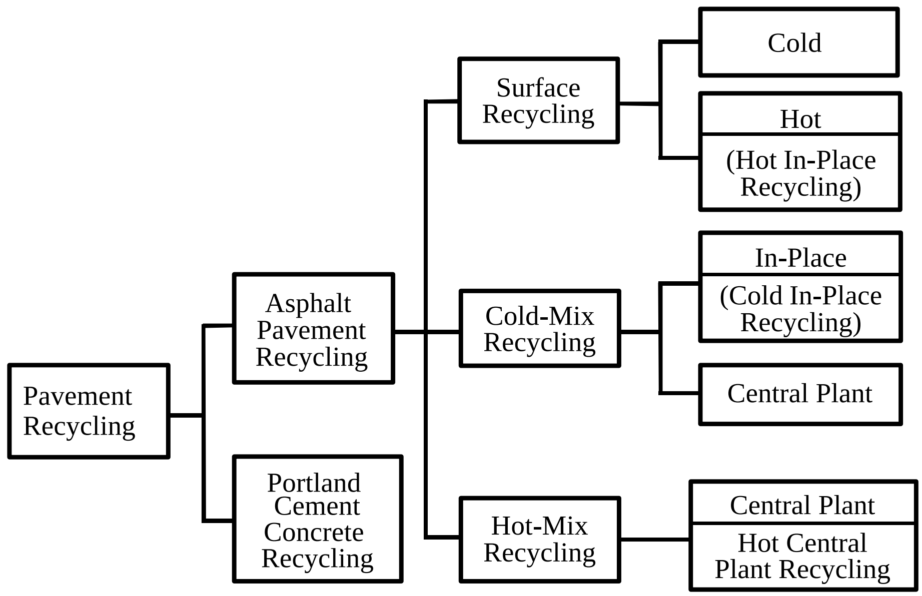


Figure 3-6.1. Categorization of pavement recycling.

As pavement recycling technology continues to develop, these definitions continue to be refined. For asphalt pavements, the three most common types of recycling are hot in-place recycling, cold in-place recycling, and hot central plant recycling. Definitions for these three forms of pavement recycling are given below:

Hot In-Place Recycling (HIR).[24] A process of correcting asphalt pavement surface distress by softening the existing surface with heat, mechanically loosening the pavement surface, mixing as necessary with recycling agent, aggregate, or hot-mix asphalt and replacing the loosened material on the pavement without removing the recycled material from the original pavement site. HIR may be performed as either a single-pass operation that recombines the restored pavement with virgin material, or as a two-pass operation, wherein the restored material is recompacted and the application of a new wearing surface then follows a prescribed interim period.

Cold In-Place Recycling (CIR).[25] A process of correcting asphalt pavement distress by processing, without heat and in-place, the existing pavement material(s) and combining as necessary with a stabilizing agent, recycling agent and/or aggregate. After processing and mixing the material is placed and compacted. A surface or wearing course is typically applied to the surface.

Hot Central Plant Recycling. The removal of a portion of an asphalt pavement with or without removal of underlying pavement layers (e.g., untreated base materials) that is processed by sizing, heating and mixing in a central plant with additional components such as aggregate, asphalt or recycling agents and then relaid and compacted.

Additional definitions associated with asphalt pavement recycling are given below:

Reclaimed Asphalt Pavement (RAP). Removed and/or processed pavement materials containing asphalt cement and aggregate.

Reclaimed Aggregate Material (RAM). Removed and/or processed pavement materials containing no asphalt cement.

Recycled HMA. The final mixture of RAP, new asphalt cement, recycling agent, if necessary, RAP or new aggregates produced at a hot-mix plant.

Asphalt Recycling Agent. A petroleum product additive with a combination of chemical and physical properties designed to restore aged asphalt to desired specifications. The recycling agent should conform to the specifications contained in ASTM D 4552: "Standard Practice for Classifying Hot-Mix Recycling Agents."

Asphalt Modifier. A generic term describing any compound or material that is used as an admixture to alter or improve the properties of the asphalt binder in the recycled asphalt mixture. Included are asphalt cements, cutback asphalts, emulsified asphalts, recycling agents, and polymers.

4. SELECTION OF RECYCLING AS A REHABILITATION ALTERNATIVE

The reasons for selecting a form of recycling as a rehabilitation alternative include initial cost, life-cycle cost, time required for the rehabilitation alternative, reliability, or chance of success of the alternative, etc. Table 3-6.1 is provided to generally define some of the advantages and disadvantages of the major types of asphalt recycling alternatives.

Detail descriptions and other information associated with pavement recycling are provided in module 3-7 for hot in-place recycling, module 3-8 for cold in-place recycling, module 3-9 for hot central plant recycling and module 4-12 for portland cement concrete recycling.

Table 3-6.1. Major advantages and disadvantages of asphalt pavement recycling techniques.[3]

Recycling Techniques	Advantages	Disadvantages
Surface	• Reduces frequency of reflection cracking. • Promotes bond between old pavement and thin overlay. • Provides a transition between new overlay and existing gutter, bridge, pavement, etc. that is resistant to raveling (eliminates feathering). • Reduces localized roughness due to compaction. • Treats a variety of types of pavement distress (raveling, flushing, corrugations, rutting, oxidized pavement, faulting) at a reasonable initial cost. • Improves skid resistance.	• Provides limited structural improvement. • Heater-scarification and heater-planing have limited effectiveness on rough pavement without multiple passes of equipment. • Limited repair of severely flushed or unstable pavements. • Some air quality problems. • Vegetation close to roadway may be damaged. • Mixtures with maximum size aggregates greater than 1-inch can not be treated with some equipment. • Limited disruption to traffic.
In-place	• Provides significant structural improvements. •Treats all types and degrees of pavement distress. • Can eliminate reflection cracking. • May improve frost susceptibility. • Improves ride quality. • Improves skid resistance. • Minimizes hauling.	• Quality control not as good as central plant. • Traffic disruption. • PCC pavements can not be recycled in-place. • Curing often required for strength gain.
Central	• Provides significant structural improvements. • Treats all types and degrees of pavement distress. • Can eliminated reflection cracking. • Improves skid resistance. • May improve frost susceptibility. • Enables geometrics to be more easily altered. • Improves quality control if additional binder and/or aggregates must be used. • Improve ride quality.	• Potential air quality problems at plant site. • Traffic disruption.

5. REFERENCES

1. "Hot Recycling of Yesterday," Recycling Report, Vol. 1, No. 2, National Asphalt Pavement Association (September 1977).

2. TRB, "NCHRP Synthesis of Highway Practice 54: Recycling Materials for Highways," Transportation Research Board, National Research Council, Washington, DC (1978) 53 pp.

3. Epps, J.A., D.N. Little, R.J. Holmgreen and R.L. Terrel, "NCHRP Report 224: Guidelines for Recycling Pavement Materials," Transportation Research Board, National Research Council, Washington, DC (September 1980) 137 pp.

4. Beckett, S., "Recycling Asphalt Pavements," Demonstration Project No. 39, Interim No. 1, Federal Highway Administration, Region 15 (January 1, 1977).

5. Brown, D.J., "Interim Report on Hot Recycling," Demonstration Projects Division, Federal Highway Administration, Region 15 (April 1977).

6. "Concrete Recycling Project Ready," FHWA Newsletter, No. 8 (October 1978).

7. "Initiation of National Experimental and Evaluation Program (NEEP) Project No. 22 — Pavement Recycling," Notice N 5080.64, Federal Highway Administration, Washington, DC (June 3, 1977).

8. "Recycled Asphalt Concrete," Implementation Package 75-5, Federal Highway Administration, Washington, DC (September 1975).

9. Anderson, D.I., D.E. Peterson, M.L. Wiley and W.B. Betenson, "Evaluation of Selected Softening Agents Used in Flexible Pavement Recycling," Report No. FHWA-TS-79-204, Federal Highway Administration, Washington, DC (April 1978).

10. "Highway Focus," Vol. 10, No. 1, Federal Highway Administration, Washington, DC (February 1978).

11. Lawing, R.J., "Use of Recycling Materials in Airfield Pavements — Feasibility Study," Report AFCEC-TR-76-7, Air Force Civil Engineering Center, Tyndall Air Force Base, Florida (February 1976).

12. Brownie, R.B. and M.C. Hironaka, "Recycling of Asphalt Concrete Airfield Pavements," Naval Civil Engineering Laboratory, Port Hueneme, CA (April 1978).

13. "Asphalt Pavement Recycling Using Salvaged Material," The Asphalt Institute, West Coast Division.

14. "State-of-the-Art: Hot Recycling," Recycling Report, Vol. 1, No. 1, National Asphalt Pavement Association (May 27, 1977).

15. "State-of-the-Art: Hot Recycling 1978 Update," Recycling Report, Vol. 2, No. 3, National Asphalt Pavement Association (October 1978).

16. "Recycling Failed Flexible Pavements with Cement," Portland Cement Association, Skokie, IL (1976).

17. Pacific Coast User-Producer Specification Committee, miscellaneous internal reports (1978, 1979).

18. “Recycling of Bituminous Pavements,” STP 662, ASTM, Philadelphia, PA (1978).

19. Transportation Research Record 780: “Proceedings of the National Seminar on Asphalt Pavement Recycling,” Transportation Research Board, National Research Council, Washington, DC (1980) 140 pp.

20. “Standard Practice for Pavement Recycling,” Technical Manual No. 5-822-10, Air Force Manual, Departments of the Army and the Air Force (February 1984).

21. Newcomb, D. and J.A. Epps, “Asphalt Recycling Technology: Literature Review and Research Plan,” Final Report, Air Force Engineering and Services Center, Tyndall Air Force Base, Florida (June 1981).

22. ARE, Inc., “Pavement Recycling Guidelines for Local Governments — Reference Manual,” Report No. FHWA-TS-87-230, U.S. Department of Transportation, Federal Highway Administration, Washington, DC (September 1987).

23. “Pavement Recycling Guidelines for Local Governments — Appendices,” prepared by ARE, Inc. for FHWA, Report No. TS-87-230 (September 1987).

24. Button, J.W., D.N. Little and C.K. Estakhri, “Hot In-Place Recycling of Asphalt Concrete,” NCHRP Synthesis 193 (1994).

25. Epps, J.A., “Cold Recycled Bituminous Concrete Using Bituminous Materials,” NCHRP Synthesis 160 (July 1990).

26. American Association of State Highway and Transportation Officials (AASHTO), “Proposed AASHTO Guide for Design of Pavement Structures,” Draft, Washington, DC (March 1985).

27. Asphalt Institute, The, “Asphalt Hot-Mix Recycling,” Manual Series No. 20 (MS-20), College Park, Maryland (August 1981) Second Edition - 1986.

28. Roads and Bridges, “Special Report: Asphalt ‘86,” Vol. 24, No. 1, Des Plaines, IL (January 1986).

MODULE 3-7

HOT IN-PLACE RECYCLING

1. INSTRUCTIONAL OBJECTIVES

This module presents information on hot in-place recycling (HIR) of asphalt pavements. Hot in-place recycling techniques include heater-scarification, repaving, and remixing. Upon completion of this module, the participant should be able to accomplish the following:

1. Identify the types of hot in-place recycling.
2. Define the types of equipment and their operational sequence for hot in-place recycling operations.
3. Describe mixture design procedures for use in hot in-place recycling operations.
4. Define structural layer coefficients for use in pavement design.
5. Describe performance of hot in-place recycled pavements.
6. Define existing economic information on hot in-place recycling operations.
7. Describe key elements of specifications and quality control/quality assurance guidelines.
8. Provide recommendations for appropriate use of hot in-place recycling techniques.

2. INTRODUCTION

Hot in-place recycling operations involve the use of heat. “Cold planing” and “cold milling” are asphalt pavement surface recycling operations that do not use heat. These “cold” operations are primarily used for pavement removal operations as part of a rehabilitation operation, such as cold mill and hot-mix asphalt overlay.

The use of hot in-place recycling operations dates to the 1930s with the development of heater-planer equipment in California.[1,2] Since the 1930s, a wide variety of hot in-place recycling equipment has been developed. Heater-scarifying equipment was developed by the 1960s and heater remixing equipment was developed in the 1980s and 1990s.

Hot in-place equipment has experienced several significant improvements since the development of the first heater planers in the 1930s. Heater-scarifiers were developed to heat, scarify, and reprofile the pavement. Over the years, equipment has been developed which allows for a greater depth of heating and scarification, as well as improved pavement smoothness associated with the laydown operation. Typical heater-scarification operations heat and scarify to depths of 10- to 25-mm. The use of hot millers in place of scarifiers and improved heaters has increased depth and versatility of the equipment.

Hot in-place recycling repaving equipment was developed in the 1950s and 1960s. A layer of hot-mix asphalt is applied on top of a heated and scarified layer. A single- or two-pass equipment operation can be used. Scarification depths of 10- to 25-mm are typical.

Hot in-place remixing operations were developed in the 1980s and 1990s. This equipment heats, scarifies or hot mills the existing equipment, mixes new materials and lays the combined recycled and new mixtures. Removal depths of from 10- to 50-mm are typical.

Details of hot in-place recycling operations are described in more detail below. References 4 through 8 are the primary recent references on hot in-place recycling and have been used as the basis for this discussion.

3. DEFINITIONS

Hot in-place recycling is the process of correcting asphalt pavement surface distress by softening the existing surface with heat, mechanically loosening the pavement surface, mixing as necessary with a recycling agent, aggregate or hot-mix asphalt, and replacing the loosened material on the pavement without removing the recycled material from the original pavement site. HIPR may be performed as either a single-pass operation that recombines the restored pavement with virgin material, or as a two-pass operation, wherein the restored material is recompacted and the application of a new wearing surface then follows a prescribed interim period.

The Asphalt Recycling and Reclaiming Association (ARRA) provides definitions for the basic hot in-place recycling operations: heater-scarification, repaving and remixing.[3]

<u>Heater-Scarification</u>. Heating, scarifying, rejuvenating, leveling, reprofiling, compacting.

<u>Repaving</u>. Heating, scarifying, rejuvenating, leveling, laying new hot-mix, reprofiling, compacting.

<u>Remixing</u>. Heating, scarifying, rejuvenating, mixing (and/or adding new hot-mix), leveling, reprofiling, compacting.

As stated above, these three forms of “asphalt pavement surface recycling” are commonly called hot in-place recycling.

4. METHODS AND EQUIPMENT

A chronological record of the evolution of HIPR shows an increasing understanding and improvement on the concept. As indicated earlier, the modern era of recycling began in the mid 1970s, but trials and experiments began much earlier. The following is a summary of the evolution of HIPR, based on the terminology adopted by ARRA.

Heater Scarification

Sometimes called a reshaping process, heater-scarification was originally developed by a Utah contractor sometime in the 1930s.[9,10] Common usage did not evolve until the 1960s. By the 1970s, further evolution moved the technology into more complex systems. This relatively simple process includes several steps, as follows:

- Heating the old pavement surface.
- Scarifying the softened surface with a bank of stationary teeth.
- Adding a liquid recycling agent (when needed).
- Mixing and leveling the recycled loose mixture with an auger and/or laydown machine.
- Compacting with conventional rollers.

Figure 3.7-1 shows a typical equipment train for heater-scarification. The depth of scarification and treatment usually was about 10- to 20-mm. Depending on the depth of scarification and condition of the pavement, the resulting surface is not always smooth and uniform.

Early attempts at heating used direct flame, but this approach was gradually replaced by radiant infrared heaters and fired by propane gas. Infrared (IR) heating helped reduce the overheating (and excessive hardening) and smoking caused by direct flame. One or more heater units (see figure 3-7.1) are used to gradually raise the pavement temperature sufficiently to allow the scarifying teeth to scrape through the surface. Surface temperatures ranging from 110 to 150 °C are generally achieved when at least two heaters are used in tandem. The scarifying teeth are normally spring loaded tines that are able to override obstacles such as manholes. The use of tines for scarification may limit the depth of scarification and cause aggregate breakage. The recycled layer contains relatively hard asphalt binder because of both the normal aging of the surface and the heating required to soften it. Thus, recycling or rejuvenating oils are commonly used to restore flexibility. The heater-scarified surface is usually overlaid using conventional hot-mix asphalt (HMA).

Repaving

When heater-scarification is simultaneously combined with an overlay of HMA, it is called repaving. Often called the Cutler process (named after its inventor), repaving is a process that started in the 1950s and was upgraded in the 1960s.[11] The repaving process has several steps as follows:

- Heating (i.e., preheating).
- Scarifying using teeth or a rotary mill.
- Adding a recycling agent.
- Mixing the recycling agent and loosened mixture.
- Spreading and screeding the recycled mixture.
- Placing a new HMA overlay.

Figure 3-7.2 and figure 3-7.3 show typical equipment trains for the repaving process. Current practice is to heat the existing surface to approximately 190 °C using infrared (IR) preheaters as well as heaters in the recycling unit. The heat-softened pavement is then removed to a depth of about 10- to 20-mm, depending on how well the heaters have softened the asphalt. For those machines that use milling to loosen the old pavement, some variations in the design and layout allow for adjustment of depth during operation. On some machines, the cutter heads can be raised or manipulated to avoid obstacles such as manholes.

Rejuvenation of the loosened mixture is accomplished by spraying the liquid additive onto the pavement or mixing chamber or windrow at a rate determined by laboratory testing. This predetermined application rate is then locked into the machine so that it is adjusted by the forward motion or progress of the train. Mixing of the loosened mixture and recycling agent is usually accomplished by auger mixers that also transfer the mixture into a windrow. Additional transverse augers then spread the recycled mixture in front of the screed. The screed levels and shapes the recycled material. Finally, a new HMA is added and spread with another screed directly on top of the recycled layer. The recycled layer may still retain temperatures well over 100 °C so that the new HMA is well bonded and integrated with the old pavement surface. Depending on the manufacturer's design, the lift thickness and/or cross shape may be controlled manually or by using automatic controls. Repaving is a practical solution to restore and improve a pavement surface in one pass of a recycling train.

Remixing

When additional materials are needed to recycle the pavement, such as mineral aggregate or virgin HMA, the remixing process is used. This approach permits upgrading the existing pavement with additional thickness and/or improving the old HMA by changing the aggregate gradation or adjusting the

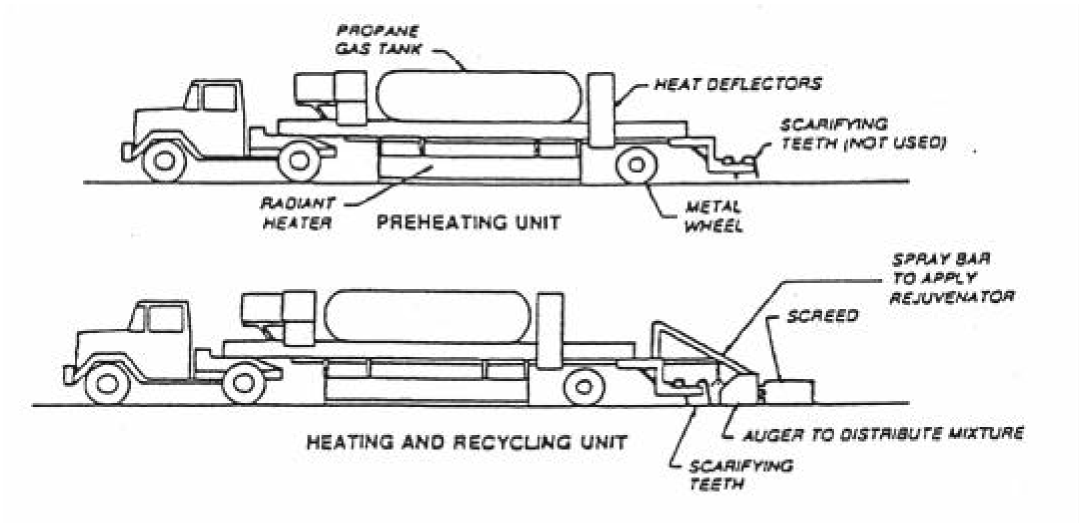


Figure 3-7.1. Heater-scarifier process.(8)

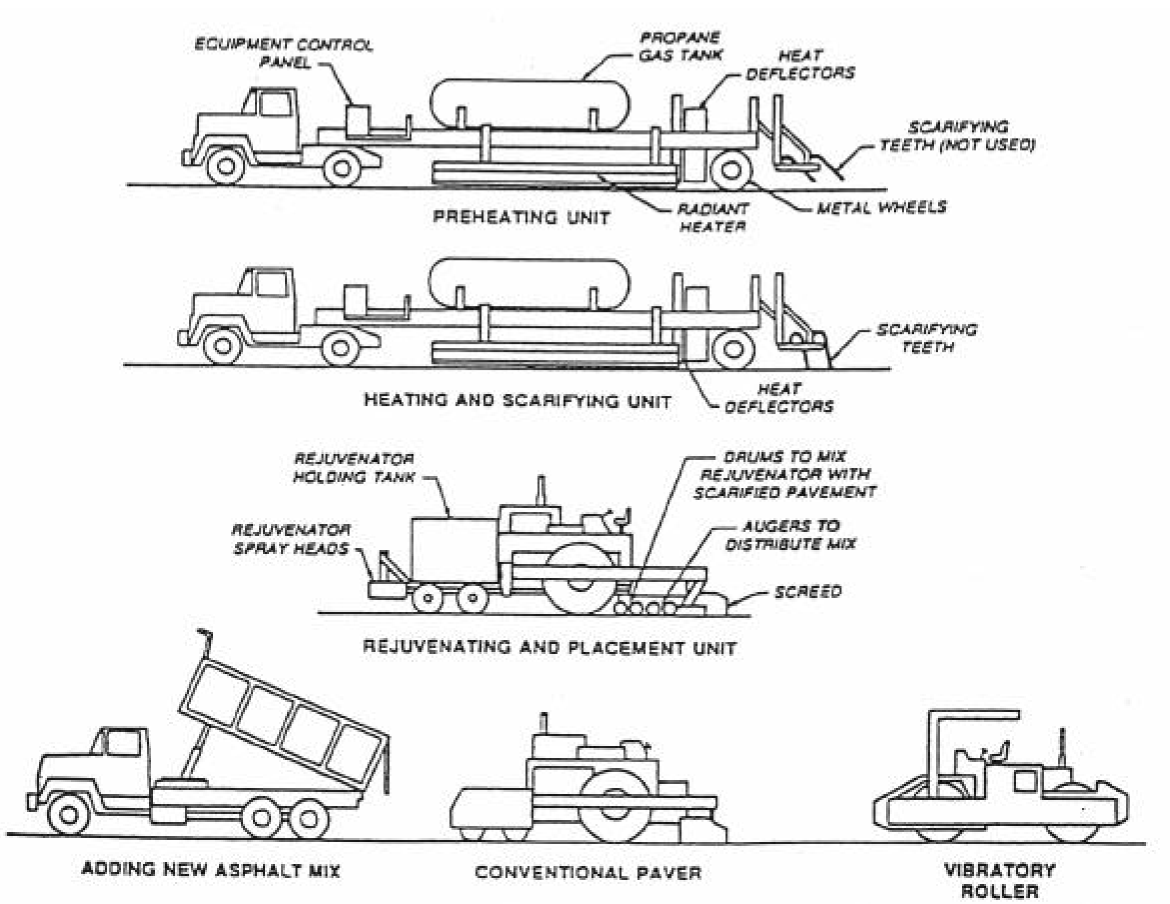


Figure 3-7.2. Multiple pass repaving process used by Dustrol.(8)

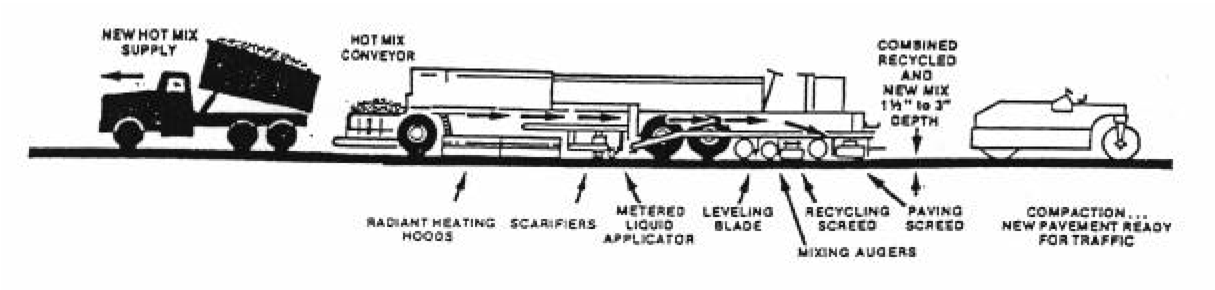


Figure 3-7.3. Single pass repaving process used by Cutler.[8]

binder properties. The process is somewhat similar to the repaving process; but usually, more thorough heating and mixing is accomplished.

Figures 3-7.4 to 3-7.9 show sketches of the equipment generally used in remixing. One or more preheater units, usually infrared, are used to warm and soften the pavement ahead of the others. In reasonably stable weather (not windy or cold pavement temperature), the preheaters can raise the temperature of the pavement to about 85 to 105 °C. The preheaters are large; a full lane wide and up to 12 m long. Each unit in the paving train typically has a heater, including the remixer, although it may be smaller than the preheaters.

A few manufacturers still utilize stationary tines or teeth to scarify the warm pavement, but most currently use rotating milling heads. These are similar to those used for cold milling, but require less power because the warm pavement is softer. Most systems can mill to a depth of 25- to 50-mm, although a target (desired) may be 50 mm. Experience has been that the higher boundary of milling depth is controlled by the temperature and is about 50 mm with conventional IR heaters. When greater depths are obtained, the lower temperatures cause aggregate breakage and the overall average temperature of the loose reclaimed asphalt pavement (RAP) is lowered, making it more difficult to obtain high quality recycled mixtures.

The remixer unit, as shown in figures 3-7.4 to 3-7.6, usually has a hopper to receive virgin HMA when needed. Some equipment picks the virgin HMA from a windrow, however, and blends it with the hot milled RAP. The remixer unit shown in figure 3-7.4 has a conveyor that lifts the virgin HMA over the heater and milling head and then into the pugmill. Here, the RAP, recycling agent and virgin HMA or aggregate are blended into the final mixture. Although some equipment designs have attempted to accomplish all the mixing on the pavement surface, this procedure has been rather unsuccessful. Most specifying agencies call for pugmills to be used.

Two general approaches to relaying the recycled mixture have been used. In one, a paving machine is snugged up to the rear of the repaver and accepts the recycled mixture directly into its hopper; after that, paving proceeds the same as for a new mixture. In the other, the recycled RAP is windrowed behind the repaver and an on-board auger screed system spreads the mixture, ready for compaction.

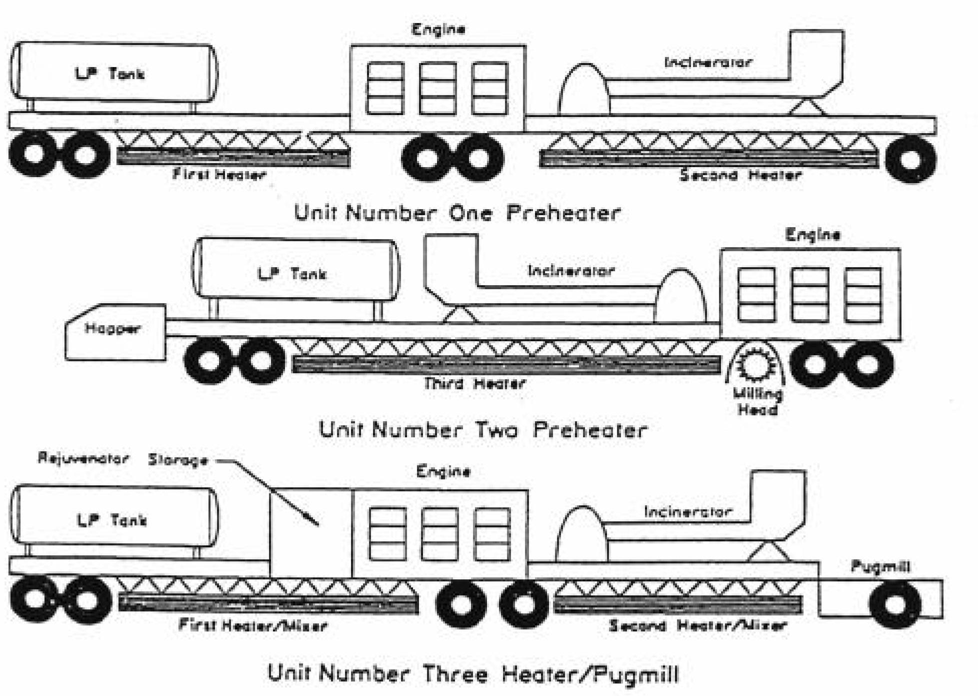


Figure 3-7.4. Artec multistage remixer process.(6)

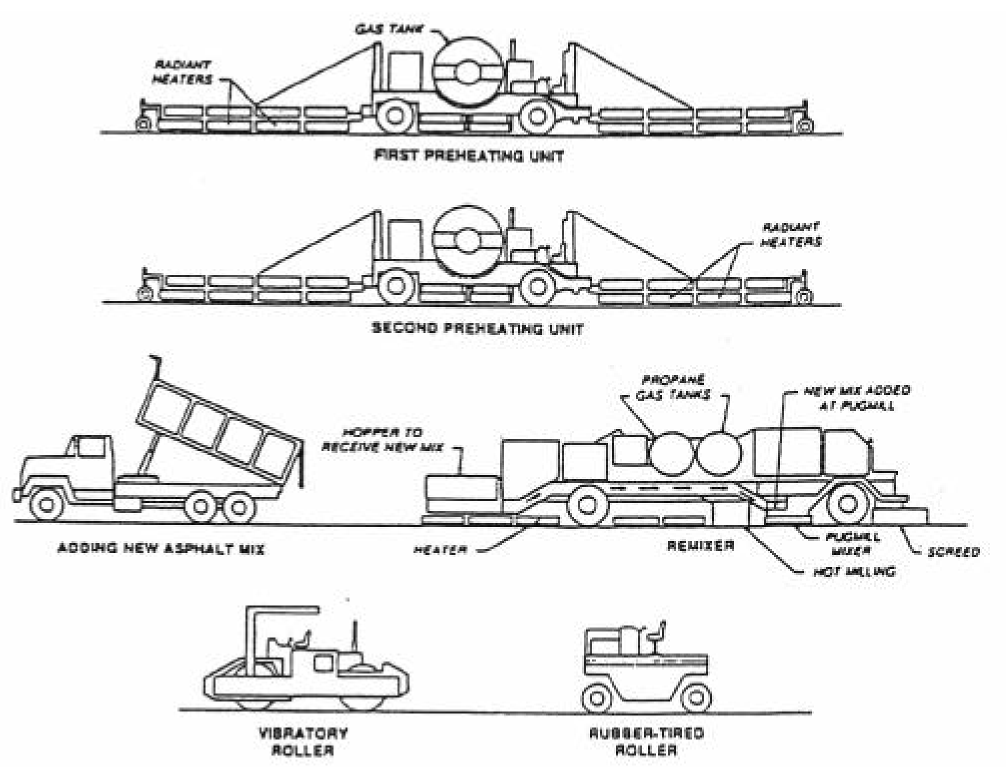


Figure 3-7.5. Single pass remix process, Taisei Rotec HIPR-5.(8)

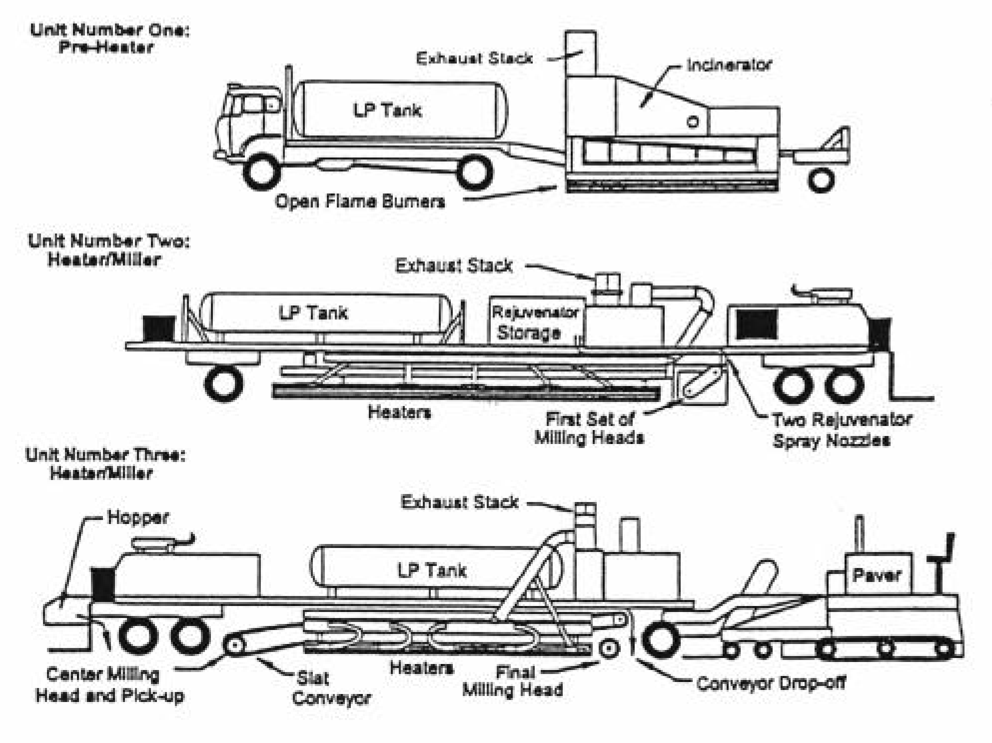


Figure 3-7.6. Pyrotech pyropaver 300E remixer process.(6)

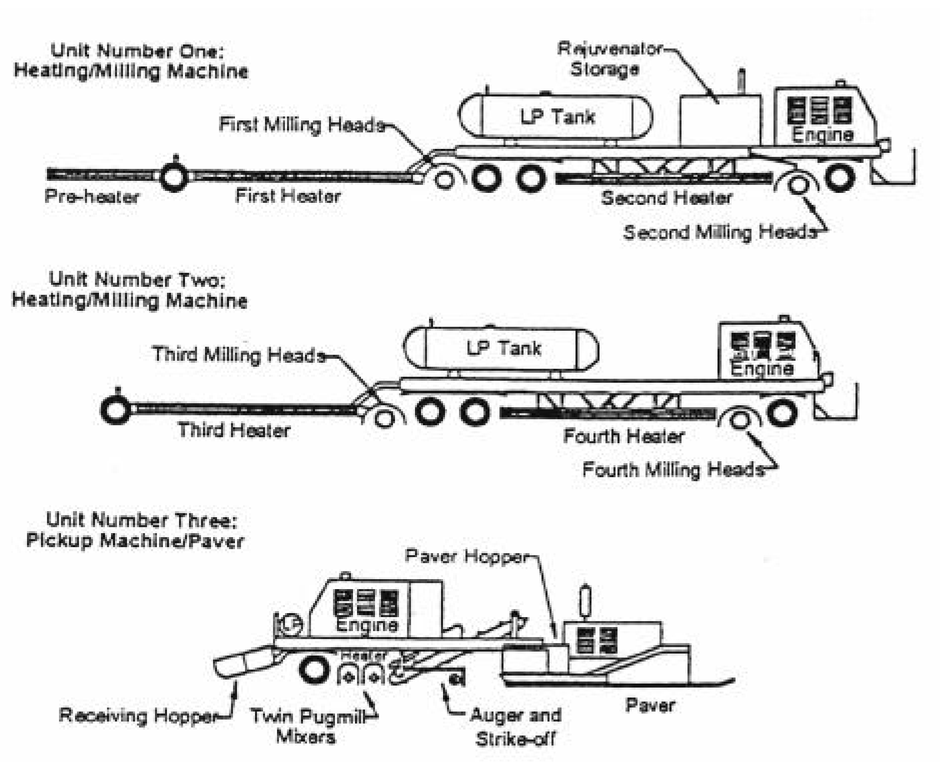


Figure 3-7.7. Artec four-state remixer process.(6)

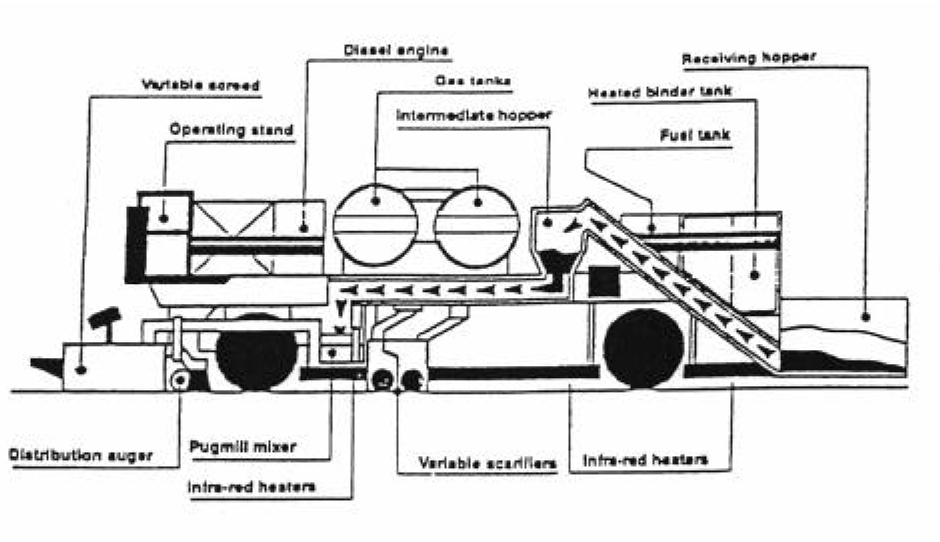


Figure 3-7.8. Wirtgen 4500 remixer.(8)

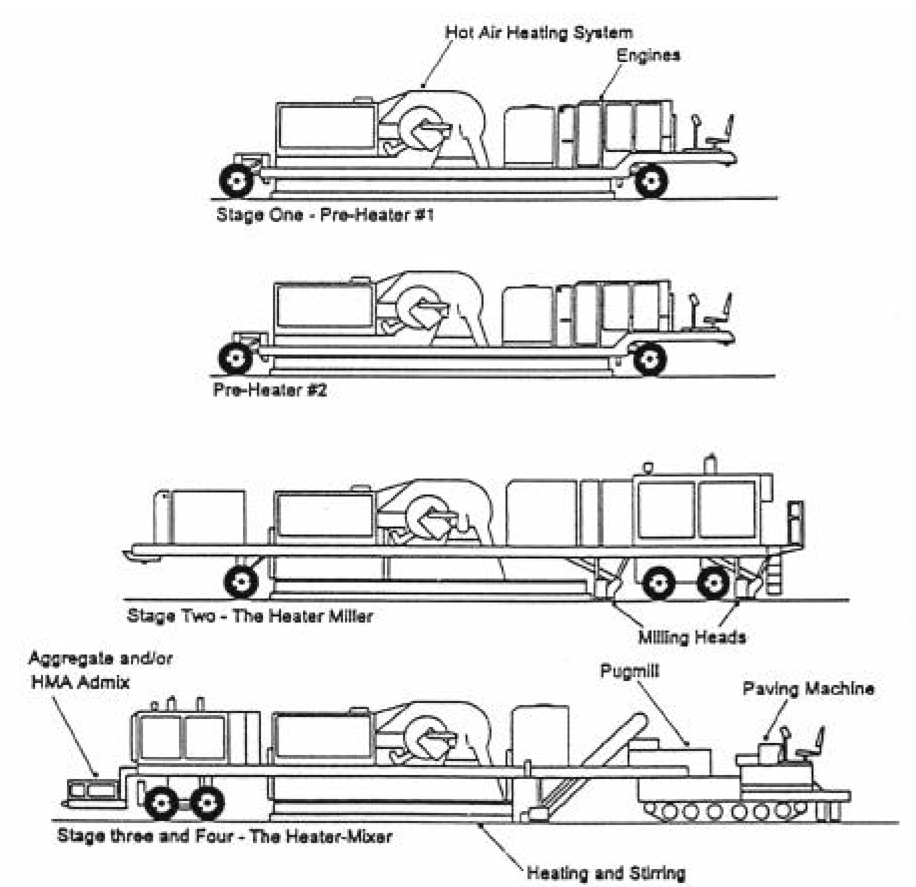


Figure 3-7.9. Martec four-stage remixer with hot air/infrared heaters and recirculating vacuum system to improve air quality.(6)

Improvements in Remixing

Variations of the repave and remix processes were developed in the mid and late 1980s and early 1990s, and this technology is being used in North America. The hot in-place systems described earlier are often called multistage because of the progressive stages of preheating, hot milling, remixing and paving. But, they were also single step, meaning that the preheating was followed by a single milling operation and the full depth of milling was done in one pass of the cutting heads. These single-stage equipment styles were utilized by several manufacturers, including Rorison-Wiley Blacktop, Wirgen, Taisei Rotec, and others.

The later variations, two-step or multistep, were developed because of the recycle depth limitations of single step trains. It is difficult to heat pavement at a depth in a timely fashion using a single IR source. Figure 3-7.6 shows typical heating depth rates[16] for IR surface heating. It is apparent that the surface is easily heated, but adequately high temperature for hot milling and remixing is not feasible at 50 mm depth, for example. Further, if the heater is slowed in an attempt to heat at depth, the high surface temperature tends to oxidize the asphalt excessively.

Two-step and four-step processes were developed by Pyrotech, Inc. and Artec, Inc., both of British Columbia, Canada. The idea was to take advantage of the ability to heat the top 12 mm or 25 mm of the pavement effectively. In the two-step process (figure 3-7.10), two miller heads are used, one on each heater unit following the preheater. The first heats the original pavement surface and then mills off approximately 25 mm (about the limit of heating). The RAP removed is windrowed in the center to expose most of the underlying surface. The second unit follows, picking up the windrow, milling off the top 25 mm under the windrow, continues heating the now-exposed surface, then mills a second 25 mm depth. The heating procedure is more efficient and the recycling process is much faster, thereby increasing productivity.

The Artec four-step process is similar to the two-step process, except that four heater-miller units are used (see figure 3-7.7). Each unit heats the surface or exposed underlying surface and mills off about 13 mm of pavement. Again, taking advantage of the rapid IR heating at the surface, it is easy and efficient to heat a 13 mm for easy milling, so forward progress is increased to very acceptable levels. Also, there is less aggregate fracture since the layer being milled is much warmer. The entire quantity of loose RAP material is heated adequately since each 12.5 mm layer is mixed with the others.

Figure 3-7.11 shows how the two-step process is effective in heating to 50 mm depth, even though there is rapid cooling between heater-miller units.[16] The four-step process does the same, even more effectively. Figure 3-7.11 shows how the higher temperature of each underlying layer contributes to a more rapid rise in overall temperature. The ability to heat a warm underlying surface is improved over a cold surface and the step-wise process takes advantage of this factor.

The hot in-place recycling developed by Wirtgen (Germany) is similar to the systems previously described. Figure 3-7.8 shows the remixer unit that includes a receiving hopper for admix, IR heaters, and milling heads. A preheater (not shown) is similar to other IR preheaters. This unit has its own leveling screed and does not rely on a paving machine. There is no special provision for air quality control.

There have been incremental changes in the HIPR equipment and processes with each succeeding year of operation. For example, alternative heating methods, such as microwaves, have been attempted on an experimental basis,[6] but no commercial size units have been placed in service as yet. This idea is

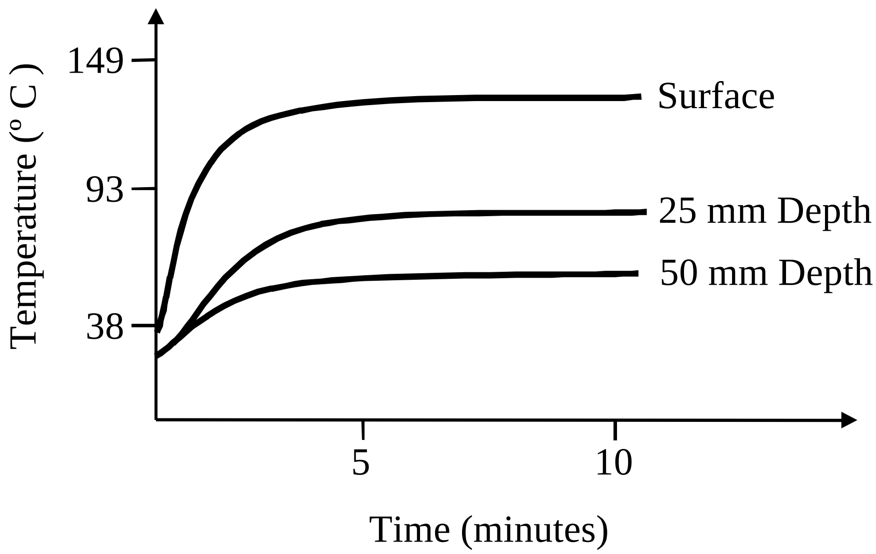


Figure 3-7.10. The heating process raises the surface temperature rapidly, while heating at depth takes much longer.[4]

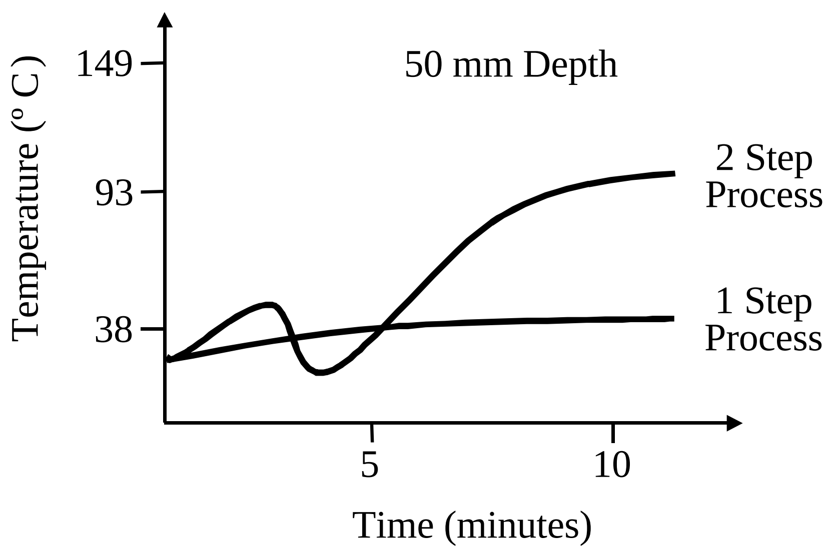


Figure 3-7.11. With a one-step heating process, it is difficult to raise surface temperature adequately within a reasonable time. Using a two-step process, (Artec or Pyrotech), the upper one-inch is removed after heating the surface and then the second stage heat is applied to the underlying layer, boosting the temperature higher.[4]

intriguing because the microwaves heat the pavement at depth (sometimes too deeply) and without the smoking of other flame-type heaters. On-board electric power requirements are high, but improvements in concentrating the microwaves where needed are making it more effective.

Through the early 1990s, the further development of HIPR technology seemed to reach a plateau. There were a number of HIPR trains operating in the United States and Canada and they were using the various technologies previously discussed. Because of the enormous potential of hot in-place recycling, several user agencies began to seriously evaluate its effectiveness and conducted surveys of projects through key people who were directly involved. These studies were based on projects constructed in 1992 and earlier. (See references 12, 13, 14, 15, and 16.) For example, questionnaires sent to various users addressed questions on 1) advantages, 2) deficiencies, and 3) cost. The responses from these queries and other sources are summarized below:

Advantages

- Conservation of energy and materials, including aggregates, asphalt, and fuel (less truck hauling).
- Construction improvements realized through shorter duration projects; less traffic delay or control; safer site conditions; easy mobilization; no milling disposal costs.
- Improved pavements through correction of surface conditions and up to 50 mm depths; correction of mix deficiencies; elevation and curb lines are unaffected; improved ride and skid resistance.
- Environmental concerns addressed through improved air quality due to less trucking; smoking on site reduced by using afterburners on recent upgrades of equipment.

Disadvantages

- Concerns about the finished pavements: compaction problems resulting in segregated and open texture and low density, cracks that soon reappeared, smoothness deficiencies, inadequate depth of milling, and insufficient mixing. Excess aggregate fines caused by milling to a depth where the pavement was too cold, thus causing aggregate fracture.
- Equipment related problems: frequent breakdowns, too much smoke and steam, and too long a paving train. Further equipment shortcomings indicated that the remixed RAP was too cold at laydown and the HIPR equipment did not do well in windy or cool weather or when the pavement was damp.
- Poor longitudinal joint matches and undersized paver may be due to factors other than the actual HIPR equipment. Remixer would not accept enough virgin HMA admix for effective remixing to upgrade the pavement.

Costs

- Cost savings over a 50 mm HMA overlay ranged from 19 to 45 percent for 14 projects reported by one study; equal to a 25 mm overlay in cost.
- Costs for HIPR vary because of different project needs: detailed curb and driveway paving, location, weather, moisture in the pavement, etc., all of which affect productivity.

Needed Improvements

Through 1993, HIPR in North America evolved rapidly and several improvements resulted. The evolution is continuing as a result of manufacturers' innovations created by competition as well as demands of the customers. Again, through the questionnaire process, a long list of needed improvements has emerged. Included among these are at least the following:

Equipment

- Improved air quality.
- Increased mixture temperature.
- Deeper recycling.
- Adjustable width of milling.
- Quieter operation.
- Increased capability to add virgin mixture and cold aggregate.
- Improved ability to climb steeper grades.
- Better interlocking of additive and remixing speed.
- Additional instrumentation for improved QC monitoring.

Procedures

- Better project evaluation and core sampling.
- More on-site testing to improve QC.
- Development of better procedures and criteria for selecting potential HIPR projects.

This last factor was emphasized by marginal results in the State of Oregon when three of four HIPR projects attempted in 1992 and 1993 were deemed inappropriate for HIPR rehabilitation.[5,17] For example, a typical problem was an attempt to recycle to 50 mm depth in a layer that was about 70 mm thick, and which was delaminated. The milling process broke loose large chunks of underlying pavement that the machinery could not accommodate.

Equipment developers continue to address the identified problems and improve the hot in-place recycling operation.

5. RECYCLED MIXTURE DESIGN

The mixture design process for hot in-place recycling and hot central plant recycling is identical. The mix design process is described in module 3-9 on hot central plant recycling. Typically, softer recycling agents and lower recycling agent contents are used for hot in-place recycling operations. Low percentages of new hot-mix asphalt and high percentages of RAP that are typically associated with hot in-place recycling operations are responsible for the differences in the type and amount of recycling agent.

6. STRUCTURAL DESIGN

A phone survey of fifty States reported in reference 8 indicated that only seventeen States considered the structural value or load carrying ability of hot in-place recycling. Fourteen States considered the structural value of hot in-place recycling to be about the same as that of new hot-mix asphalt. Three States indicated that they assigned a structural value that is slightly less than new hot-mix asphalt. One research project indicated the same structural value for hot in-place recycling and new hot-mix asphalt.[18]

7. PERFORMANCE

A recent survey of fifty States indicated that twenty-eight States used hot in-place recycling on an experimental basis and an additional ten States used hot in-place recycling on a somewhat regular basis. Thirteen States have reported using heater-scarification while fifteen States use repaving and sixteen States remixing.[8]

Hot in-place recycling has been on both major and secondary highways. Some States place a surface seal or hot-mix asphalt overly depending on the specific project condition.[8]

Projects completed in Canada have the best performance documentation, as most of the recent developments in HIPR development have been in Canada. Long-term performance is not yet well documented, although numerous projects were constructed as early as 1987. More recent and short term behavior or performance data are available on some projects.

It is difficult to generalize about how well pavements are recycled, because each project is different, but it is fair to say that the objectives were met on most projects. For example, the earlier hot in-place recycling projects conducted in British Columbia by Pyrotech and Artec were usually rejuvenated with recycling agent, so the binder was softened as measured by viscosity and penetration test results. The resilient modulus of recycled mixtures generally is a good indicator of the recycling and is much more sensitive to binder changes than is the Marshall Stability, for example. If an existing old pavement shows age hardening, then the resilient modulus is relatively high and the values are significantly lowered after hot in-place recycling using a recycling agent.[4] Alternatively, a too-soft, rut-prone mixture in an existing pavement may be stiffened by the HIPR process due to additional aging by the heating, as well as by addition of HMA with better aggregate and perhaps a stiffer asphalt rather than a recycling agent.

The considerable experience with HIPR in Canada has led to a Government-sponsored project to evaluate the performance of HIPR pavements constructed in Alberta since 1990.[19] The ten projects were recycled using single-pass, two-stage HIPR trains at 50 mm depth. The project focused on assessing the binder rheology and the mixture volumetric properties before and after construction. In addition, these same properties (binder and mixture) were compared at the time of construction and again in 1996 to evaluate the effects of time and traffic. For example, the data in figure 3-7.12 from the Alberta study indicate that adding a recycling agent increased the penetration about 30 percent and thus was effective in restoring the binder. For those projects where no recycling agents were used, the HIPR process reduced the penetration about 20 percent.

A pavement that exhibits stripping may not be improved by hot in-place recycling, even though the initial coating may appear to be adequate. Test results show that there is often a loss in resistance to water damage after hot in-place recycling,[4,22] probably because the binder is usually softened, thus resulting in a more water susceptible mixture. Therefore, it may be prudent to consider anti-stripping measures such as liquid agents or lime as additives.

The early trials using the Martec HIPR equipment in British Columbia (B.C.) have provided an opportunity to assess the process. A good project for this evaluation of mixture properties was a section of Highway 1A in Abbotsford, British Columbia, recycled in October 1994. Because a recycling agent was not used, the properties of the asphalt mixture and binder could be compared before and after hot in-place recycling. The process was selected by B.C. Ministry of Transportation and Highways because the pavement was rutting prematurely after only 1 year of service. Both the B.C. Ministry and a consultant conducted tests.

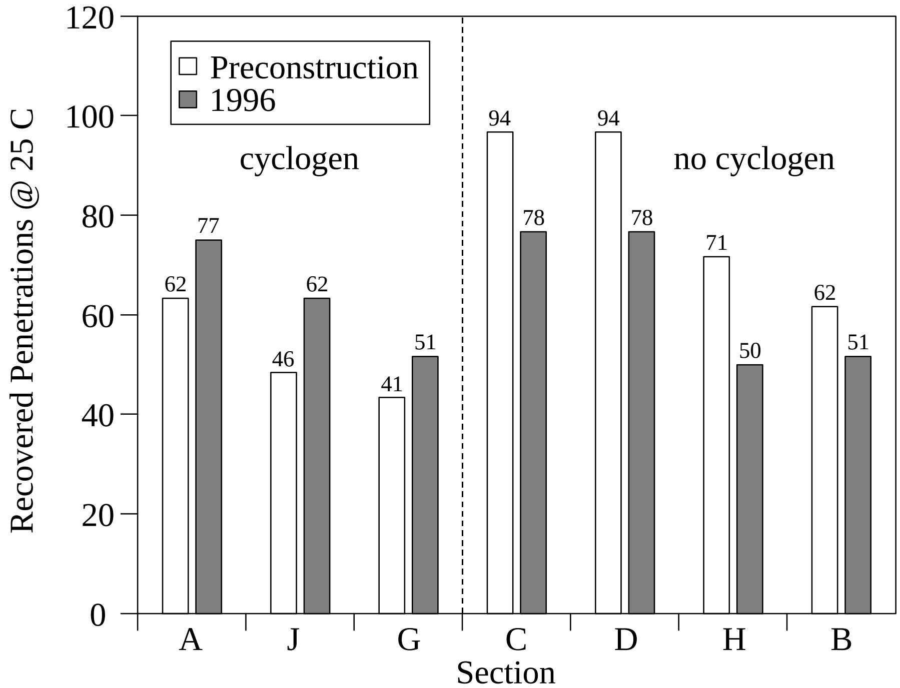


Figure 3-7.12. Effect of recycling agent on HIPR pavements.[19]

Table 3-7.1 shows a summary of data from the Abbotsford project. The data from core samples taken at four locations show that the air voids increased an average of 0.7 percent, intended to provide space for future traffic compaction. The gradation of the aggregate after hot in-place recycling was finer, but the dust (<0.075 mm) increased only 0.4 percent. As would be expected, the asphalt content did not change and the viscosity increased only slightly, from 395 to 318 mm^2/sec, while the penetration decreased only one point (dmm). This value can be compared to the 20 percent loss in penetration as shown in figure 3-7.12. These data would indicate that the hot air/IR heating was relatively gentle and that the age-hardening of the binder was insignificant.

8. ECONOMICS

A limited amount of comparative cost information is available in references 4 and 8. Because of different processes, equipment and pavement/project conditions, comparisons between hot in-place recycling and conventional rehabilitation alternatives are difficult. Some typical first costs are given on table 3-7.2.

First costs are important, but life-cycle costs must also be considered. Reference 8 indicates that States have not developed life-cycle costs. A few life-cycle cost examples have been prepared and are contained in reference 4. Favorable first costs and life-cycle costs are possible, depending upon the project particulars. First cost savings of from 5 to 50 percent have been reported.[8]

9. GUIDELINES FOR USE

Guidelines for selecting hot in-place recycling as a rehabilitation alternative can be found in references 4, 7, 8, and 21.

Table 3-7.1. Summary of test data from trial project at Abbotsford, B.C., Canada, October 1994[1] (See references 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, and 20.)

Property	Before Recycling	After Recycling
From core samples:		
Thickness of top lift (mm)	54.0	56.0
Air voids (%)	3.3	4.0
Bulk specific gravity (kg/m^3)	2379.0	2387.0
Compaction (% of Marshall)	100.3	98.5
Marshall samples:		
Bulk specific gravity (kg/m^3)	2415.0	2432.0
Theoretical max specific gravity (kg/m^3)	2506.0	2498.0
Air voids (%)	3.6	2.6
Stability (N)	10,072.0	10,105.0
Flow (0.25 mm units)	9.3	10.2
Aggregate gradation:		
Sieve size (mm)		
19.00	100.0	100.0
12.50	92.3	94.9
9.50	81.5	82.4
4.750	63.8	64.3
2.360	50.4	49.4
1.180	35.9	36.8
0.600	24.6	25.9
0.300	12.8	14.0
0.150	7.1	8.3
0.075	5.2	5.6
Asphalt binder:		
Asphalt content (% total mix)	5.0	5.0
36.8 Recovered (Abson) asphalt properties Kinematic viscosity @ 135EC (mm^2/sec)	305	318
Penetration @ 25EC (dmm, 100g/5 sec)	36	35

[1]Average of four sets of core samples from four locations on the four-lane highway.

Table 3-7.2. Typical cost information.[4,8]

Hot-In Place Recycling Operation	Approximate Cost, Dollars Sq Meter
Heater-scarification (25 mm + recycling agent)	1.20
Heater-scarification + 25 mm overlay	3.17
Repaving (recycle 25 mm + 25 mm hot-mix asphalt mixed together)	3.50
Remixing (recycle 25 mm + 10-20 percent new hot-mix asphalt)	2.75
Remixing (recycle 50 mm + 10-20 percent new hot-mix asphalt)	3.25

The major considerations that must be taken into account when designing a mixture for a hot in-place recycling project are as shown below:

- Uniformity.
- Depth of HMA.
- Presence of chip seals.
- Asphalt content (bleeding).
- Aggregate gradation.
- Asphalt properties.
- Traffic.
- Types of pavement distress.

The uniformity of the existing asphalt bound materials within the project limits must be established. The construction, rehabilitation and maintenance records should be consulted to determine the uniformity along the project and with depth. A single project may require several mix designs if the existing materials are not uniform.

Uniformity with depth is an important consideration. The specified recycling depth is the depth of the material to be used for mixture design. If nonuniform materials are found at depth, the mixture design phase of the project may require that a change in the specifications be made.

The presence of chip seals or other types of seals within the depth of recycling may require special mix design considerations. The uniform gradation of the chip aggregate and its relatively high asphalt content must be taken into account.

Hot-mix asphalt pavements experiencing bleeding may require the addition of new hot-mix or aggregate of a gradation that will create voids in the mixture. The gradation of the aggregate may have to be altered to improve stability, increase air voids, provide skid resistance or other factors. New aggregate is typically precoated with a new asphalt binder.

The properties of the binder in the existing asphalt bound pavement must be determined to provide information for the selection of the characteristics of the new binder and the amount of new binder to be added. The physical properties of viscosity and penetration are normally determined. Binder physical properties associated with PG graded binders will be used in the near future.

The traffic that will use the pavement should also be considered in the design process. High volumes of heavy traffic will require mixtures with high stability and resistance to permanent deformation.

The type of pavement distress occurring in the existing pavement may indicate deficiencies in the hot-mix asphalt to be recycled. A pavement with rutting and corrugations indicates that the mixture in its present condition is unstable. A pavement with alligator cracking indicates a structural deficiency that most probably should be corrected with additional thickness. Transverse cracking may indicate that the asphalt binder is excessively stiff for the environment in which it has been placed. Altering the physical and chemical properties of the binder during the recycling process may be required.

The type, extent, and severity of distress will therefore determine if hot in-place recycling should be selected as a rehabilitation alternative. The performance of hot in-place recycling is also dependent upon the type, extent, and severity of distress in the existing pavement. Guidelines for predicting performance of hot in-place recycling based on these criteria in an existing pavement are not available in the literature.

The above factors affect the mixture design and structural design considerations. Other considerations include the time required for rehabilitation, project and cross section geometry, thickness of asphalt abound materials and presence of manholes, utility covers, vegetation, etc.

10. SPECIFICATIONS

Specifications for hot in-place recycling operations are available from several States, ARRA and reference 4. The key sections of the hot in-place recycling guide specifications include a description of the process, materials required, mixture design information requirement, equipment, construction operation, quality control and quality assurance, measurement and payment. Comments on key factors for some of the specifications sections follows.

A general description of the recycling process to be utilized should be included in the specifications. The capability of the equipment to heat and to scarify/mill to a specific minimum depth should be included. A specification reference for the recycling agent, such as ASTM D 4552 and other new materials, such as antistrip agent and new hot-mix asphalt, should be included.

The party responsible for conducting the mixture design should be designated (contractor or public agency). The information required for the mixture design submittal should also be identified.

A description of the recycling equipment may be needed in the specifications: equipment capability for heating, removal; and distribution of recycling agent, antistrip agent and new hot-mix asphalt. Spreading and leveling unit, and compaction equipment descriptions should also be considered for inclusion. This section of the specifications should not prevent capable equipment of performing the work, but should be prepared to insure that a high degree of success can be expected if the equipment specified is utilized.

Specify recycled mixture temperatures behind the recycling equipment, but prior to compaction. Since heating should not adversely harden the asphalt, an extracted and recovered penetration, viscosity or dynamic shear rheometer value could be specified. Air quality requirements should be identified and required safety equipment defined.

The depth of pavement removal and the test method to determine this depth should be defined in the specification. Criteria for acceptance complete with point of sampling and test method, should be specified. A section on measurement and payment is appropriate.

11. QUALITY CONTROL/QUALITY ASSURANCE

Quality control/quality assurance requirements for hot in-place recycling should be similar to those for conventional hot-mix asphalt. The quality of hot in-place recycling projects is very dependent upon the uniformity of the existing pavement (along with pavement, across the pavement and with depth). Therefore, the specification requirements and pay factors may have to be adjusted for this variability.

Key items that should be considered for inclusion in QC/QA specifications include asphalt binder content, in-place density, laboratory molded density, smoothness and depth of recycling. QC/QA specifications with pay factors for hot in-place recycling are not widely used at this time. Project variability needs to be defined, test methods developed, etc.

12. SUMMARY

Through the past 20 plus years, the concept of hot in-place recycling has grown steadily; although more rapidly in the late 1980s and early 1990s. Early attempts were limited by equipment. But the development of companion technologies such as cold milling and hot central plant recycling have added to the knowledge base and spurred equipment and materials improvements that are useful to hot in-place recycling.

Through the early 1990s, the techniques and equipment developed by the pioneers in organizations like Artec, Pyrotech, Wirtgen, Cutler, Jackson, and Taisai, have incrementally approached the goal of being able to recycle at depth and put down an acceptable high quality asphalt pavement that can directly compete with other HMA pavements.

Guidelines for selecting suitable hot in-place recycling projects are important so that the process is used where most effective. Further education and training will be needed to help expand the industry. This is being accomplished through the efforts of manufacturers, ARRA and other associations, highway agencies, contractors, and others. An important group that will be working toward standardization is Task Force 40 of the AASHTO-AGC-ARTBA Joint Committee.

13. REFERENCES

1. Epps, J.A., “Recycling Materials for Highways,” NCHRP Synthesis 54 (1978).

2. Epps, J.A., D.N. Little and R.J. Holmgreen, “Guidelines for Recycling Pavement Materials,” NCHRP Report 224 (1980).

3. Asphalt Recycling and Reclaiming Association, “Hot In-Place Recycling — First in the Line of Pavement Maintenance,” Hot In-Place Recycling Technical Committee, Annapolis, MD (1992).

4. “Hot In-Place Asphalt Pavement Recycling — A Technical Training Seminar and Workshop,” Pyrotech Asphalt Equipment Manufacturing Company, British Colombia.

5. Rogge, D.F., W.P. Hislop and D. Dominick, “Oregon’s Hot In-Place Recycling Guidelines,” Better Roads (July 1996).

6. Terrel, R.L., J.A. Epps and J.B. Sorenson, “Hot In-Place Recycling,” Symposium on Recycling, Association of Asphalt Paving Technologists (March 1996).

7. “Western States Round Table — Hot In-Place Asphalt Recycling,” Conference Proceedings, Report No. 1049-1, Nevada Transportation Technology Transfer Center, University of Nevada (July 1995).

8. Button, J.W., D.N. Little and C.K. Estakhri, “Hot In-Place Recycling of Asphalt Concrete,” NCHRP Synthesis 193 (1994).

9. National Asphalt Pavement Association, “Hot Recycling of Yesterday,” Recycling Report, Vol. 1, No. 2 (September 1977).

10. Whitney, G.F., “America’s Recycling Future AARA’s View,” Rural and Urban Roads (March 1992).

11. Rathburn, J.R., “One-Step Repaving Speeds Country Work,” Roads and Bridges (March 1990).

12. “Seminar on Hot In-Place Recycling,” various authors, WSU Roadbuilders Clinic, Washington State University, Pullman, WA (March 2-4, 1993).

13. Haughton, D.R., “Performance and Economics of Hot In-Place Recycling in British Columbia,” Ministry of Transportation and Highways, Victoria, B.C. (March 1993).

14. Gavin, J. and C. McMillan, “Alberta Transportation and Utilities Experience with Hot In-Place Recycling,” Proceedings, Canadian Technical Asphalt Association (1993).

15. Kazmierowski, T.J., A. Bradbury and P. Marks, “Seven Years of Experience with Hot In-Place Recycling in Ontario,” Proceedings, Canadian Technical Asphalt Association (1993).

16. Rogge, D.F., W.P. Hislop and D. Dominick, “Exploratory Study of Hot In-Place Recycling of Asphalt Pavements,” Transportation Research Report 94-23, Transportation Research Institute, Oregon State University, Corvallis, OR (November 1994).

17. Terrel, R.L., “Artec AR2000 Super Recycler for HIPR,” letter from Terrel Research, Edmonds, WA (December 9, 1994).

18. Bandyopadhyay, S.S., “Structural Performance Evaluation of Recycled Pavements by Using Dynamic Deflection Measurements,” Transportation Research Record No. 888, Transportation Research Board, Washington, DC (1982).

19. Development of Guidelines for the Design of Hot In-Place Recycled Asphalt concrete Mixtures,” Report EA-13745, AGRA Earth & Environmental Limited, Edmonton, Alberta (July 10, 1996).

20. Fyvie, K., "Observations, Comments and Test Data Related to the Operation of the Artec AR2000 Super Recycler Multi-Staged Hot In-Place Asphalt Recycling Process; South Fraser Way (Highway 1A) at Abbotsford, British Columbia, Canada," Letter Report, Terra Engineering Ltd., Vancouver, B.C. (December 9, 1994).

21. Emery, J.J., J.A. Gurowka and T. Hiramine, "Asphalt Technology for In-Place Surface Recycling Using the Heat Reforming Process," Proceedings, The 34th Annual Conference of Canadian Technical Asphalt Association, Vol. XXXIV (1989).

22. "Pavement Recycling Guidelines for Local Governments — Reference Manual," prepared by ARE, Inc. for FHWA, Report No. FHWA-TS-87-230 (September 1987).

MODULE 3-8

COLD IN-PLACE RECYCLING

1. INSTRUCTIONAL OBJECTIVES

This module presents information on cold in-place recycling of asphalt pavements. Cold in-place recycling (CIPR) techniques include full-depth and partial-depth. Upon completion of this module, the participant should be able to accomplish the following:

1. Identify the types of cold in-place recycling.
2. Define the types of equipment and their operational sequence for cold in-place recycling operations.
3. Describe mixture design procedures for use in cold in-place recycling operations.
4. Define structural layer coefficients for use in pavement design.
5. Describe performance of cold in-place recycled pavements.
6. Define existing economic information on cold in-place recycling operations.
7. Describe key elements of specifications and quality control/quality assurance guidelines.
8. Provide recommendations for appropriate use of cold in-place recycling techniques.

2. INTRODUCTION

As shown on figure 3-8.1, cold recycling may be performed in-place or at a central plant. Cold mix asphalt pavement recycling is the reuse of untreated base materials and/or asphalt concrete pavement that is either processed in-place or at a central plant with the addition of asphalt emulsions, cutbacks, portland cement, lime and/or other materials as required to achieve desired mix quality, followed by placement and compaction.(1) Cold in-place recycling, as compared to cold central plant recycling, is used the majority of the time.

Cold in-place recycling is defined as a process of correcting asphalt pavement distress by processing without heat and in-place, the existing pavement material(s) and combining as necessary with a stabilizing agent, recycling agent and/or aggregate. After processing and mixing, the material is placed and compacted. A surface or wearing course is typically applied to the surface.

Cold in-place recycled materials have been used for subbases, bases, and surfaces. The most common use to date has been for base courses. Although stabilization with bituminous materials is the most popular process, literature indicates that lime, portland cement, and calcium chloride have been used.(2)

Two forms of cold in-place recycling with bituminous binders have evolved in the United States: full-depth and partial-depth. Full-depth (reclamation/stabilization) cold in-place recycling is a rehabilitation technique in which the full flexible pavement structure and predetermined portions of the base material are uniformly crushed, pulverized, and mixed with a bituminous binder, resulting in a stabilized base course. Additional aggregate may be transported to the site and incorporated in the processing. This process is normally performed to a depth of 100- to 300-mm.(2)

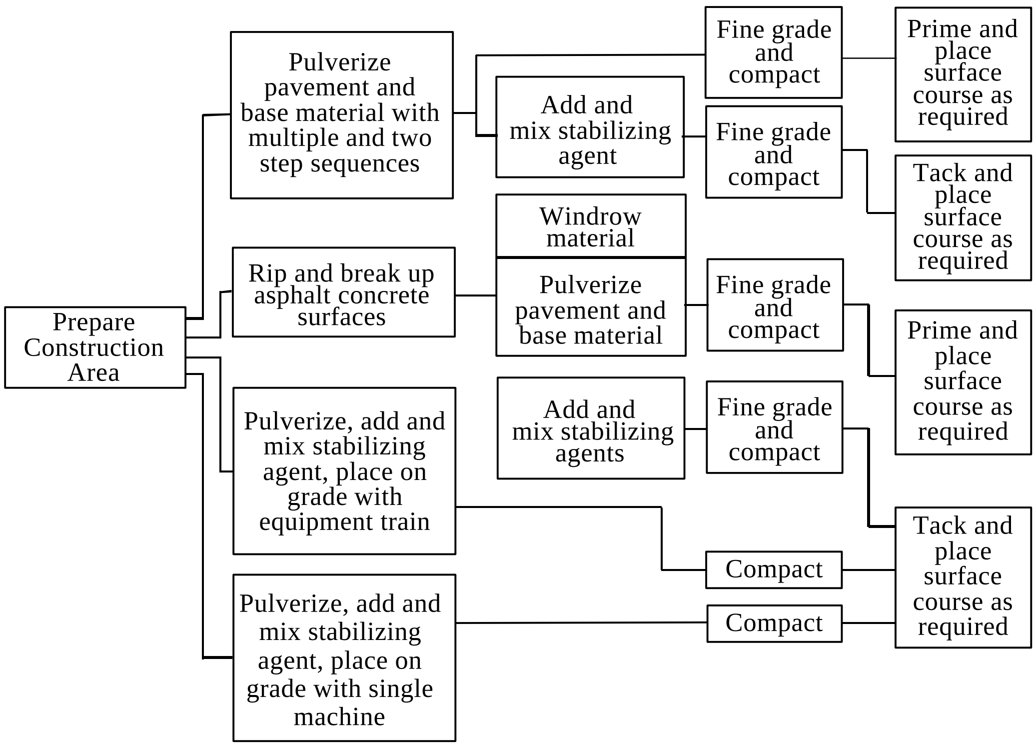


Figure 3-8.1. Full-depth cold in-place recycling.

Partial-depth cold in-place recycling is a rehabilitation technique that reuses a portion of the existing asphalt-bound materials. Normal recycling depths are 50- to 100-mm. The resulting bituminous-bound recycled material is often used as a base course, but can be used as a surface course on low-to-medium-traffic-volume highways. When this form of cold in-place recycling is performed on an old uniform pavement, a uniform, higher-quality end product is expected.[2]

3. BACKGROUND

The use of full-depth cold in-place recycling with bituminous binders probably dates to the 1910s, although available references indicate 1966. States with extensive experience with full-depth, cold in-place recycling include California, Indiana, Kansas, Michigan, Nevada, and New Mexico. A number of other States have completed numerous projects identified later.[2]

Partial-depth cold in-place recycling dates to 1980 with contract-size projects. The States of California, Kansas, Maine, Nevada, New Mexico, Oregon, and Pennsylvania have experience with this form of cold in-place recycling. Oregon has placed numerous projects.[2]

A nationwide survey of cold in-place recycling was conducted in early 1987 for Asphalt Recycling and Reclaiming Association (ARRA).[3] This survey did not differentiate between full-depth and partial-depth cold in-place recycling. ARRA received responses from all state highway agencies, as well as numerous counties, cities, and private contractors. Twenty-four states indicated use of cold in-place recycling, five states indicated that they have placed only experimental test sections, and the remaining 21 states did not use cold recycling. Several states, including California, Kansas, New Mexico, Oregon, and Pennsylvania, indicated that they have constructed numerous projects. Based on the ARRA survey,[3] county roads and secondary highways composed equal proportions of cold in-place recycling projects (31 percent of responses each). City street projects account for 19 percent and primary and interstate highways compose 12 and 7 percent shares, respectively.[3]

The literature indicates the use of cold in-place recycling for all types of roads and structural section components. However, some agencies restrict its use. Twenty percent of the ARRA reporting agencies restrict cold in-place recycling to rural areas; an additional 20 percent limit use to road with low traffic volumes. Most agencies limit the use of cold in-place recycling to base courses (95 percent). Of these base course projects, 12 percent placed fog, sand, or slurry seals as surfaces; 33 percent of the projects were surfaced with aggregate chip seals; and 50 percent were surfaced with an asphalt concrete. Three States use cold in-place recycling for shoulder reconstruction on interstate highways.[3]

A listing of the more comprehensive references on cold in-place recycling is given below. Those references dealing with full-depth cold in-place recycling are:

- NCHRP Synthesis 54 (1978).[4]
- NCHRP Recycling Guidelines (1980).[5]
- TRB National Seminar (1980).[6]
- Chevron Cold Mix Recycling Manual (1982).[7]
- The Asphalt Institute (1983).[8]
- Scherocman (1983).[9]
- FHWA (1987).[10]
- Wood L.E. (1988).[11]
- NCHRP Synthesis 160 (1990).[2]

References dealing with partial-depth cold in-place recycling are primarily those based on Oregon research and field experience. (See references 12, 13, 14, 15, and 16.)

NCHRP Synthesis 160[2] has been used as the base for the background for this module.

4. RECYCLING METHODS AND EQUIPMENT

Figures 3-8.1 and 3-8.2 describe construction operations associated with full-depth and partial-depth cold in-place recycling. A wide variety of equipment has been used. The type of equipment and the sequence of operations is largely dictated by the specifications, the contractor's experience, and the type of cold in-place recycling (full-depth or partial-depth).

Cold in-place recycling consists of nine identifiable operations:

- Pavement sizing.
- Addition of new aggregate.
- Addition of new asphalt/recycling agent.
- Mixing.
- Laydown.
- Aeration.
- Compaction.
- Curing.
- Application of wearing surface.

Many of these operations are combined with a single machine or operation, whereas others, such as “addition of new aggregate,” may not be necessary on some projects. For convenience of discussion, several of these operations have been combined.

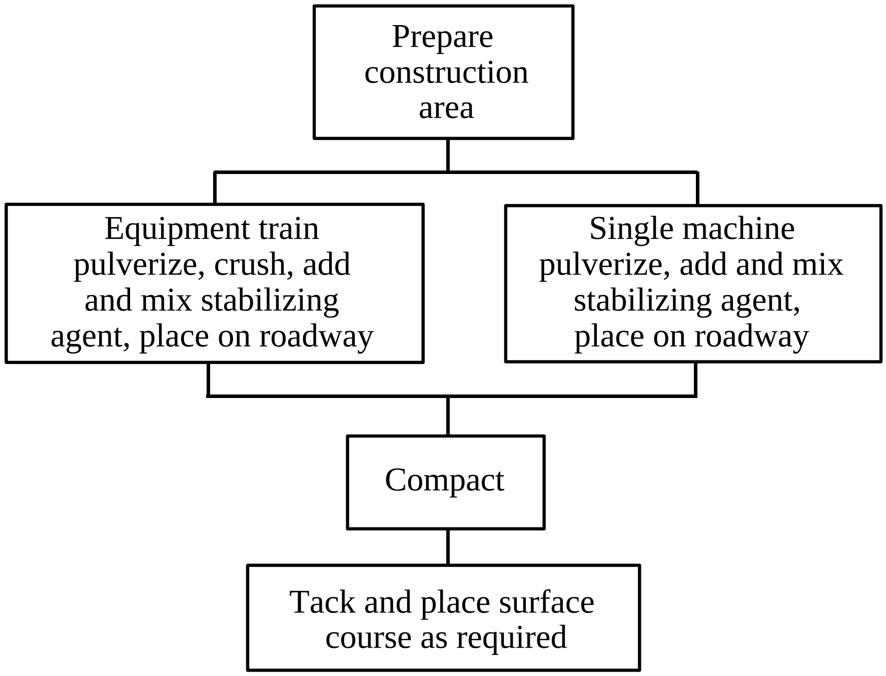


Figure 3-8.2. Partial-depth cold in-place recycling.

Sizing and Mixing Operations

The methods acceptable for in-place sizing and mixing for cold-recycling operations can be conveniently separated into four techniques:

- Multiple-step sequence.
- Two-step sequence.
- Single machine.
- Single-pass equipment train.

All of these methods are used for full-depth, cold in-place recycling; only the single machine and equipment train are used for partial-depth cold in-place recycling. These methods are briefly discussed below.

Two-Step Sequence

This method combines the breaking and pulverizing or sizing steps as described above into a single operation using a cold-milling machine (figure 3-8.3) or large pulverizing machine (figure 3-8.4). The stabilizer is then added and mixed in the second step. Common methods of adding stabilizers in this cold-recycling approach include the use of soil stabilization mixing equipment (figure 3-8.5) and traveling mixers (figure 3-8.6 and figure 3-8.7).

Figure 3-8.3. Cold-milling machine for pavement removal and sizing.

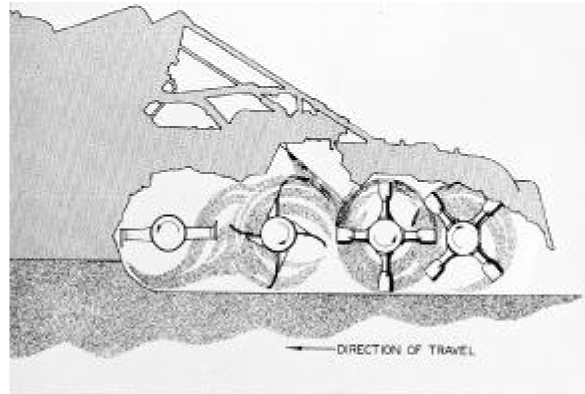


Figure 3-8.4. Large pulverizing machine for pavement removal and sizing.

Figure 3-8.5. Soil stabilization mixing equipment for adding stabilizer.

Figure 3-8.6. Traveling mixer adding stabilizer asphalt pavement form cold-milling machine.

Figure 3-8.7. Traveling mixer using windrow elevator for recycled asphalt pavement pickup.

Cold-milling machines have a rotating drum lined with a variable number (depending on width) of replaceable, tungsten-carbide-tipped cutting teeth to grind the old pavement. The advantages of cold-milling machines for breaking and pulverization include:

- Accurate control of depth and profile.
- Ability to pulverize and size in a single pass, resulting in less interference with traffic.
- Handling of conventional curb-reveal and other cold-planing work (i.e., use is not restricted to recycling).
- Use for mixing when fitted with pump and metering system.
- High productivity in almost any weather.

The disadvantages of cold-milling machines include the need for trained personnel to operate them and their relatively high cost of operation, which can make them uneconomical for use on seal-coat or thin plant-mixed asphalt roads. Care must be taken to ensure that all the pavement is reduced to the proper size and that the mix design takes into account the increase in the number of fines.

The drum of the cold-milling machine may be set to operate in an upcutting mode, in which the teeth cut from the bottom of the pavement layers upward as the machine moves forward, or in a downcutting mode, in which the teeth strike the top of the pavement surface in a downward direction as the machine travels ahead. For partial-depth cuts, the upcutting mode generally offers the most accurate cutting depth, with lower cost, greater speed, less tooth wear, less power to operate, and less damage to the underlying surface. However, upcutting can result in the production of significant amounts of oversize material. With downcutting, the reclaimed materials are pinched against the underlying layers, resulting in proper sizing. The productivity of a milling machine is a function of the resistance of the pavement material to the penetration of the cutting teeth. Three of the most important factors affecting this resistance are material quality, aggregate characteristics, and depth of cut.

Single Machine

Single-pass equipment capable of breaking, pulverizing, and adding stabilizers is used for both full-depth and partial-depth cold in-place recycling. Figures 3-8.8, 3-8.9, and 3-8.10 show large cold-milling machines capable of sizing and mixing in a single pass. These operations have the same advantages and disadvantages as cold-milling machines used for pavement removal only.

Figure 3-8.8. Cold-milling machine (RayGo).

Figure 3-8.9. Cold-milling machine.

Figure 3-8.10. Cold-milling machine
with portable crusher (CMI).

Single-Pass Equipment Train

Several contractors have developed a single-pass equipment train capable of full-depth and partial-depth cold in-place recycling. Large quantities of pavement can be recycled daily. Figure 3-8.11 shows an overall view of the equipment. The equipment train usually consists of a cold-milling machine (figure 3-8.12), portable crusher (figure 3-8.13), travel-plant mixer (figure 3-8.14), and laydown machine (figure 3-8.15). The oversize material from the milling operation is sized by the small, portable screen and crusher unit. The cold-milling machine's conveyor discharges the recycled asphalt pavement (RAP) into the crusher unit, which passes it over a screen with large sieve sizes (e.g., 38 mm). The particular sieve size will depend on the job specifications. The material retained on the screen is rerouted to the roll unit for crushing and then back to the screen. Eventually, 100 percent of the RAP will pass through the screen and onto another conveyor where it can be weighed before being deposited into the pugmill or a paver. The screen and crusher unit can also be fitted with a pugmill and asphalt feeder system for mixing. The recycled mix can then be windrowed directly behind or to either side of the mixer or, in some cases, directly into the hopper of a self-propelled asphalt laydown machine.

Comparison of Sizing and Mixing Operations

A partial list of advantages and disadvantages associated with each category of breaking, sizing, and mixing operation is given below:

- Multiple-step sequence.
    - Readily available equipment can be used.
    - Depth-control problems.
    - Removal of entire asphalt concrete layer is necessary.
    - Mixing of asphalt concrete and base.
    - Limited width operations.
    - Slow production rates.
    - Traffic control problems.
    - Construction coordination.
    - Aggregate oversize.

- Two-step sequence.
    - Depth limitations.
    - Partial-depth removal of asphalt concrete possible.
    - Aggregate oversize.
    - Specialized equipment.
    - High production capacities.

- Single machine.
    - Depth limitations.
    - Partial-depth removal of asphalt concrete possible.
    - Aggregate oversize.
    - Specialized equipment.
    - High production capacities.

Figure 3-8.11. Single-pass equipment train.

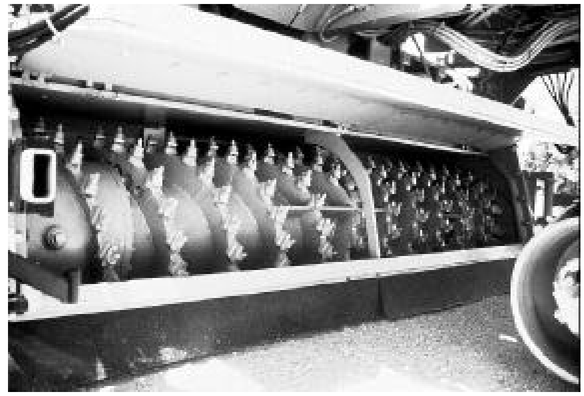


Figure 3-8.12. Teeth on drum of cold-milling machine.

Figure 3-8.13. Portable crusher attached to cold-milling machine.

Figure 3-8.14. Travel-plant mixer.

Figure 3-8.15. Laydown machine.

- Single-pass equipment train.
    - Depth limitations.
    - Partial-depth removal of asphalt concrete possible.
    - Aggregate gradation control.
    - Specialized equipment.
    - High production capacities.

Mixing Operations

Asphalt products used as modifiers in cold recycling include emulsified asphalts (usually either slow-setting or medium-setting), cutback asphalts, high-penetration asphalt cements (heated to a minimum temperature of 166°C for in-place recycling), and emulsified versions of commercial recycling agents. In addition, water may be added initially to help in the dispersion of the asphalt modifier during the mixing process. A small percentage of portland cement may also be added with emulsified asphalts to help stabilize the recycled mix and reduce curing time. The percentages of any added modifiers should be established in a laboratory mix design as discussed in a later section.

As with pavement removal and size reduction, there are several alternatives for mixing. There are four general types of soil-stabilization construction equipment that can be used for in-place cold recycling:

- Blade type.
- Flat type.
- Windrow type.
- Hopper type.

All of these equipment types are used for full-depth cold in-place recycling; however, the hopper-type mixer is most often used for partial-depth cold in-place recycling.

Blade Mixing

Blade mixing is the simplest method, but it usually is slow and inefficient (figure 3-8.16). The basic sequence involves:

- Using a motor grader to windrow the pulverized reclaimed material.
- Adding the prescribed amount of water (if required) to the windrow, preferably using a pressurized water truck rather than gravity flow for reasons of accuracy of application.
- Blading the windrow across the road with a rolling action to blend in the water.
- Reshaping into a windrow and adding the prescribed amount of asphalt modifier, normally in two or three passes, using an asphalt distributor.
- Using the grader to fold the material around the applied asphalt modifier, followed by working the mixture back and forth across the roadway surface until the modifier is uniformly distributed and proper fluids content is achieved.

If new aggregate is to be added, it should be windrowed next to the existing pulverized material and mixed in with the motor grader before water or modifier is added.

Figure 3-8.16. Blade mixing.

Figure 3-8.17. Single transverse-shaft rotary mixer.

Flat Type

Mixing operations are often performed with single (figures 3-8.17 and 3-8.18) and multiple (figure 3-8.19) transverse-shaft rotary (flat type) mixers. The asphalt modifier can either be applied to the windrowed material by an asphalt distributor before mixing, or it can be added directly by the mixer by means of a spray-bar in the cutting chamber fed by an asphalt supply tanker. With the spray-bar system, mixing can be combined with pulverization in a single-pass operation provided the recycled pavement is sufficiently reduced in size with one pass of the mixer. However, several passes of the machine are normally required to add the proper amount of asphalt and to achieve uniform mix quality. Typically, pulverizing and mixing are completed in separate passes.

Windrow Type

Windrow mixers can pick up the material from the grade and mix with parallel shafts (figure 3-8.20). These types of mixers are not commonly available today. Windrow, transverse-shaft mixers that do not elevate the material above grade are available. Improved quality control is normally obtained from those mixers that elevate the material above grade.

Hopper Type

Hopper type or travel-plant mixers are pugmill plants that can mix recycled pavement with liquid modifier, applied at a controlled rate, as they move along the road (figure 3-8.21). There are several options when using these mixer-pavers for in-place recycling. One is to have a windrow pickup attachment for loading the pulverized, recycled pavement directly from the roadway surface into the pugmill. The windrow type of equipment can use either parallel or transverse shafts. Another option is to feed the recycled pavement, new aggregate, or both, into the plant's aggregate receiving hopper. This requires an intermediate step of loading the recycled pavement into trucks by conveyor or other means. A third option, when using cold-milling machines, is to load the receiving hopper directly by means of a truck-loading conveyor set at the proper angle.

If water is required in addition to the asphalt modifier, a separate water-delivery system is required. Difficulties may arise if the recycled mix requires variable water content.

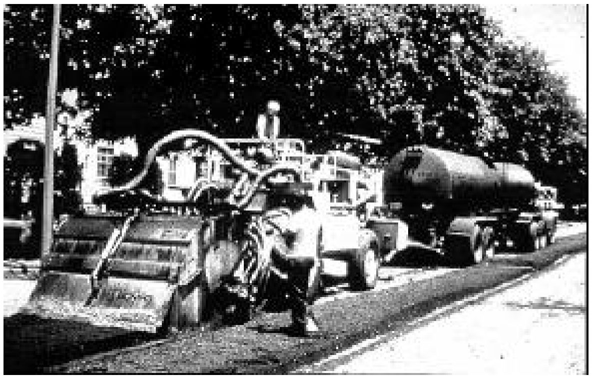


Figure 3-8.18. Single-shaft rotary mixer with asphalt supply tank.

Figure 3-8.19. Multiple transverse-shaft rotary mixer.

Figure 3-8.20. Parallel-shaft windrow mixer.

Laydown, Aeration, Compaction, Curing, and Surfacing

Construction techniques for in-place cold recycling laydown, compaction, and surfacing are the same as those used for conventional stabilization operations. Figure 3-8.22, figure 3-8.23, and figure 3-8.24 show typical operations. Proper curing prior to trafficking is important for the success of a project.

Figure 3-8.21. Hopper of mixer-paver receiving cold-recycled mix.

Figure 3-8.22. Laydown of cold-recycled base using Midland Motopaver.

Figure 3-8.23. Pneumatic-tired roller for compaction.

Figure 3-8.24. Hopper type paver for laydown of surface course over cold-recycled base.

5. RECYCLED MIXTURE DESIGN

The ARRA questionnaire[3] addressed the mix design process for cold in-place recycling, although it did not differentiate between full-depth and partial-depth processes. Public agencies obtained block, core, and loose-milled samples for mix design purposes. Sixteen percent of the agencies obtained block samples, 42 percent core samples, and 42 percent obtained samples from the milling operation. Sample location and frequency is based more on judgment than statistical procedures.

New Aggregate

The addition of new aggregate to the recycled pavement appears to be a widespread standard practice. Approximately two-thirds of the reporting agencies allowed for the addition of aggregate. The reasons cited for adding aggregate include providing additional thickness, correcting gradation, and providing for acceptance of additional binder. The new aggregate can be added in front of the pulverizing or milling machine or after partial pulverization; or the existing base course can be used with the pulverized asphalt concrete. The amount of new aggregate ranges from 15 to 50 percent and the amount of salvaged base ranges from 33 to 50 percent.[3]

Binder

Slow-setting and medium-setting asphalt emulsions are most often used for cold in-place recycling. Almost one-third of the respondents to the ARRA questionnaire use CMS-2 and CSS-1h. In general, the full-depth cold in-place recycling operations use the slow-setting emulsions, whereas the partial-depth operations have used medium-setting emulsions. High-float emulsions have also been used on several projects–full-depth and partial-depth. The New Mexico State Highway Department has used a high-float emulsion with a polymer additive to reduce thermal cracking, resist rutting and provide improved early strength. The western United States uses emulsified recycling agents proposed by the Pacific Coast User-Producer Group, among other types of binders.[16] Cutbacks and soft asphalt cements are used by some agencies. The type and amount of diluent should be known by the engineer before any of these liquid asphalts is used.

Amount of Binder

The amount of binder for cold in-place recycling generally ranges from 0.5 to 3 percent emulsion, with 0.5 to 1.8 percent suggested by Oregon, and 1.2 to 1.5 percent in Pennsylvania, as starting points for mixture design. This equates to 0.3 to 2 percent residual asphalt for emulsions. States using full-depth cold in-place recycling operations usually require binder contents at the upper end of this range, whereas the partial-depth operations usually use less than 2 percent emulsions. One-third of the respondents to the ARRA questionnaire use laboratory mix design procedures to determine binder and additive content.

Mix Design

The Marshall mix design procedure was used by 20 of 30 agencies using mix design procedures. The Hveem resilient modulus and indirect tensile tests were used by the other agencies. One-fourth of responding agencies reported relying on field workability or experience for determining binder content.[3]

Eighty percent of reporting agencies analyze the recycled pavement for asphalt content and aggregate gradation. Sample preparation was performed by processing or crushing in the laboratory (47 percent), heating and breaking of bulk samples (22 percent), and use of samples from field pulverized or milling operations (31 percent).[3]

Agencies have used Marshall compaction (50 and 75 blows), kneading, and gyratory method of compaction. Curing after compaction varies among agencies and ranges from 1 hour to 7 days. Curing temperatures among agencies range from 23 to 121°C. Density, stability, and air voids are frequently used to select binder contents.[3]

States, agencies and groups that appear to have the most developed mix design procedures for cold in-place recycling include:

- California.
- Chevron.
- Corps of Engineers.
- Nevada.
- New Mexico.
- Oregon.
- Pennsylvania.
- Purdue.
- Texas.
- The Asphalt Institute.

Methods most applicable to the full-depth process include those developed by Chevron, Corps of Engineers, Pennsylvania, Purdue, Texas, and the Asphalt Institute. The procedure developed by Oregon is for partial-depth recycling, whereas those developed by California, Chevron, Nevada, New Mexico, and Pennsylvania can be used for partial-depth cold in-place recycling. In addition, experience gained by Canessa has proved to be of value.

A standard national method is not available; however, certain basic steps are normally included in the mix design process. These include:

- Obtaining representative field samples from the pavement or from stockpiles of reclaimed materials.
- Processing of field samples for use in mix design.
- Evaluation of recycled pavement.
  - Asphalt content.
  - Asphalt physical properties (penetration, viscosity).
  - Aggregate gradation.
  - Recycled pavement gradation.
- Selection of amount and gradation of new aggregate.
- Estimate of asphalt demand.
- Selection of type and amount of recycling agent.
- Mixture, compaction, and testing of trial mixture.
  - Initial cure properties.
  - Final cure properties.
  - Water sensitivity.
- Establishment of job mix formula.
- Adjustment in field.

Methods proposed by California, Chevron, Oregon, Pennsylvania, and the Asphalt Institute are reviewed in reference 2.

These methods provide approaches for selecting the type and amount of binder and the amount of water. Methods of compaction, curing and testing differ. Most methods define mix property measurements soon after compaction, at or near a final cure condition, and after exposure to water. These are desirable properties from a rational mixture design approach.

A good cold in-place recycling binder is recognized as one that (a) produces initial softening of the RAP asphalt, (b) has good initial coating of RAP and new aggregate at low fluids contents, (c) allows for early compaction and traffic, (d) is relatively insensitive to binder content, and (e) does not continue to soften for several months to create rutting and bleeding problems. The grade of binder is chosen to soften the RAP asphalt to a selected level. Depending on environmental conditions, complete mixing of the new binder and the RAP binder may or may not occur in a timely manner. If complete mixing does not occur, it is possible that the new binder or recycling agent will remain on the surface of the hard RAP aggregate and create an unstable mixture. In selecting recycling agents, it is better to err on the hard side of the residual asphalt (high viscosity or low penetration) in the emulsified and asphalt cements.

6. PERFORMANCE

Comprehensive nationwide information on performance of cold in-place recycling is not available. The FHWA-sponsored research project to define performance of recycled pavements at Iowa State University and ARE, Inc. is limited in its evaluation of cold in-place recycled pavements. Reports that define performance of cold in-place recycled pavements are available in the literature; however, they do not use a common method of defining performance nor do they provide an equal amount of project detail. A summary of information from California, Indiana, Kansas, Maine, Nevada, New Mexico, Oregon and Pennsylvania is presented in reference 2. These performance studies have identified advantages and problem areas associated with cold in-place recycling.

The benefits most often cited by those using cold in-place recycling, regardless of the form (full-depth or partial-depth), include:[2]

- Significant pavement structural improvements may be achieved without changes in horizontal and vertical geometry and without shoulder reconstruction.
- All types and degrees of pavement distress can be treated.
- Reflection cracking normally is eliminated if the depth of pulverization and reprocessing is adequate.
- Pavement ride quality can be improved.
- Hauling costs can be minimized.
- Old pavement profile, crown and cross slope may be improved.
- Production rate is high.
- Only thin overlay or chip seal surfacing is required on most projects.
- Engineering costs are low.
- Aggregate and asphalt binder are conserved.
- Energy is conserved.
- Air quality problems resulting from dust, fumes and smoke are minimized.
- It is a cost-effective solution for a number of situations.
- Frost susceptibility may be improved.
- Pavement widening operations can be accommodated.
- It is environmentally desirable, because disposal problems are eliminated.

Identified problem areas with cold in-place recycling include:[(2)]

- Construction variation is larger for in-place versus central plant operations. (Partial-depth cold in-place recycling can result in a uniform pavement layer.)
- Curing is required for strength gain.
- Strength gain and construction are susceptible to climatic conditions, including temperature and moisture.
- Traffic disruption can be greater relative to other rehabilitation alternatives. (The use of the recycling train greatly reduces traffic disruption.)
- Placement of a wearing surface is required.

7. ECONOMICS

Table 3-8.1 gives a summary of agency costs associated with cold in-place recycling operations.

Detailed performance histories are not available to allow for the time scheduling of rehabilitation and maintenance operations. Thus, detailed life-cycle cost information does not appear in the literature. However, preliminary performance information obtained from State records indicates that significant life-cycle cost savings will be obtained when comparisons are made between conventional overlay techniques and cold in-place recycling operations. In some instances, the first cost of cold in-place recycling will be greater than conventional overlays; however, improved performance and the use of stage construction techniques with the cold in-place recycling option will lower life-cycle costs.

A review of the FHWA Demonstration Project 39 reports, as well as other information, indicates the following component costs for cold in-place recycling operations:

- Materials - 46.6 percent.
- Equipment - 29.7 percent.
- Labor - 23.7 percent.

The main economic advantage that recycling offers is in material cost savings. The majority of the material costs are associated with new binder. Increases in new aggregate will increase recycling costs. Typical square yard costs for cold in-place recycling operations are shown in table 3-8.1.

First costs of cold in-place recycling operations are project dependent, and, hence, generalizations should be used as guides only. First-cost savings of 6 to 67 percent are reported in the literature (table 3-8.1).

8. GUIDELINES FOR USE

Cold in-place recycling is a very versatile rehabilitation alternative for pavements with low to moderate traffic. Significant structural improvement can be achieved with full-depth cold in-place recycling operations without adversely affecting the horizontal and vertical geometry of the pavement. The full-depth option of cold in-place recycling has the ability to treat all forms of distress as the entire asphalt-treated portion of the pavement is pulverized and recycled. This full-depth recycling eliminates reflection cracking.

Table 3-8.1. Full- and partial-depth cold in-place recycling cost differences.[(2)]

Agency	Year	Cost Difference (%)[1] Range	Cost Difference (%)[1] Rep. Value	Cost of Cold In-Place Recycling ($) Range	Cost of Cold In-Place Recycling ($) Rep. Value	Comments
California (99)[4]	1979-83	15-43	31	16.20/Mg 26.80/Mg	22.22/Mg	Relative to conventional mix
California (40)			37		24.25/Mg	
California (46)	1980		21		6.18/m^2	Relative to equivalent section
Illinois (56, 57)	1982				4.55/m^2	
Indiana (64)	1976			13.17/Mg 24.25/Mg		
Iowa (65)	1988	67			7.60/Mg	Relative to asphalt concrete
Kansas (70)	1977	53				Relative to equivalent section
Kansas (172)	1988					
Missouri (181)	1978	50				Relative to equivalent section
Montana (86)	1978	21			23.80/Mg	
New Mexico[2]	1984-86			1.25/m^2 2.40/m^2	1.67/m^2	Average 66 mm of recycling

[1]Relative to commonly used rehabilitation alternatives used by identified States.
[2]Personal communication with D. Hanson (1987).
[3]Cost increase on one project.
[4]References are identified in reference 2.

Table 3-8.1. Full- and partial-depth cold in-place recycling cost differences.[(2)] (continued)

Agency	Year	Cost Difference (%)[1] Range	Cost Difference (%)[1] Rep. Value	Cost of Cold In-Place Recycling ($) Range	Cost of Cold In-Place Recycling ($) Rep. Value	Comments
N. Carolina (180)[3]	1977	6			4.77/m^2	Relative to equivalent section
Oklahoma (92)	1979				4.14/m^2	
Oregon (41)	1984	24		2.16/m^2 2.90/m^2	2.40/m^2	Relative to equivalent section
Pennsylvania (98)	1983	16				Relative to equivalent section
Vermont (107)	1978	28			9.45/m^2	Relative to equivalent section
Vermont (108)	1982	31			1.64/m^2	Relative to equivalent section
Wisconsin (111)	1978				0.014/m^2-mm	
FHWA (114)					5.65/m^2	

[1]Relative to commonly used rehabilitation alternatives used by identified States.
[2]Personal communication with D. Hanson (1987).
[3]Cost increase on one project.
[4]References are identified in reference 2.

Cold in-place recycling is among the most economical forms of pavement recycling for a relatively large number of projects.

Cold in-place recycling operations depend upon a curing of the asphalt material or other stabilizing material to gain strength. This strength gain is, therefore, dependent upon the environment and the depth of recycling. Cold in-place recycling operations usually have difficulty obtaining density. A loss of strength and perhaps permanent deformation in the recycled layer can cause performance problems.

Construction of all in-place recycling operations is larger than central plant operations. Large variability in the quality of construction can cause performance problems. The placement of a wearing surface is usually required to achieve the desired performance of the rehabilitated pavement.

Cold in-place recycling is a viable rehabilitation alternative. Partial-depth, cold in-place recycling has a proven performance record in several States, including New Mexico and Oregon. Full-depth, cold in-place recycling is very economical and has a proven performance record, provided project selection has been performed appropriately.

Pavements that have shown poor performance can be traced to the following causes:

- Too high a recycle agent content in the early years. Contents more than 2 percent with 7 percent to 10 percent diluent were shown to create excessive softening.
- Placing a tight seal or dense-wearing course too soon, resulting in trapping water and diluent, followed by stripping and rutting.
- Depth of recycle stopped at a delaminated layer of old pavement, resulting in loss of bond.
- Failure to provide some type of seal before freeze/thaw conditions.

Pennsylvania has developed guidelines for this climate to define the use of cold in-place recycling. Table 3-8.2 contains these guidelines.(17)

Table 3-8.2. Guidelines for use of cold recycling in Pennsylvania.(17)

Average Daily Traffic	Wearing Surface
1,500 or less	Surface treatment (double application) as a minimum
1,500 to 3,000	Hot-mix wearing course
More than 3,000	Do not use cold recycling

9. SPECIFICATIONS

Specification and quality control procedures for cold in-place recycling operations which have been developed by a number of State highway agencies,(2) contain specifications developed by Michigan, Pennsylvania, and the Asphalt Recycling and Reclaiming Association and used primarily for full-depth cold in-place recycling operations. Specifications developed by Oregon and New Mexico are used primarily for partial-depth cold in-place recycling operations. These specifications contain elements of both method specifications and end-result specifications.

These types of specifications rely on the expertise of the public agency, contractors, material suppliers, and equipment manufacturers to obtain the desired end product at a reasonable cost. A discussion of the general elements of the specifications follows.

Most cold-asphalt recycling specifications tend to focus on the material properties of the reclaimed material and of the recycled mix rather than on the exact methods of construction. Specifications are often written to allow for the use of a wide variety of in-place equipment, provided the recycled mix meets the job specification for depth, maximum particle size and gradation. The typical specification for cold-mix recycling will contain sections on some or all of the following topics:

- Overall description of work.
- Materials (RAP, new aggregate, asphalt binder, water).
- Equipment.
- Method of construction:
    - Scarification and pulverization.
    - Addition of asphalt modifier and mixing.
    - Aeration.
    - Spreading.
    - Compaction.
- Approval of job-mix formula.
- Inspecting, sampling and testing.
- Quantity and basis of payment for each material.
- Wearing course (asphalt concrete, chip seal coat, slurry seal, etc.).
- General (weather, traffic control, safety, etc.).

10. MATERIALS

The primary materials specifications deal with aggregate gradation, asphalt binder type, asphalt binder content, water content (if applicable), and density requirements.

Maximum Size

Most specifications limit the top size of the RAP produced by the pulverization equipment. Some public agencies require 100 percent passing the 25.0 mm sieve. This type of specification can be overly restrictive (and can slow down the recycling process). The top size of milled or pulverized materials is a function of all of the following:

- Condition of existing pavement (if alligator cracking is present, oversized pieces will most likely be produced).
- Top size of original aggregate.
- Speed of milling (higher speeds produce larger sizes).
- Depth of cut (thicker cuts tend to produce chunks of greater size).

The preferred alternative is to allow some oversized material to be present in the recycled mix by specifying a minimum percentage for the nominal maximum size instead of placing restrictions on top size (e.g., 97 percent passing the 38 mm sieve) with no chunks larger than 100 mm or with the remaining 3 percent, not so large as “to affect adversely the stability and structural integrity of the mixture, nor to hamper the shaping operation.”[9]

Aggregate Gradation

It is not practical to have the aggregate gradation of the RAP specified for all sieve sizes because of the variability associated with the pulverization process. However, consideration should be given to the amount passing the 75 µm sieve because milling tends to increase the filler content by two to three percentage points (a maximum of 12 percent is reasonable). Because of the variability of the material being cold-recycled, allowance must be made in the specification (from the viewpoints of both engineering and economics) such that the gradation reflects what is present in the roadway and not what the designer considered to be optimum values. In addition to meeting RAP gradation requirements, the scarification and pulverization equipment should also be capable of reasonable accuracy in cutting the existing pavement to a specified depth.

Asphalt Binder

The specified asphalt binder should conform to the appropriate standard AASHTO, ASTM, or State specifications for emulsified asphalt, cutback asphalt, emulsified recycling agents, viscosity-graded asphalt cement, penetration-graded asphalt cement or performance graded (Superpave PG grade) asphalt cement. Specifications for emulsified recycling agents have been developed by a committee of the Pacific Coast User-Producer Conference on Asphalt Specifications.

Binder Content

The equipment for adding the modifier should be capable of an accurate application rate such that the total binder content of the recycled mix is equal to the job-mix formula amount within a specified tolerance, typically ±0.5 percent. Provision should also be made for the accurate application of any required pre-mix water as specified by the job-mix formula.

Job-Mix Formula

The responsibility for establishing the job-mix formula and required sampling procedures, test methods, and design criteria for the mix design needs to be clearly outlined in the job specifications. The specifications for full-depth cold in-place recycling generally do not place limits on the amount of RAP in the mix, unless additional aggregate materials are required to increase the thickness of the stabilized layer. Extraction and recovery tests, which are part of the mix design process, can be used to determine if any new aggregate is needed to improve the quality of the RAP.

New Aggregate

If new aggregate is to be incorporated into the recycled mix, the aggregate should be tested for compliance with standard specifications for virgin aggregate such as an equivalent, resistance to abrasion, and so forth.

11. EQUIPMENT

Equipment specifications for the various phases of construction can be either the method or end-result type. The user agency's choice of which to use will depend on factors such as contractor and equipment availability, economics, and the desired quality of work.

12. DENSITY

A major item in the job specifications is the required density of the compacted mix, which can be specified in one of three ways:

- Percentage of theoretical maximum density.
- Percentage of laboratory density.
- Percentage of field density.

Some agencies recommend the use of percentage of theoretical maximum density instead of percentage of lab density. Agencies citing the problem with variation in the original pavement suggest that a target density (i.e., an actual density in lb/ft^3 or other units) combined with a rolling pattern that can be changed may be the most realistic type of density specification. This control-strip approach is used in Nevada and Pennsylvania.

The extent of agency experience with cold recycling and environmental factors will probably determine which type of density specification is appropriate. Typical specifications require air void contents in the 12 to 15 percent range.

13. SUMMARY

- Cold in-place recycling is a viable engineering and economic rehabilitation alternative for asphalt-surfaced pavements with moderate to low traffic volumes. Some States have, however, successfully used cold in-place recycling on interstate highways.
- Cold in-place recycling is a rehabilitation alternative that can be used to rehabilitate the pavement from the "bottom up" and thus can be used to strengthen a roadway with minimal change in the vertical cross section. This technique also lends itself to stage construction.
- Two forms of cold in-place recycling with asphalt binders have evolved in the United States: full-depth and partial-depth. The full-depth process uses the full flexible pavement structure and/or predetermined portions of base and/or new aggregate in combinations with an asphalt binder. The partial-depth process primarily uses existing asphalt-bound materials and typically recycles 50 mm to 100 mm in depth. Higher-quality, more uniform paving mixtures are usually produced from the partial-depth process.
- Mixture design methods have been developed for both full-depth and partial-depth cold in-place recycling operations. Emulsion contents of 0.5 to 3 percent are used. Quantities in the range from 0.5 to 1.5 percent are used for the partial-depth operations, whereas quantities in the range from 1.5 to 3 percent are used for the full-depth operations. Medium-setting and high-float emulsions and emulsified-recycling agents are typically used with the partial-depth operations. Slow-setting and medium-setting emulsions and soft asphalt cements are typically used with the full-depth operations. Mixing water contents are established as part of the design process.

Physical properties of cold in-place recycled materials are typically between those of an asphalt concrete mixture and a cold, asphalt-stabilized base material. Because properties vary from project to project, laboratory tests should be used to establish strength coefficients.

- Construction equipment and contractor capability are available to perform quality cold in-place recycled projects. Contractors have developed high-capacity, mobile equipment that creates minimum disruption to traffic during construction.
- Surfacing materials should be placed on all cold in-place recycled projects. Chip seals and hot-mixed dense and open-graded mixes are typically used as surface courses. Moisture contents in the recycled material should be reduced from 1 to 1.5 percent before placement of the surface. Summer curing of 7 to 14 days is typically required to achieve these moisture contents, depending on local climate.
- General performance data have been collected by several States. Overall performance has been very good on a large percentage of the projects. Some problems with raveling, rutting, and cracking have been noted. There is a lack of data on long-term performance.
- Adequate specification and quality control guidelines have been developed by State highway agencies.

14. REFERENCES

1. “Pavement Recycling Conditions for Local Governments - Course Outline,” prepared by ARE Inc. for FHWA, Report No. TS-87-230, September 1987.

2. Epps, J.A., “Cold-Recycled Bituminous Concrete Using Bituminous Materials,” NCHRP Synthesis 160, July 1990.

3. “Cold In-Place Recycling Across American,” Asphalt Recycling and Reclaiming Association (1988).

4. TRB, “NCHRP Synthesis of Highway Practice 54: Recycling Materials for Highways,” Transportation Research Board, National Research Council, Washington, DC (1978) 53 pp.

5. Epps, J.A., D.N. Little, R.J. Holmgreen, and R.L. Terrel, “NCHRP Report 224: Guidelines for Recycling Pavement Materials,” Transportation Research Board, National Research Council, Washington, DC (September 1980) 137 pp.

6. “Transportation Research Record 780: Proceedings of the National Seminar on Asphalt Pavement Recycling,” Transportation Research Board, National Research Council, Washington, DC (1980) 140 pp.

7. “Recycling Manual,” Chevron USA, In Asphalt Division (August 1982).

8. “Asphalt Cold-Mix Recycling,” The Asphalt Institute, Manual Series No. 21 (MS-21) March 1983).

9. Scherocman, J.A., “Cold In-Place Recycling of Low-Volume Roads,” in Transportation Research Record 898: Low-Volume Roads: Third International Conference, 1983, Transportation Research Board, National Research Council, Washington, DC (1983) pp. 308-315.

10. ARE, Inc., “Pavement Recycling Guidelines for Local Governments–Reference Manual,” Report No. FHWA-TS-87-230, U.S. Department of Transportation, Federal Highway Administration, Washington, DC (September 1987).

11. Wood, L.E., T.D. White, and T.B. Nelson, “Current Practice of Cold In-Place Recycling of Asphalt Pavements,” in Transportation Research Record 1178: Flexible Pavement Construction, Transportation Research Board, National Research Council, Washington, DC (1988) pp. 31-37.

12. Cooper, G.L., D. Dline, and D. Allen, “Cold Recycling Experiences in Arizona, Nevada and Oregon,” 12th Pacific Coast Conference on Asphalt Specifications (May 12-13, 1987).

13. “The Latest Programs in Paving Methods, Technology,” Highway and Heavy Construction (January 1988).

14. Allen, D.D., “Cold Recycle Projects on Oregon’s High Desert,” Oregon Department of Transportation, Salem, Oregon (December 1985).

15. Allen, D.D., R. Nelson, D. Thurston, J. Wilson, and G. Boyle, “Cold Recycling–Oregon 1985,” draft of technical report for Oregon State Highway Division (January 1986).

16. “Guide Specifications for Partial-Depth Cold In-Place Recycling Agents,” Pacific Coast User-Producer Conference (May 1989).

17. Kandhal, P.S. and W.C. Koehler, “Cold Recycling of Asphalt Pavements on Low Volume Roads,” paper presented to 4th International Conference on Low Volume Roads (August 1987).

MODULE 3-9

HOT CENTRAL PLANT RECYCLING

1. INSTRUCTIONAL OBJECTIVES

This module presents information on hot central plant recycling of asphalt pavements. Hot central plant recycling can be performed in batch or drum mix plants. Upon completion of the module, the participant should be able to accomplish the following:

1. Identify the types of hot central plant recycling.
2. Define the types of equipment and their operational sequence for hot central plant recycling operations.
3. Describe mixture design procedures for use on hot central plant recycling operations.
4. Define structural layer coefficients for use in pavement design.
5. Describe performance of hot central plant recycled pavements.
6. Define existing economic information on hot central plant recycling operations.
7. Describe key elements of specifications and quality control/quality assurance guidelines and
8. Provide recommendations for appropriate use of hot central plant recycling techniques.

2. INTRODUCTION

Hot central plant recycling is a process in which reclaimed asphalt pavement (RAP) materials, reclaimed aggregate materials (RAM) or both, are combined with new aggregate and/or asphalt, and/or recycling agents, as necessary, in a central plant blending and mixing operation to produce hot-mix asphalt paving mixtures. Hot central plant recycled materials can be used for base courses, intermediate layers, or surface courses. The finished product is generally required to meet standard materials specifications and construction requirements for the type of asphalt concrete mixture being produced. With equipment now available, all hot-mix producers can recycle using relatively inexpensive additions or modifications to their existing plants, or using plants designed specifically for recycling, without violating air quality regulations.

Small percentages of RAP are routinely recycled in both conventional batch and drum mix plants. Inclusion of high percentages of RAP require special mixture designs and often require specialized equipment. A definition for hot central plant recycling is given below:

- <u>Hot Central Plant Recycling</u>. The removal of a portion of an asphalt pavement with or without removal of underlying pavement layers (e.g., untreated based materials) that is processed by sizing, heating, and mixing in a central plant with additional components such as aggregate, asphalt, or recycling agents and then relaid and compacted.

The uses and benefits of hot central plant recycling include the following:[1]

- Surface and base structural problems can be corrected.
- Significant structural improvements can be obtained with little or no change in thickness. For example, untreated granular bases can be recycled into hot-mix asphalt (HMA) concrete and then placed back in the same thickness.
- Existing mix deficiencies, such as aggregate gradation problems, can be corrected.

Hot central plant recycling is a relatively proven technology and is much less experimental in nature than cold in-place and hot in-place recycling. Hot central plant surface and base recycling has been practiced for a number of years. In 1915, Warren Brothers practiced the recycling of asphalt paving surface into asphalt concrete using central plant operations, but very little experimentation was conducted from that time until 1974.[2,3] A widespread rebirth of central plant recycling occurred in 1974 because of the rapid increase in the price of asphalt cement, and increased costs of other construction materials and equipment. With the increased use of the drum mixer and its adaptability for recycling, the amount of hot central plant recycled material being produced has increased.

In the comparison of cold in-place recycling with hot in-place recycling, there are several advantages of hot central plant operations. Improved quality control can be obtained in terms of modifier and total binder contents, blending percentages of new and recycled aggregate, and mixture homogeneity. Processes involving the use of heat generally produce mixtures that do not have to be cured before obtaining near maximum strength. The process can be used to repair all types of pavement, including high-traffic volume facilities. With proper scheduling, it is possible to remove a section of pavement and replace it the same day, using recycled mixtures made with aggregate from the previous day's removal operation.

The disadvantages of hot-mix recycling when compared to in-place methods are that 100 percent RAP generally cannot be used in the recycled mix, additional costs result from transporting the material to and from the central plant, and traffic may be disrupted for longer periods of time.

3. BACKGROUND

FHWA's recent review of hot central plant recycling operations indicates that States remove about 50 million metric tons of asphalt pavements annually.[4] About 33 percent of this 50 million ton quantity is reused in hot central plant recycling operations. An additional 47 percent of the asphalt pavement removed is used in some type of highway application (unstabilized base, shoulder, erosion control, cold in-place recycling, hot in-place recycling, etc.).[4]

Recycled asphalt pavement comprises only about 4 percent of all HMA produced in the United States of American (450 million metric tons of HMA produced annually). Based on a survey of 17 States, 9 of the 17 States indicated that HMA containing RAP (any percentage of RAP) was less than 20 percent of their total production of hot-mix asphalt. Six States reported that between 20 and 50 percent of all hot-mix asphalt produced contained some percentage of RAP. One State (Florida) indicated that more than 50 percent of all hot-mix asphalt produced contained some RAP.[1] A 1986 survey performed by NAPA indicates that about 23 percent of all hot-mix asphalt produced contained RAP.[4,5] A comparison of the production surveys conducted in 1986[5] and 1992[4] indicates that the amount of RAP utilized in the United States has not increased during this period.

A review of the literature suggests that while some States have significant hot central plant recycling operations, other States perform little or no hot central plant recycling. Common reasons given for not using hot central plant recycling include:

- RAP variability.
- Uncertainty of blending of new binder and aged RAP binder.
- High variability of hot central plant recycled mixtures.
- Unavailability of performance information hot central plant mixture.

Information relative to these cited barriers are summarized below. Key references for hot central plant recycling are:

- NAPA Information Series 123.[6]
- FHWA Report TA 92-76.[4]
- NHI Reference Manual.[7]
- NCHRP Synthesis 54.[3]

4. RECYCLING METHODS AND EQUIPMENT

Four basic construction activities are required in the hot central plant recycling process:

- Removing the existing pavement.
- Preparing the RAP for hot-mix recycling, (stockpiling, and crushing).
- Processing the blend of old and new materials in a hot-mix plant.
- Placing the materials on the roadway.

The recently completed National Asphalt Pavement Association (NAPA) report[6] and other publications are summarized below and define the equipment and processes used for hot central plant recycling.

Pavement Removal

Much of the same equipment that is used for pavement removal and size reduction in cold recycling processes can also be used for hot recycling. The methods can be generalized as:

- Ripping and crushing. The pavement is removed and then reduced in size at another location by crushing.
- In-place removal and sizing. The pavement is reduced in size as part of the removal process.

In the ripping and crushing operation, earthmoving equipment, scarifiers, grid rollers, or rippers are used to break up the existing asphalt pavement, which is then loaded into trucks and hauled to a crushing site. The type of removal equipment that is used will depend on the maximum size of the RAP pieces that available crushing equipment can handle. For example, relatively large pieces from a simple ripping process may be suitable for a primary crusher, while the use of a grid, sheepsfoot, or similar type roller or dozer after ripping may be needed to reduce the size for acceptance by secondary jaw or roll crushers. On-grade pulverizers, such as a traveling hammermill, can also be used for preliminary crushing. Ripping and central plant crushing is usually economical only when the full depth of the asphalt-treated layer is to be removed. Its main advantage is that no investment in new or specialized equipment is required, but special care must be taken to minimize contamination of the RAP by underlying untreated base or subgrade materials. The disadvantages include increased time to complete the recycling process due to the need to crush the removed RAP, and greater potential for segregation of the material and congealing after stockpiling.

The alternative to ripping and crushing is to remove and size the RAP in-place, using equipment normally associated with in-place recycling such as single- or multiple-shaft rotary mixers (figure 3.9.1) and cold-milling machines. This type of procedure is primarily used on projects which require only

Figure 3-9.1. Milling machine.

partial depth removal. The major advantage of this method is that the RAP is reduced to a manageable size on the spot, and no further crushing, or only a minimal amount of crushing, is required at the plant site. The advantages of using cold milling include:

- High productivity in almost any weather.
- No heat, dust or toxic fumes.
- Usefulness for nonrecycling work (i.e., pavement profiling or texturing).

The disadvantages of this procedure include:

- Special attention needed to ensure that entire pavement is reduced to proper size.
- Increase in the amount of fines generated.

Crushing and Sizing

The degree to which reclaimed pavement materials must be processed after removal depends largely on the removal method and the requirements of the mix design. Asphalt pavement removed by ripping will have to be crushed and screened to reduce the maximum particle size to acceptable limits. The Asphalt Institute(1) recommends that at least 95 percent of the RAP pass the 50 mm sieve, while another guideline is that the RAP be processed such that 100 percent will pass the 38 mm sieve and 90 percent will pass the 25 mm sieve.(2) The National Asphalt Pavement Association(8) recommends that if the RAP contains aggregate that is larger than the maximum size permitted by the mix specifications, the material should be sized by screening or crushing to the maximum size of the comparable virgin mix (i.e., 19 mm maximum in mix specifications, crush to 19 mm), although larger sizes such as 38 mm or 50 mm are acceptable for thick base course mixes produced in drum-mix plant. Obviously, the size specification will depend on the characteristics of the particular job and economic considerations, since the larger the allowable crushed RAP size, the less expensive is the crushing process.

Different types of RAP crushing and sizing are available. Typical equipment presently available to the contractor is identified below:(6)

Rap Breaker. This type of equipment is not designed for extensive crushing and down-sizing of RAP. Occasional large pieces of RAP may pass through the cold feed. The RAP breakers on “lump breakers” are designed to handle these larger pieces. Scalping screens placed between the RAP cold feed and transfer belt conveyors can be used to divert the large size material to the RAP breaker.

Some RAP breakers resemble a small cold milling machine head, while others resemble a small roll crusher (figure 3-9.2).

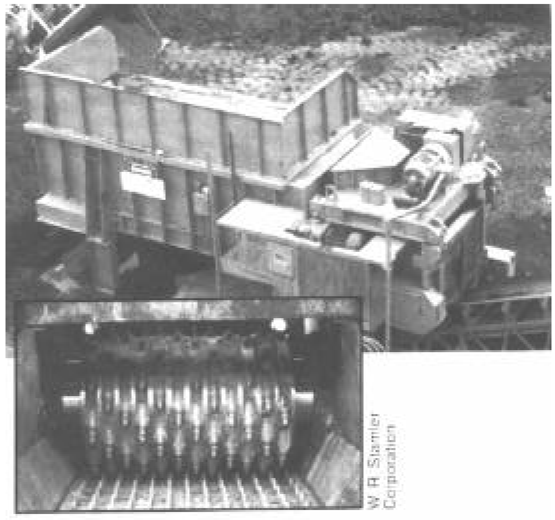


Figure 3-9.2. RAP breaker.[6]

Rap Crushers. Horizontal impact crushers, hammermill impact crushers, jaw/roll combination crushers, and milling/grinding reduction units have been used to reduce the size of RAP.

Horizontal impact crushers have solid breaking bars fixed to a solid rotor (figure 3-9.3). RAP is crushed as a result of impact with the breaking bars and a striker plate. Horizontal impact crushers can be used as either a primary or secondary crusher. Jaw primary and horizontal impact crushers are sometimes used to produce the desired RAP size reduction.

Hammermill impact crushers use breaker bars that pivot on a rotor, creating a wing-hammer type action (figure 3-9.4). The swing-hammer allows for “foreign material” to pass through the crusher unit without damage. Hammermill impact crushers can be used as either a primary or secondary crusher. Jaw primary and hammermill impact second crushers are used to process RAP.

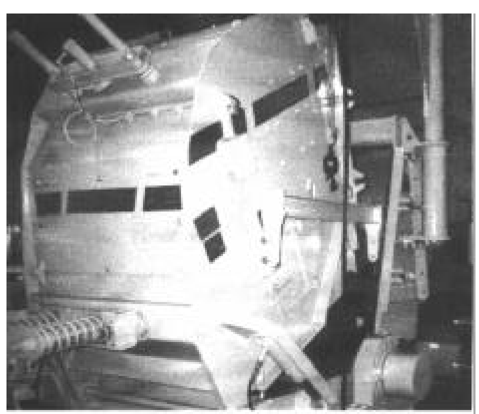


Figure 3-9.3. Horizontal impact crusher.[6]

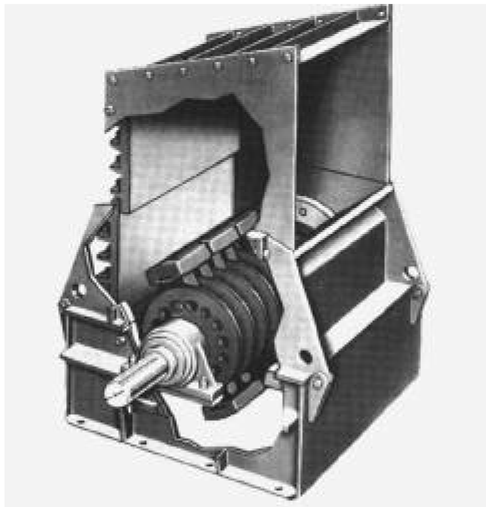


Figure 3-9.4. Hammermill impact crusher.[6]

Combinations of jaw and roll crushers are effective in reducing the size of RAP. The jaw crusher is used as a primary crusher to reduce large RAP "slabs." Secondary roll crushers are then typically used to produce RAP to minus 35- to 50-mm (figure 3-9.5).

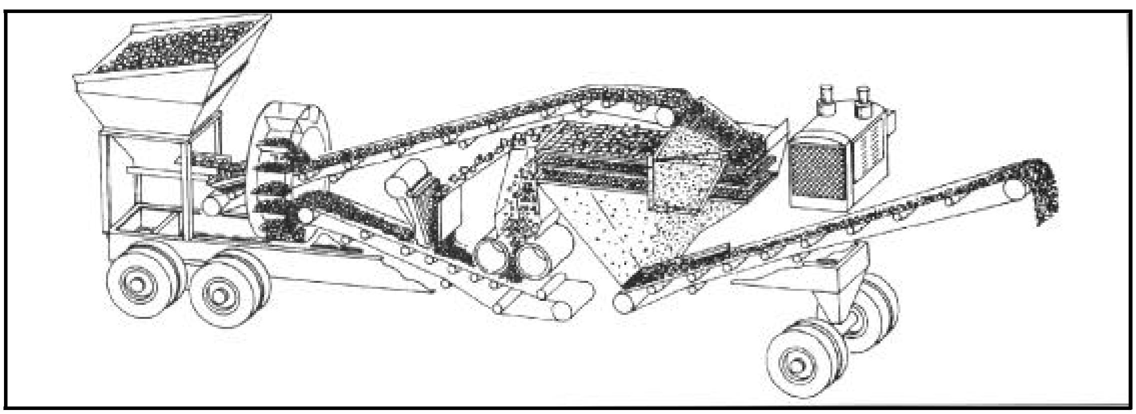


Figure 3-9.5. Jaw/Roll crusher combination.[6]

On hot days, both roll and jaw crushers can cause agglomeration or "pancaking" of RAP. The "pancake" will reduce production and may have to be removed. Horizontal impact and hammermill impact crushers typically have little agglomeration.

Crushing-sizing equipment that is infrequently utilized can be described as milling/grinding units. These units are not designed to reduce the stone size in the RAP, but to break the asphalt-aggregate bond or the asphalt binder.

Stockpile Operations

Literature of the 1970s and 1980s indicated that RAP stockpiles should be low in height as the RAP would agglomerate under its own weight. Experience has shown that stockpiles of considerable height do not agglomerate significantly. However, independent of stockpile height, a crust of RAP (200- to 250-mm) will form on the surface of the stockpile. A front-end loader usually can easily break through this crust. The RAP material inside the stockpile is easily handled.

The exterior crust on the stockpile is caused by heat generated by the air and the sun. The crust may help "shed" water from entering the stockpile (figure 3-9.6).

Figure 3-9.6. RAP stockpile.[6]

RAP stockpiles have a tendency to hold rain water and not drain like conventional aggregate stockpiles. Low height stockpiles (because of their relatively large surface area) tend to absorb larger percentages of water than stockpiles of considerable height. Water contents of 7 to 8 percent are not unusual in low height stockpiles. High moisture content RAP reduces production and limits the amount of RAP that can be recycled in a mixture.

Covering RAP stockpiles will reduce the water content in the material and increase production. Tarps or plastic covers should not be used as water will accumulate under the covers and increase the water content in the stockpiles. The use of open-sided buildings with a paved surface for the stockpile is the best method to use for controlling water content in stockpiles (figure 3-9.7).

Materials handling equipment should not drive directly on the stockpiles of RAP. The resulting compaction from this traffic will create agglomerations of RAP and handling problems.

Figure 3-9.7. RAP shed.[6]

Management of stockpiles is an important part of a successful hot central plant recycling operation. RAP stockpiles should be formed to ensure that the gradation of the aggregate, the type of aggregate, the amount of asphalt binder, and the hardness of the asphalt binder are as uniform as possible. When pavement is removed from large projects, single stockpiles of the RAP should be formed from single projects.

If small quantities of RAP are received from several sources or projects, the crushing and processing operation should be used to blend the materials and to ensure that uniform stockpiles of processed RAP are created.

Hot central plant recycling operations that use small percentages of RAP can tolerate more variability in the stockpile than recycling operations that use large percentages of RAP. The management of stockpiling operations to ensure material uniformity, low moisture contents, and a minimum amount of agglomeration is important for successful hot central plant recycling operations.

Cold Feed Operations

Conventional cold feed systems can feed RAP materials, but the best feeder bins are steep-sided. Longer feeder belts and larger openings onto the feeder belts are utilized. Since RAP has a tendency to agglomerate, the front-end loader operator must feed the cold feed bins slowly (an entire front-end loader bucket should not be dropped into the hopper). RAP cold feed bins should not be filled to capacity.

RAP should not remain in the cold feed bin for an extended period or agglomeration may occur. Pneumatic air “cannons” or “blasters” have been used to ensure the free flow of RAP from the cold feed bin to the collection belts. Vibrators on the bins tend to agglomerate the RAP and should not be used. Agglomeration problems are most severe on hot and humid days.

Mixing Equipment

In the 1970s and 1980s, several innovative approaches were developed to heat RAP in hot central plant facilities. Two different types of heat transfer techniques are primarily used to heat RAP in central plant facilities; conductive heat transfer and convective heat transfer. Conductive heat transfer occurs when two materials of different temperatures are in contact with each other. Convective heat transfer occurs when solid particles are exposed to a hot gas stream (figure 3-9.8).

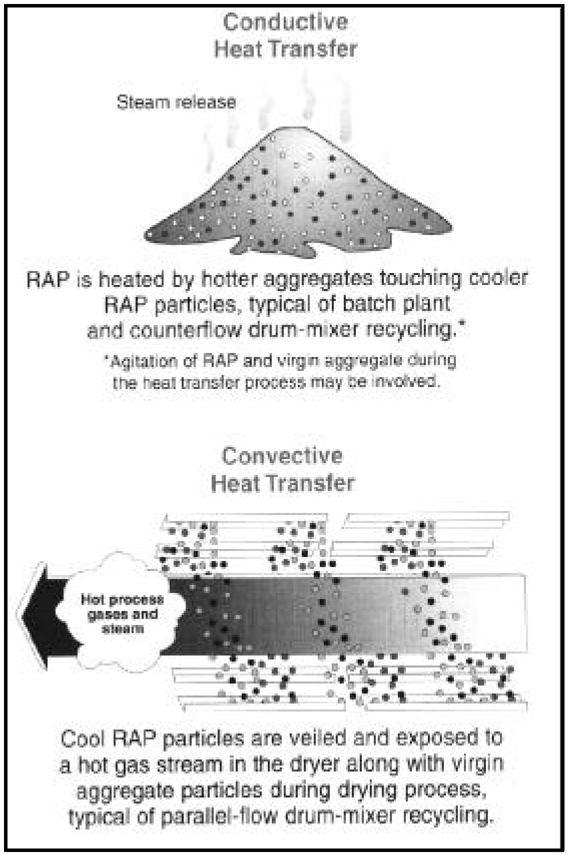


Figure 3-9.8. Conductive versus convective heat transfer.[6]

When RAP is used in batch plant types of operations and in most counter-flow drum mixers, conductive heat transfer is responsible for the majority of the heat transfer between the new aggregate and the RAP. Parallel flow drum mixers primarily use convective heat transfer.

A number of heating and mixing plants that use these principles of heat transfer are presently used to hot central plant recycle. A brief summary of this equipment is provided below.[6] Most of the equipment developments have been focused on solving environmental problems (blue smoke) by using larger percentages of RAP, by developing equipment that can recycle relatively large percentages of RAP from a heat transfer point of view and by providing adequate mixing of the RAP, new aggregate and recycling agent.

Batch Plant or Weight Bucket Method

The Minnesota or weight bucket method is a batch plant process in which the RAP (cold and wet) is introduced into a weigh hopper and mixed with the "superheated" new aggregate in the pugmill (figure 3-9.9). Conductive heat transfer takes place between the RAP and the superheated new aggregate.

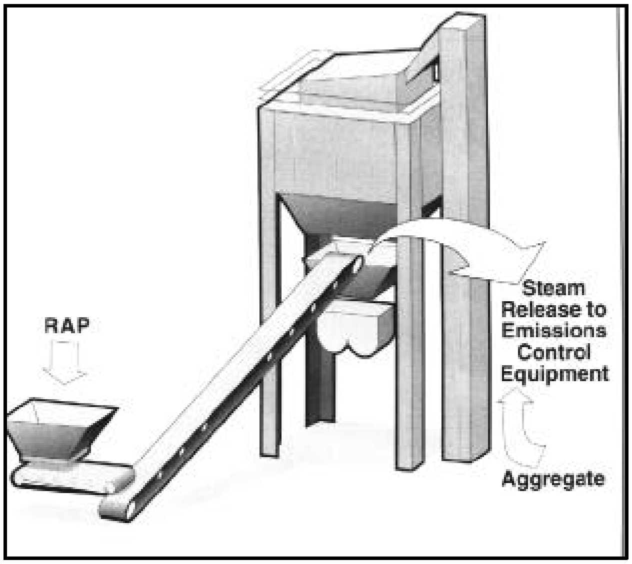


Figure 3-9.9. Weight bucket recycling technique.[6]

Since RAP stockpiles are often relatively high in water content, a significant amount of steam is generated when the RAP comes into contact with the superheated new aggregate. Environmental control systems on the plant must be capable of handling this large amount of steam. Additional air handling equipment may have to be installed to handle the steam problem.

A typical batch plant can recycle up to about 50 percent RAP. Most batch plant operations limit the RAP content to about 25 to 30 percent. Moisture content of the RAP, capability of the bag house to handle high air exhaust temperatures, and the capacity of the emission control system associated with the pugmill are often the controlling factors that limit the quantity of RAP.

Batch Plant-Separate RAP Dryer

This process typically utilizes two conventional parallel flow dryers; one for heating the new aggregate and the second for heating the RAP. The exhaust gas from the RAP dryer is introduced into the burner end of the dryer for the new aggregate to reduce air quality problems and to reduce heating costs of the new aggregate (figure 3-9.10). The heated RAP is stored in a bin and is proportioned into the pugmill mixing chamber.

The capacity of this system to handle RAP is typically controlled by the ability of the new aggregate dryer to handle the steam and “blue smoke” from the RAP dryer. The “blue smoke” is typically burned in the combustion area of the new aggregate dryer.

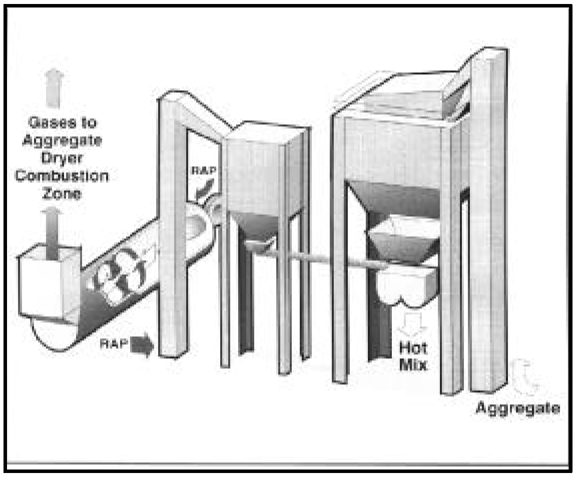


Figure 3-9.10. RAP dryer system.[6]

Parallel Flow Drum Mixer-RAP Collar

Mid length entry systems on conventional parallel flow drum mixers have been used extensively for hot central plant recycling operations (figure 3-9.11). The mid length entry allows for the RAP to enter the drum when the hot gases from the burner have been reduced in temperature. Depending on the characteristics of the asphalt binder on the RAP, the type of recycling agent used and the temperatures of the gases in the drum, air quality problems may arise. Some plants can operate with RAP percentages to 70 percent without significant air quality problems. Percentages as low as 25 percent may be required with other plant-RAP combinations.

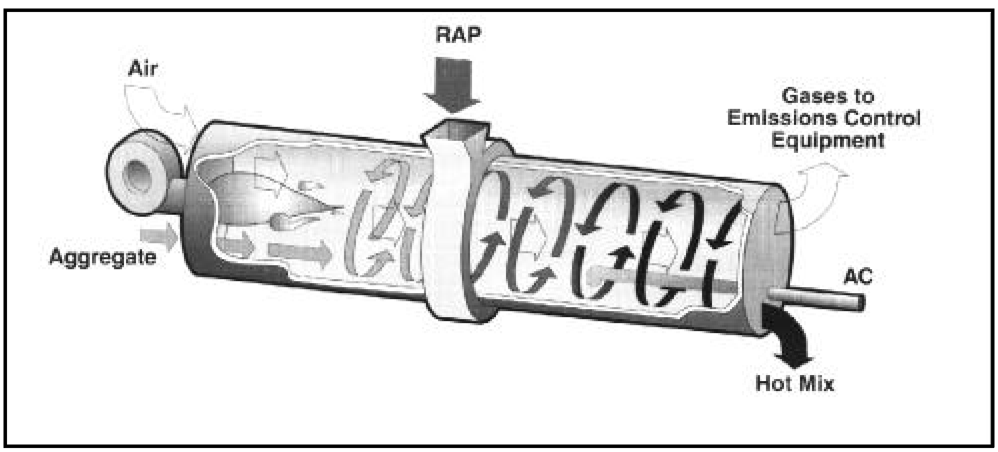


Figure 3-9.11. RAP in a parallel-flow drum mixer.[6]

The parallel flow drum mixer is presently the most common type of plant utilized to produce hot central plant recycled material. Because of air quality problems, coating problems and the desire to increase production, a number of other approaches have been utilized and are summarized below.

The following is a list of approaches:

- Parallel flow drum mixer-RAP collar and continuous mixing device.
- Parallel flow drum mixer-RAP collar and isolated mixing area.
- Parallel flow drum mixer-counter flow RAP drying tube.
- Parallel flow drum mixer-RAP introduced in continuous mixing device.
- Counter flow dryer-RAP introduced in continuous mixing device.
- Counter flow dryer-RAP introduced in the aggregate dryer.
- Counter flow drum mixer.
- Counter flow dryer and continuous mixer.
- Indirect heat transfer.
- Microwave heat transfer methods.
- Laydown and compaction.

5. RECYCLED MIXTURE DESIGN

Several methods are available for designing hot central plant, hot in-place and cold in-place recycled mixtures.[1,2] The procedure published as appendix A in reference 2 is used as the basis for the method described below.

The recycling of old bituminous-bound pavements often requires special consideration because the binder is often hard and brittle. Asphalt recycling agents can be used to soften these old binders and produce mixtures with properties similar to those of conventional asphalt-bound materials. The method outlined in the following allows the engineer to select the types and amount of bituminous modifiers to produce the desired mixture. The methods is applicable for both hot and cold recycling operations and includes modifiers such as softening agents, rejuvenators, flux oils, and soft asphalt cements. The method consists of the following general steps: 1) evaluation of salvaged materials, 2) determination of the need for additional aggregates, 3) selection of modifier type and amount, 4) preparation and testing of mixtures, and 5) selection of optimum combinations of new aggregates and asphalt modifiers.

The overall philosophy of this approach is to use the recycled materials, new aggregate, and modifier to produce a mixture with properties as nearly like a new hot-mix asphalt as possible. Standard test methods are used where possible.

Field Samples[1]

Representative field samples should be obtained from the pavement to be recycled. A visual evaluation of the pavement should be made together with a review of construction and maintenance records to determine significant differences in the material to be recycled along the pavement section. Roadway sections with significant differences in materials should not be lumped together because uniformity and predictability of results will be impaired. Locations within a project can be determined on a random basis using the procedure outlined in the Asphalt Institute Manual Series (17). At least five or six locations should be used as a minimum, and a total composite sample of about 90 kg is recommended for laboratory evaluation. If desired, core samples may also be obtained and used for comparison of original and recycled properties such as stability and resilient modulus.

Obtaining samples from a processed RAP stockpile is the best method to sample for mixture design. However, on many projects, the plans and specifications must be prepared prior to processing the RAP. Cold milling machines can be used for sampling existing pavements; but, availability of equipment is typically a problem. If core samples are taken, the gradation of the RAP aggregate will not be typical.

Since coring is easier to perform than other types of sampling methods, some States have developed correlation factors between core and processed RAP samples. Table 3-9.1 contains correlation factors suggested by several States.[4]

Table 3-9.1. Core gradation correlation factors.

	ARIZONA		FLORIDA			KANSAS	WYOMING
Sieve Size	Visc. 60°C Less than 5000 Pa s	Visc. 60°C Greater than 5000 Pa s	Coarse Mix	Inter-mediate Mix	Fine Mix		
19 mm (3/4")	1.00	1.00	1.00	1.00	1.00	+3%	1.05
12.5 mm (½")	1.05	1.05	1.03	1.02	1.00	+3%	1.05
09.5 mm (3/8")	1.05	1.05	1.06	1.03	1.00	+3%	1.05
6.3 mm (¼")	1.05	1.05					
4.75 mm (#4)	1.05	1.05	1.16	1.08	1.00	+3%	1.05
2.36 mm (#8)	1.05	1.10				+3%	1.05
2.00 mm (#10)	1.05	1.10	1.24	1.12	1.00		
1.18 mm (#16)	1.05	1.10				+3%	
600 µm (#30)	1.10	1.15				+3%	1.10
425 µm (#40)	1.10	1.15	1.27	1.13	1.00		
300 µm (#50)	1.10	1.15				+2%	
180 µm (#80)			1.49	1.25	1.12		
150 µm (#100)	1.15	1.25				+2%	1.15
75 µm (#200)	1.20	1.35	1.84	1.42	1.21	+2%	1.20

Extract and Recover Asphalt and Aggregate[2]

Extraction and recovery tests should be performed at each sample location. Results of these tests (penetration, viscosity, asphalt content and/or performance graded related tests), together with thickness measurements made from the cores, should help determine the uniformity of the section under consideration for recycling. Sufficient asphalt should be recovered to permit blending with asphalt modifiers for further testing.

Relatively large variability in extraction and recovered asphalt binder properties should be expected due to variability associated with the test method and the RAP. Table 3-9.2 shows typical variability of asphalt binder properties for a project in the State of Washington.[4]

Table 3-9.2. Interstate 90 Renslow to Ryegrass compositional data for RAP and recycled HMA.[4]

	Stockpiles RAP (n = 18)		Production Samples (n = 28)	
Gradation	Average	Range	Average	Range
25 mm	100	100	100	100
16 mm	100	100	100	100
12.5 mm	100	99 - 100	97	95 - 99
9.5 mm	96	95 - 98	84	76 - 89
6.3 mm	84	77 - 88	65	54 - 70
2.0 mm	45	40 - 52	37	33 - 40
425 µm	21	19 - 26	19	17 - 20
180 µm	15	12 - 17	14	13 - 15
75 µm	8.5	6.6 - 10.2	8.4	7.8 - 9.1
Asphalt Cement Content	6.2	5.3 - 7.2	5.6	4.8 - 6.6
Asphalt Cement Properties	* n = 16 ** n = 3		* n = 27 ** n = 3	
Pen 4°C **	7	5 - 8	19	18 - 21
Pen 25°C *	15	10 - 17	48	32 - 60
Viscosity (Pa s), 15°C **	1.2×10^8	$0.95 - 1.6 \times 10^8$	6.2×10^6	$5.3 - 7.2 \times 10^6$
Viscosity (Pa s), 60°C *	3052.1	2156.3 - 5091.0	389.4	246.6 - 740.8
Viscosity (cst), 135°C **	966	828-1155	365	332 - 403

(Sullivan 1996)

Aggregate Properties[3]

Aggregate recovered from the samples in step 2 should be tested for gradation and durability, such as Los Angeles Abrasion and Polish value, if the recycled mixture is to be used as a surface course. These data can be used to establish project uniformity together with the recovered asphalt data obtained in step 2.

New Aggregate[4]

New aggregate may have to be added to the mixture for one or more of the following purposes: 1) satisfy gradation requirements, 2) skid resistance requirements for surface courses, 3) air quality problems associated with hot, central plant recycling, 4) thickness requirements, and 5) improved stability, durability, flexibility, etc. Gradation requirements for recycled mixtures should be those presently required by the specifying agency or those in ASTM D3515.

To provide initial and long lasting skid resistance for the recycled bituminous surface course, it may be necessary to blend coarse nonpolishing aggregate with the recycled pavement. It appears as if 40 percent by volume of the plus 4.75 mm fraction should be nonpolishing to provide the desired skid performance on moderate-to-high traffic volume facilities.

Air quality regulations for hot, central plant operations may necessitate using a minimum of about 60 percent by volume new aggregate. This requirement will be gradually reduced as equipment manufacturers and contractors improve the hot recycling operation.

Replacing the recycled pavement with a thicker section of asphalt-stabilized material may be required from a structural pavement design standpoint. This can be accomplished by blending new aggregate with the recycled material or by the additional layers of new asphalt-stabilized materials. If hot, central plant operations are to be used, it appears practical to blend the new aggregate with the recycled pavement.

Asphalt Demand[5]

The asphalt demand of the proposed recycled material can be estimated from the following:

$$D_T = V_R D_R + V_N D_N \qquad \text{(A-1)}$$

$$D_R = D_{\mathrm{CKE}} - A_N \qquad \text{(A-2)}$$

in which:

D_R = Asphalt demand for salvaged or recycled aggregate, percent.
D_{CKE} = CKE derived oil ratios for salvaged or recycled aggregate, percent.
A_R = Asphalt content of salvaged or recycled aggregate.
D_N = CKE derived oil ratios for new aggregate, percent.
V_R = Volume of recycled aggregate in mixtures.
V_N = Volume of new aggregate in mixtures.

Note that if new aggregate is not used, equation A-1 becomes equation A-2.

The asphalt demand determined in this manner should be considered an estimate and can be used as a starting point for mixture design purposes. It should be noted that the asphalt demand will be satisfied by the recycling agent as specified in table 3-9.3 for hot recycling operations and table 3-9.4 for cold recycling operations. These modifiers can be softening agents, asphalt cements, or blends of the two.

Asphalt Properties[6]

Asphalt recovered from the samples in step 2 should be tested for penetration at 25°F and viscosity at 60°F. Asphalt content, penetration, and viscosity should be determined on all extracted samples. These data can be used to determine project uniformity.

Determine Type and Amount of Modifiers[7,8,9]

The type and amount of modifiers can be selected by using figure 3-9.12 or figure 3-9.13 and table 3-9.3 or table 3-9.4, together with a definition of the penetration or preferable viscosity of the binder in the processed recycled mixture and a knowledge of the asphalt demand of the recycled mixture which was obtained in step 5, equation A-1. For example, assume the following:

- Centrifuge Kerosene Equivalent (CKE) oil ratios on extracted salvaged or recycled aggregate, D_{CKE} = 5.0 percent.
- Percent asphalt in salvaged or recycled material, A_R = 4.0 percent.
- Viscosity of aged asphalt = 20,000 poises.
- Additional new aggregate, V_N = 30 percent.
- CKE oil ratio of new aggregate, D_N = 6.0 percent.
- Desired viscosity of recycled asphalt = 2,000 poises.

From equation A-1 and equation A-2, the following asphalt demand can be calculated:

$D_T = V_R D_R + V_N D_N$
$D_R = D_{\mathrm{CKE}} - A_R$
$D_R = 5.0 - 4.0 = 1.0$
$D_T = (0.70)\,(1.0) + (0.30)\,(6.0)$
$D_T = 2.5$ percent

The maximum percent modifier by weight of total binder in the recycled mixture is therefore:

$$= \frac{D_T}{V_R A_R + D_T} \text{ X } 100$$

$$= \frac{2.5}{(0.70)\,(4.0) + 2.5} \text{ X } 100$$

$$= 47 \text{ percent}$$

Using figure 3-9.12, the viscosity of the modifier can be approximated. Enter the figure with the volume percent of lower viscosity modifier (47 percent) and the desired viscosity of the recycled binder to locate point A. Connect point A with the viscosity of the recovered salvaged binder and the line projected to obtain the viscosity of the modifier. Modifier grade RA 5 would likely be suitable according to table 3-9.3.

Table 3-9.3. Physical properties of hot-mix recycling agents.

	ASTM	RA 1		RA 5		RA 25		RA 75		RA 250		RA 500	
Test	Test Method	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Viscosity *140°F, cSt	D 2170 or D 2171	50	175	176	900	901	4500	4501	12500	12501	37500	37501	60000
Flash Point, COC, F	D 92	425	...	425	...	425	...	425	...	425	...	425	...
Saturates, wt, %	D2007	...	30	...	30	...	30	...	30	...	30	...	30
Tests on Residue from RTFO or TFO oven 325°F	D2872 or D 1754												
Viscosity Ratio[A]	“	...	3	...	3	...	3	...	3	...	3	...	3
Wt Change, ±, %	“	...	4	...	4	...	3	...	3	...	3	...	3
Specific Gravity	D 70 or D 1298	Report		Report		Report		Report		Report		Report	

[A] Viscosity Ratio = $\dfrac{\text{Viscosity of Residue from RTFO or TFO Oven Test} \bullet 140^\circ\text{F, cSt}}{\text{Original Viscosity} \bullet 140^\circ\text{F, cSt}}$

Table 3-9.4. Specifications for emulsified recycling agents.

Tests	Test Method	ER-1		ER-2		ER-3	
		Min	Max	Min	Max	Min	Max
On emulsion							
Viscosity, 50°C, SSF	D 244		100	20	450	20	450
Sieve, %	D 244		0.1		0.1		0.1
Storage stability, 24 h, %	D 244		1.5		1.5		1.5
Residue, by distillation, %	D 244	65		65		65	
Dilution			report[A]				
Specific gravity	D 70		report		report		report
Compatibility[B]	varies		report		report		report
On residue from distillation							
Viscosity, 60°C, cSt	D 2170	50	200				
Saturates, %	D 2007		30		30		30
Solubility in Trichloroethylene	D 2042	97.5		97.5		97.5	
On residue from distillation after RTFO[C]							
Penetration, 4°C, 50 g, 5 s	D 5			75	200	5	75
RTFO, weight change, %	D 2872		4		4		4

[A] ER-1 shall be certified for dilution with potable water.
[B] This specification allows a variety of emulsions, including high-float and cationic emulsions. The engineer should take the steps necessary to keep incompatible materials from co-mingling in tanks or other vessels. It would be prudent to have the chemical nature (float test for high float emulsions, particle charge test for cationic emulsions, or other tests as necessary) certified by the supplier.
[C] RTFO shall be the standard. When approved by the engineer, the Thin Film Oven Test (Test Method D 1754) may be substituted for compliance testing.

Note that new asphalt cement and a softer modifier could be used to form the new binder, provided air quality requirements can be met.

Modifier Tests[9]

Samples of modifiers to be used on the job should be obtained and subjected to tests to establish their conformance to specifications (table 3-9.2 or table 3-9.3), as well as to establish the viscosity of the modifier in order to obtain a more realistic modifier content (figure 3-9.12 or figure 3-9.13).

Blend Modifier With Recovered Asphalt[10]

The modifier that may consist of an asphalt cement and softener should be blended with the recovered asphalt and subjected to viscosity and penetration tests to determine if the predicted viscosity (penetration) of the blend was accurate. It is suggested that two blends, one 5 percent above and one 5 percent below the percent recycling agent determined in step 7 and step 8, be prepared, using about 75 to 100 grams of recovered asphalt for each blend. A third blend may be required to confirm the desired viscosity or penetration.

Some recycling modifiers may not be compatible with the salvaged asphalt. Therefore, a thin-film oven test should be performed on the selected recovered salvaged asphalt-modifier blend. A ratio of the aged viscosity to original viscosity of less than 3 will indicate that the recycling agent is likely compatible with the recovered salvaged asphalt.

Preliminary Mixtures[11]

Five different mixtures of recycled aggregate, new aggregate if desired, and modifier should be fabricated. Three samples of each mixture should be fabricated and subjected to stability testing and tests to determine the air void content. These preliminary tests should vary the percentage of new asphalt cement and/or the type and amount of modifiers. Having an experienced engineer present during the mixing and modeling operation is helpful because subsequent trial mixtures may depend on the appearance of the first few trial mixtures. Realize that the modifiers often have a delayed softening reaction.

Use standard mixing and molding operations. An oven curing procedure after mixing and prior to compaction such as that used in California (15 hours at 60°C), appears to be desirable.

Detailed Mixture Evaluations[12]

Evaluate in detail the three most promising mixtures evaluated in step 11 for properties that can be used in pavement thickness design and for durability considerations such as water susceptibility use. Use the testing plan shown in figure 3-9.14 as a guide. The amount of testing will depend on the capability of the agency considering the recycling project.

Select Optimum Mixture Design[13]

Base the optimum mixture design on results of step 11 and step 12, and economic and energy considerations.

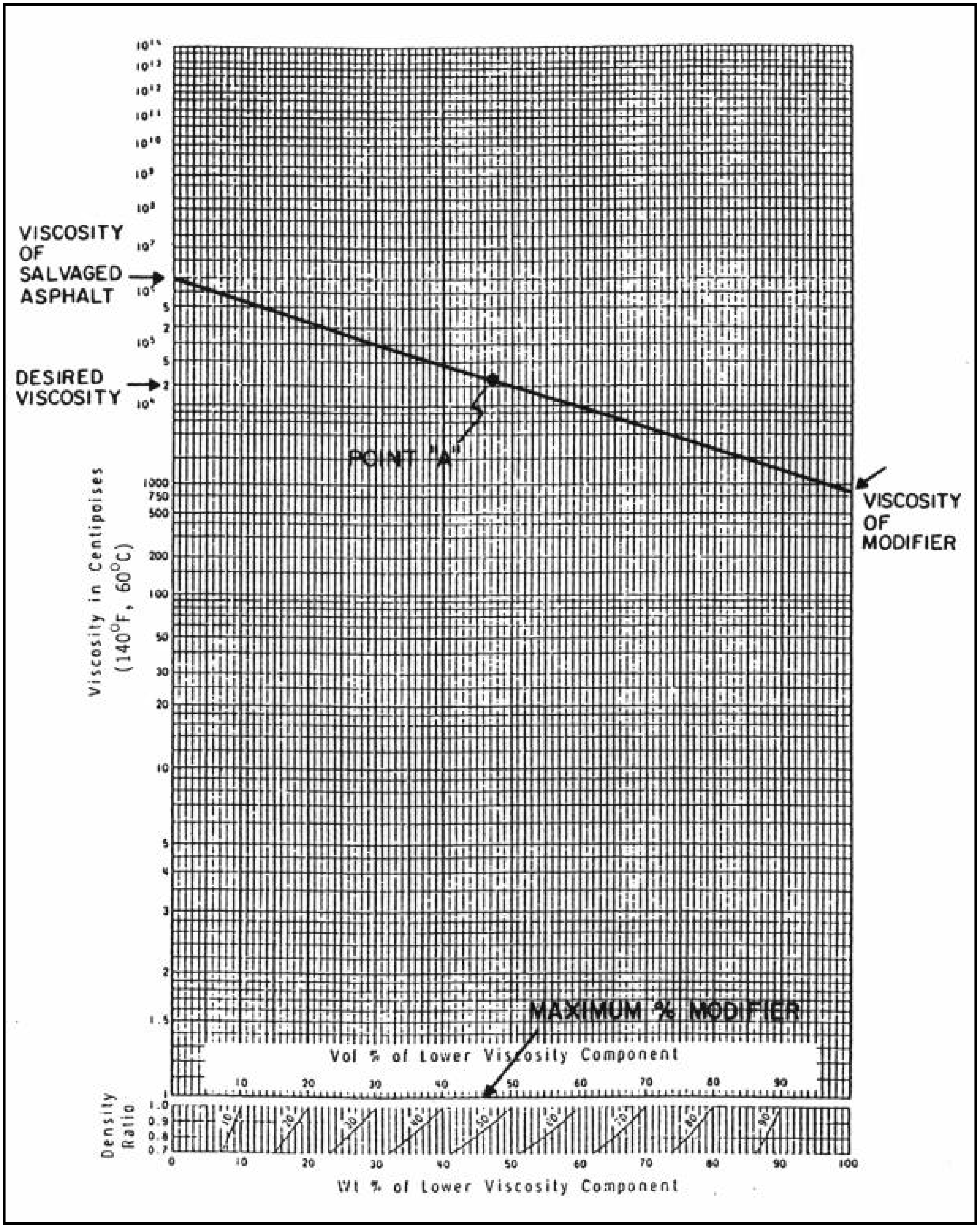


Figure 3-9.12. Viscosity blending chart.

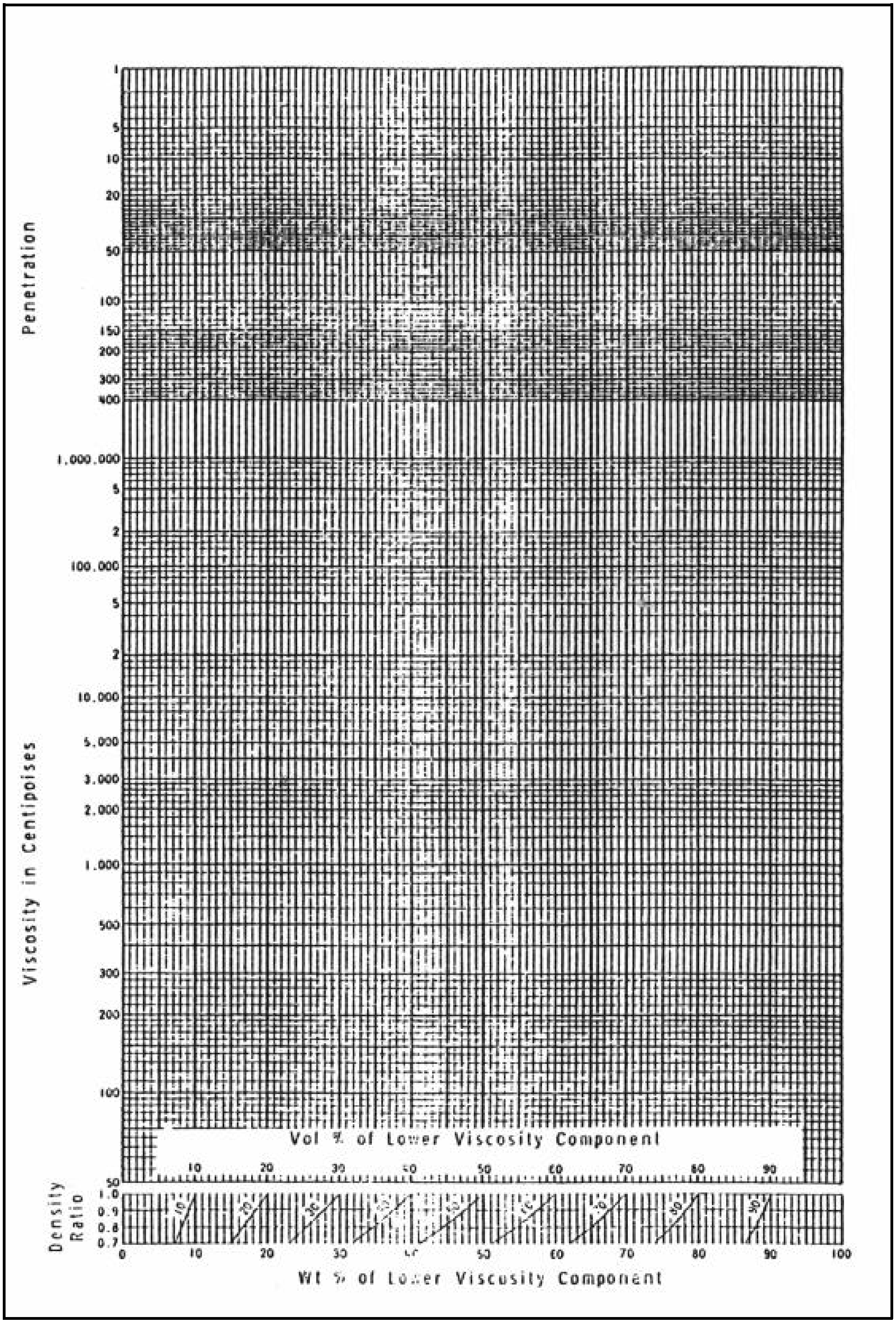


Figure 3-9.13. Viscosity penetration blending chart.

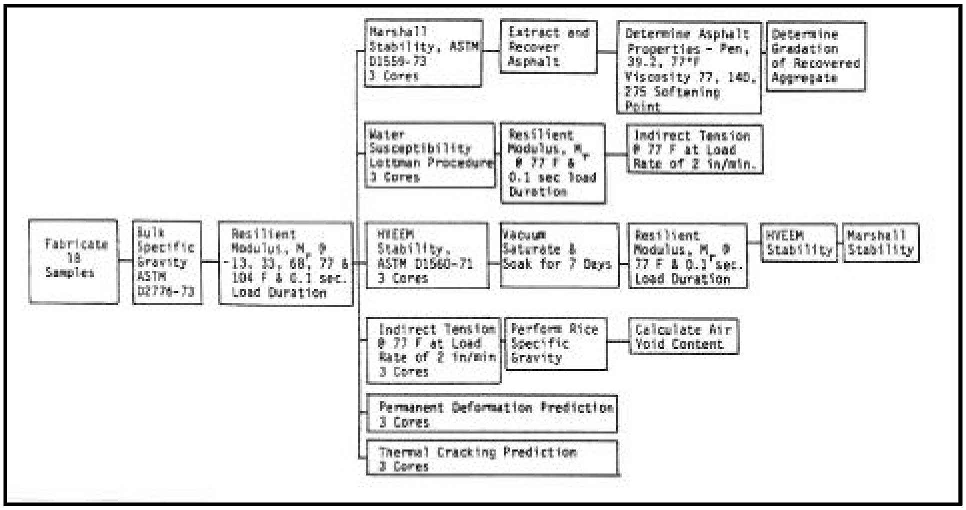


Figure 3-9.14. Test sequence for field cores.

The foregoing discussion is primarily directed toward the use of hot, central plant operations. Recycling in-place with emulsified recycling agents (table 3-9.3) can be accomplished by using the base modifier and the recovered salvaged binder properties.

6. PERFORMANCE

FHWA[4] has recently completed a literature review and visited 17 States to collect performance data on hot central plant recycled projects. Only limited performance information was available, and it is summarized below.

Asphalt Binder Properties

If proper mixture design is performed, asphalt binder properties of recycled mixtures should be typical of those obtained from conventional hot-mix asphalt. A study recently completed for Georgia reports on material properties and performance information.[9] No statistical difference exists between properties of recycled hot-mix asphalt and conventional hot-mix asphalt (table 3-9.6). Since none of the studied pavement sections had significant distress, no difference in performance has been noted between recycled and conventional hot-mix asphalt.[9]

A Washington State study of hot central plant recycled pavements indicated comparable performance between recycled and conventional hot-mix asphalt pavements (table 3-9.7).[4]

A study conducted by Louisiana indicated that there is no significant difference between recycled and conventional hot-mix asphalt when performance is measured by pavement condition rating indicators. A significant difference was not noted on asphalt binder properties (viscosity, penetration, and ductility) between recycled and conventional hot-mix asphalt. Some increase in cracking was noted on the recycled pavements with higher recovered asphalt viscosity properties.[10]

FHWA[4] concludes that hot central plant recycled material that was designed under established mixture design guidelines and produced under appropriate quality control and acceptance measures will perform comparably to conventional hot-mix asphalt. Note that a range of performance has resulted on hot central plant recycled projects. Premature cracking and rutting have been observed on some projects.

7. ECONOMICS

Limited first cost comparison information is available in the literature. Detailed life-cycle cost studies have not been conducted. Typical first cost savings of 5 percent to 20 percent are possible depending upon the following:

- Amount of RAP in mixture.
- Haul distance to central plant.
- Availability of mixing plant.
- Sequencing of construction operations.

The following first cost savings have been reported:

- New York - $7.25 per metric ton
- Wisconsin - $4.10 to $4.50 per metric ton
- Georgia - $1.25 to $6.35 per metric ton
- FHWA - $0.45 to $10.40 per metric ton

If life-cycles and maintenance requirements are nearly equal for both hot central plant and conventional hot-mix asphalt, life-cycle costs will favor hot central plant recycling as compared to conventional hot-mix asphalt.

Table 3-9.5. Comparison of recycled HMA and conventional HMA[25] average test results (standard deviation).[9]

	Construction Details				Field Core Information						
Project	% RAP	Absolute Viscosity (Pa s)	% AC	% Voids (Mat)	Age	% Voids (Mat)	Absolute Viscosity (Pa s)	G*/Sin(Delta) 64°C (kPa) Spec > 2.2kPa	G*/Sin(Delta) 22°C (kPa) Spec < 5000kPa	Indirect Tensile 25°C (kPa)	M_R 25°C (MPa)
18C	0	298.8	6.0	9.0	1.5	7.6 (0.45)	5581.0	20.8	2078	1766 (104)	7505
18R	15	298.8	5.7	9.3	1.5	8.2 (0.81)	5537.0	21.9	2012	1594 (145)	6572
22C	0	270.3	6.0	6.6	1.75	9.4 (0.70)	3309.2	12.1	781	1035 (28)	4942
22R	10	191.2	5.7	6.9	1.75	7.5 (0.89)	3677.3	11.9	655	980 (62)	4210
23C	0	280.7			1.5	3.6 (0.80)	3467.7	12.3	1030	1166 (124)	4816
23R	25	199.0	5.4	6.5	1.5	4.9 (0.52)	3300.2	10.3	721	1111 (117)	4728
25C	0	296.5	5.8	7.9	2.25	6.2 (1.07)	10344.0	28.1	1789	1511 (166)	8289
25R	20	205.5	5.7	7.4	2.25	5.3 (0.70)	5934.1	16.1	1341	1346 (104)	5732
28C	0	304.7	6.0	8.3	1.5	8.3 (1.34)	4627.2	16.0	1102	1497 (69)	7104
28R	20	304.6	5.8	7.8	1.5	6.5 (0.99)	4990.7	16.9	1712	1428 (62)	9519

Table 3-9.6. Performance summary of Washington State DOT recycled HMA recycling projects.[4]

Route	Mileposts	RAP Content	Year Complete	80 kN ESAL (millions)	1993 PSC	Predicted Program Rehab Date	Reason for Rehab	Predicted Project Service Life	Predicted Average Convention HMA Service Life
I-90	121.92 to 126.14	72%	1977	4.3	58	4/93	Rut & PSC	16 years	15 years
I-90	102.61 to 106.34	79%	1978	7.3	86	93	Rut	15 years	15 years
I-90	239.11 to 255.29	65%	1982		80	9/92	Rut	10 years actual perf.	
SR-395	183.69 to 190.61	70%	1982	1.1	73	9/95	PSC	13 years	9 years
SR-2	240.77 to 245.40	40%	1982	0.4	60	8/92	PSC	10 years	9 years
I-90	126.14 to 137.20	75%	1982	3.3	76	9/95	PSC	13 years	13 years
I-90	164.25 to 175.62	75%	1982	2.6	51	2/93	PSC	11 years	12 years
I-90	191.89 to 200.35	65%	1982	2.7	81	3/98	PSC	16 years	13 years
I-90	244.90 to 254.31	71%	1982		80	9/92	Rut	10 years actual perf.	
SR-9	5.35 to 7.15	8%	1982	0.6	80	1/98	PSC	16 years	
SR-97	144.64 to 149.56	9%	1982	0.4	76	8/94	PSC	12 years	9 years
I-90	175.62 to 179.05	35%	1983	2.8	85	5/98	PSC	15 years	13 years
SR-99	22.53 to 25.98	33%	1984	4.3	79	1/98	PSC	15 years	16 years
SR-5	88.02 to 102.70	70%	1984	11.3	97	1/93	Rut	9 years	18 years
SR-527	8.90 to 10.34	35%	1985	0.4	76	1/98	PSC	13 years	21 years

Guidelines for Use

Hot central plant recycling is a very versatile rehabilitation alternative for pavements at all levels of traffic and types of distress. Partial depth and full depth recycling, using the hot central process, can treat all types of distress, provided adequate mixture design and structural design is performed. Full depth recycling removes all of the asphalt and hence can eliminate reflection cracking. Since the pavement can be removed and replaced in this recycling operation, vertical and geometric control problems can be avoided.

Hot central plant recycling has the lowest construction variability of all forms of recycling and can produce the highest quality paving material. Performance information suggests that a properly designed hot central plant recycled mixture will perform the same as conventional hot-mix asphalt.

8. SPECIFICATIONS

A summary of State specifications is presented annually in "Roads and Bridges." The 1996 survey indicates that some States limit the amount of RAP in the various asphalt-bound pavement layers and that some States do not allow RAP in surface courses. Higher RAP percentages are typically allowed in the lower pavement layers. The reason for these limits is based primarily on State perceptions or performance information indicating that poor performance can be obtained at high RAP percentages, that recycling agents do not soften the aged asphalt binder on the RAP, and that air quality problems exist at the higher RAP percentages.

Maximum RAP sizes are typically limited to 50 mm. Many States do not allow the use of soft recycling agent. Soft asphalt cements or typical asphalt cements are used in most States.

Specifications typical of those used for conventional hot-mix asphalt are common.

9. QUALITY CONTROL/QUALITY ASSURANCE

Quality control/quality assurance practices used for hot in-place recycling are those typically used for conventional hot-mix asphalt. Variability of the RAP should be considered when preparing QC/QA limits and pay factors for individual projects. A few States require that the asphalt binder on hot central plant recycled projects conform to center limits after extraction and recovery.

10. SUMMARY

- The amount of RAP recycled in hot central plant operations has not increased significantly in 10 years.
- The use of RAP in hot central plant operations is not widely accepted throughout the United States.
- Most States limit the amount of RAP in hot central plant operations.
- Several States do not allow hot central plant recycled materials as surface courses.
- Performance of hot central plant recycled materials that have been properly designed has been equivalent to conventional hot-mix asphalt.
- First cost savings are possible when using hot central plant recycling.

12. REFERENCES

1. Asphalt Institute, The, "Asphalt Hot-Mix Recycling," Manual Series No. 20 (MS-20), College Park, Maryland, 1981 (Second Edition - 1986).

2. Epps, J.A., D.N. Little, R.J. Holmgreen and R.L. Terrel, "Guidelines for Recycling Pavement Materials," NCHRP Report 224, Transportation Research Board, Washington, DC, 1980.

3. Epps, J.A., "Recycling Materials for Highways," NCHRP Synthesis 54, 1978.

4. Sullivan, John, Review of Hot Central Plant Recycling," FHWA Internal Report, Report TA 92-76, draft copy provided by FHWA, 1996.

5. "Survey of Hot-Mix Production 1985 and 1986," The Futures Group, Inc., prepared for NAPA, Special Report 126, 1988.

6. Young, T., "Recycling Hot-Mix Asphalt Pavements," Information Series 123, National Asphalt Pavement Association, February 1996.

7. "Pavement Recycling Guidelines for Local Governments — Reference Manual", prepared by ARE Inc. For FHWA, Report No. FHWA-TS-87-230, September 1987.

8. National Asphalt Pavement Association (NAPA), "Handling and Processing of Reclaimed Asphalt Pavement (RAP)", Information Series 88, Lanham, Maryland, 1993.

9. Kandahl, P.S., ____, S.S. and Young, B., "Performance of Recycled Mixtures in State of Georgia," Report FHWA-GA-94-9209, January 1994.

10. Paul, H.R., "Evaluation of Recycled Projects for Performance," paper prepared for 1996 Annual Meeting of AAPT.

11. "States Plans of Excess in RAP Specs," Roads and Bridges, October 1996.

MODULE 3-10

HOT-MIX ASPHALT OVERLAYS

1. INSTRUCTIONAL OBJECTIVES

This module is intended to address most of the key issues associated with rehabilitation of an existing flexible pavement by means of a hot-mix asphalt (HMA) overlay. Discussed in this module are the advantages and disadvantages of HMA overlays, their intended functions, and the approaches that have been used to determine the overlay thickness requirement for a given design situation. The importance of preoverlay repair and timely application of overlays are also described. Although the focus of this module is on HMA overlays of existing flexible pavements, it should also be understood that many of the issues discussed here apply to portland cement concrete (PCC) overlays and existing rigid pavements.

Upon completion of this module, the participants should be able to:

1. List the functional and structural deficiencies that can be corrected with a properly designed HMA overlay.
2. Identify the conditions for which an HMA overlay is best suited and most cost-effective.
3. Determine if an overlay is needed for a functional or a structural improvement.
4. Describe the various approaches for HMA overlay thickness design (as well as their strengths and weaknesses).
5. Determine the feasibility and extent of preoverlay repair.
6. Describe the consequences of deferring overlay placement.

2. INTRODUCTION

HMA overlays are, by far, the most popular method of pavement rehabilitation — and rightfully so. They provide a relatively fast, cost-effective means of correcting existing surface deficiencies, restoring user satisfaction and (depending on the thickness) adding structural load-carrying capacity. However, it is not uncommon for these overlays to perform poorly. In fact, there are a number of design and construction factors that need to be considered by the design engineer, if premature failure is to be avoided and maximum performance achieved. As discussed in block 2, many of the important factors are brought into consideration as part of the evaluation of the existing pavement and the assessment of traffic loading. The remaining factors are those considered by the engineer in defining (or designing) the specific features of a given rehabilitation alternative. This module focuses on the latter as it pertains to HMA overlays on existing flexible pavements.

3. DEFINITIONS

<u>Functional Performance</u>. The ability of the pavement to provide a safe, smooth-riding surface to the traveling public.

<u>Structural Performance</u>. The ability of the pavement to carry traffic loads without exhibiting structural distress.

Empirical. Refers to the use of past experience or observation in choosing the best course of action for the future. In the case of overlay design, it refers to the selection of an asphalt mixture and/or overlay thickness based upon experience or what mixtures and/or thicknesses have worked well in the past.

Mechanistic. Refers to the use of engineering mechanics and fundamental engineering properties as part of an analytical or design process. In the case of overlay design, it refers to the determination of an overlay thickness based upon an engineering mechanics-based simulation of the state of stress and strain or deformation that exists in the pavement structure.

4. PURPOSE AND APPLICATIONS

General

The general purpose of an overlay HMA or PCC is to improve the functional or structural performance of an existing pavement or road surface. As previously indicated, overlays can be very effective at addressing problems with existing surface deficiencies and/or increasing the load-carrying capacity of the pavement. HMA and PCC overlays do have some significant differences in terms of initial cost, traffic control requirements, potential life, etc. that affect their applicability or suitability for any given design situation. This module focuses on HMA overlays of existing flexible pavements, while module 4-13 and module 4-14 focus on PCC and HMA overlays of existing rigid pavements.

Specific

HMA overlays have a wide variety of applications ranging from upgrading low-volume roads to improving heavily-trafficked freeways and highways. (They have also been used effectively on airfields experiencing heavy and frequent aircraft loads.) In addition, with proper attention to traffic and environmental effects during structural and mix design, HMA overlays can be applied successfully over a wide range of climate and support conditions. In fact, HMA overlays are the most prescribed feasible alternative by any State or local agency's pavement management system.

The most common HMA overlay is constructed with dense-graded, hot-mixed asphalt concrete, although the type of mixes for binder or surface layers vary from State to State.(2) HMA overlays may be placed on either existing flexible or rigid pavements. Depending upon the problem in the existing pavement, and on the purpose of the overlay, the thickness of HMA overlays may range from 25- to 200-mm or more.

HMA overlays may often be applied in conjunction with cold milling. The use of cold milling is an effective means of restoring cross slope, maintaining curb lines, and preparing the existing pavement to receive the overlay. The removal of a portion of the existing structure must, however, be accounted for in determining the required overlay thickness.

5. LIMITATIONS AND EFFECTIVENESS

General

The fact that overlays have a wide variety of applications does not mean that they have no limitations or that their effectiveness is always assured. Many overlays fail to provide the performance level and useful life expected due to one or more of the following reasons:

1. Improper selection of an overlay as the appropriate rehabilitation method.
2. Selection of the wrong type of overlay.
3. Insufficient overlay thickness design, mixture design, or joint design.
4. Insufficient preoverlay repair of deteriorated areas.
5. Lack of direct consideration of reflection cracking.

It is also important that the need for an overlay be accurately identified. A realistic evaluation of the existing pavement condition is critical in selecting the most appropriate resurfacing alternative, and in determining the type of preoverlay repair required to prepare the existing pavement for an overlay. The design of the overlay is also very important. When used to correct surface deficiencies, minimum required constructible overlay thicknesses are employed to solve the specific problem a pavement is experiencing (e.g., deficient cross slope or inadequate ride quality). A more detailed thickness design procedure is necessary when the purpose of resurfacing is to increase structural capacity.

Specific

For HMA overlays on existing flexible pavements, the limitations and effectiveness are defined mostly by:

1. The distress exhibited by the existing pavements - Pavements that exhibit more fatigue cracking, rutting and/or active transverse cracking will adversely impact the performance of the overlay.

2. The intended design life - Even under the best of conditions, HMA overlays will require some type of rehabilitation again before 15 years.

3. Availability of quality materials - The lack of good sound aggregates and/or select asphalt binders that are suited to the prevailing temperature conditions can limit the performance of an overlay.

Despite their sensitivity to these factors, agencies have been able to maximize the effectiveness of HMA overlays as a rehabilitation option by exercising various types of preoverlay treatments (e.g., geotextiles, heater-scarification, cold-milling, leveling courses, chip seals, crack seals, etc.), repairs (i.e., patching) and through the use of better quality materials and construction practices. Consequently, if sound engineering judgement is exercised during design, it is possible to minimize the limitations of HMA overlays and simultaneously improve their effectiveness.

6. SELECTION OF AN OVERLAY TO CORRECT PAVEMENT DEFICIENCIES

In general, both HMA and PCC overlays are used to correct deficiencies in existing pavements. The choice of the most appropriate alternative should take into consideration the functional and structural adequacy of the existing pavement. Certain performance indicators can be used to determine whether a pavement is structurally or functionally deficient. Examples of functional deficiencies include:

1. Polishing of the surface in the wheelpaths, resulting in decreased surface friction.
2. Roughness resulting from nonload-associated distress, such as joint spalling in a rigid pavement or raveling in a flexible pavement.
3. Inadequate cross slope that results in poor surface drainage.

Many of these deficiencies can be partially or totally corrected with a nonstructural overlay, such as a thin, level-up overlay or a chip seal. Often, this may be performed in combination with cold milling or surface recycling. While this module primarily covers the use of structural overlays, it is important to recognize that nonstructural overlays are often appropriate repair alternatives. A thin overlay can be successfully used to improve a pavement exhibiting the functional deficiencies listed above. However, it should not be used on a pavement that is structurally deficient.

A pavement is structurally deficient if it is experiencing structural distress, when nondestructive testing indicates that a deficiency exists, or when future traffic loadings are expected to exceed design levels. Visible signs of a structural deficiency include:

1. Alligator (fatigue) cracking, permanent deformation, and patches of either distress, in flexible pavements.
2. Deteriorated reflection cracking, particularly transverse cracking over joints and existing cracks, in HMA overlays of rigid pavements.
3. Corner breaks, transverse working cracks, shattered slabs, and patches of either distress in rigid pavements.
4. Punchouts, and patches of punchouts, in continuously reinforced concrete pavements (CRCP).

The presence of these distresses in significant amounts indicates that the structural capacity of the pavement has been approached or exceeded. An overlay that is placed to correct structural deficiencies must address the cause of the structural deterioration of the pavement, and be able to provide sufficient strength to resist further deterioration resulting from future traffic and environmental loadings.

More detailed distress definitions and illustrations useful in determining the condition of a pavement are provided in reference 1 and are presented in block 2 of this course. Where extensive distresses exist, a structural evaluation of the pavement, as described in block 2, should be conducted to determine the extent of the structural deficiency.

7. CONSIDERATIONS IN OVERLAY SELECTION

A thorough examination of pavement deficiencies and the cause of deterioration must be made prior to selecting an overlay as a candidate repair alternative. There is a point where the extent of pavement deterioration could make reconstruction more cost-effective than placing a thick overlay. For example, poor drainage and poor base or subbase support cannot be corrected through the placement of an overlay alone. Also, reconstruction may be the desired alternative when serious concrete durability problems are present in the existing concrete. Figure 3-10.1 shows the conceptual relationship between a pavement's condition and the optimum rehabilitation need.[3] Although this figure was developed for rigid pavements, it is applicable for flexible pavements as well.

Once an overlay has been identified as a candidate alternative, an examination of the overlay types is made to determine which are feasible in a given situation. This determination is based on consideration of construction feasibility, desired performance period, and available funding. These critical aspects are described below and are based on the information found in reference 3.

Construction Feasibility

The first step in determining if a rehabilitation alternative is feasible is to determine if it is constructible. The major factors that determine construction feasibility are described below.

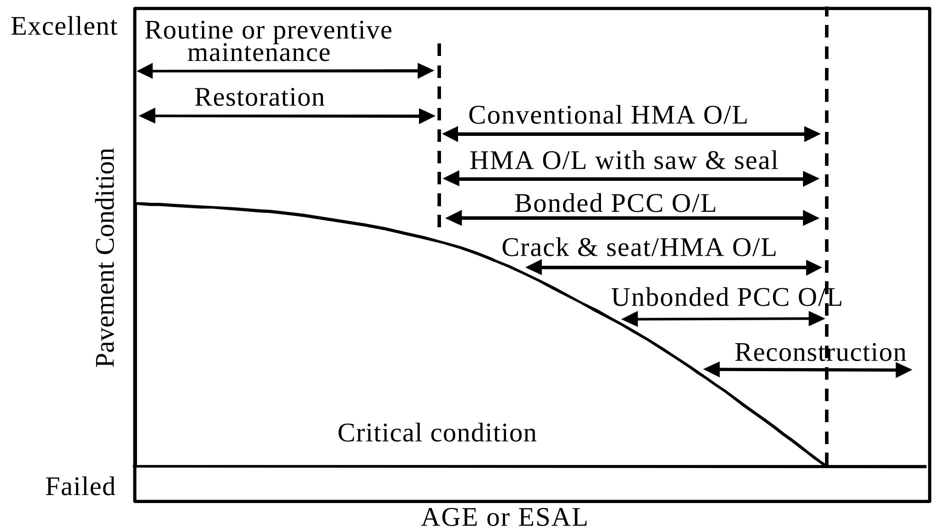


Figure 3-10.1. The spectrum of pavement rehabilitation alternatives.[3]

Traffic Control

HMA overlays often require less traffic control since the overlay can be constructed one lane at a time, and because the project can be opened to traffic much sooner. On roads with high traffic volumes, the ability to do this can translate into considerable cost savings and a substantial reduction in user delay. PCC overlays often do not have this same advantage, although the introduction of fast-track paving, zero-clearance pavers, and unique traffic control plans have greatly increased their construction feasibility.

Constructibility

Lack of equipment, materials, or skilled contractors may limit the feasibility of certain rehabilitation alternatives. In addition, certain construction details play an important part in determining the success of the overlay. Conventional HMA overlays are a common rehabilitation procedure, but there are several issues to consider in their construction. As with new construction, developing the proper mix design and achieving the proper density of the overlay are very important. In HMA overlays with sawed and sealed joints, one of the most critical steps is the location of the underlying joints and the sawing of the overlay in the proper location. For HMA overlays placed on fractured PCC slabs, good performance requires that appropriate steps have been taken to adequately fracture and seat the PCC slab.

Overhead Vertical Clearances and Other Surface Elevation Change Problems

All types of overlays have the disadvantage of decreasing overhead clearances by adding additional thickness to the surface. However, correcting the same deficiency might require different thicknesses for different types of overlays. Where there is the possibility for clearance problems, it must be considered during initial design and steps must be taken to provide adequate clearance. This can be accomplished by using thinner overlays under structures (undesirable), or by reconstructing the pavement directly under the structure with tapers to the overlaid sections on either side. In rare instances, it may be more cost-effective to raise the structures. If many overhead bridges exist along the project, a thick overlay may result in prohibitively high costs.

Overlaying traffic lanes also necessitates overlaying shoulders, which may be in good condition and not in need of an overlay. Overlays may also require the raising of guard rails and the placement of additional fill material adjacent to shoulders. Overlays also decrease curb height and could disturb drainage patterns. One innovative method that has been used in this situation is an inlay, in which the existing pavement is milled or removed and a new HMA or PCC overlay is placed between the shoulders.

Performance Period

The AASHTO Guide defines a rehabilitation's performance period as the amount of time that a rehabilitation will last before the pavement will again require some type of rehabilitation. Because of the difficulty, hazards, and costs involved with closing traffic lanes, some minimum life must be attainable with the proposed rehabilitation. The life of the rehabilitation alternative depends upon many factors:

- Existing pavement type and design.
- Existing pavement condition.
- Structural adequacy to carry future traffic.
- Material deterioration within the pavement structure.
- Climate (temperature, moisture, freeze-thaw cycling).
- Subdrainage adequacy.
- Presence of swelling soils.
- Extent of repair performed.

The extent of preoverlay repair performed may be the single most important factor in determining how well the overlay will perform, and will be discussed in a later section. Two other factors that may influence the performance of the overall are reflection cracking and permanent deformation. These are discussed below.

Reflection Cracking

Reflection cracking is the development of cracks in the overlay caused by horizontal and vertical movement of cracks and joints in the underlying existing pavement. It is a severe problem in both HMA overlays and bonded PCC overlays, causing early deterioration, increased maintenance costs, and decreasing the overlay's useful life.[4,5,6] It is typically not a problem in unbonded PCC overlays. Methods currently used to prevent or control reflection cracking are discussed in detail in module 4-14.

Permanent Deformation

Permanent deformation (rutting) may occur in HMA overlays, sometimes within the first few years of overlay construction. This has become a growing concern in many highway agencies, which report much higher development of rutting than in the past. This recent increase in rutting has been attributed to increases in axle loads, traffic volumes, and tire pressures. Improvements in mix design and construction are necessary to minimize rutting. As discussed in module 3-1, the critical factors to consider to reduce rutting include, among others, the angularity of the coarse and fine aggregate, the rut-resistance of the mix design, the proposed field compaction efforts, asphalt content, gradation, and volumetric properties.

Several States have experimented with stone matrix asphalt (SMA), which is purported to have increased rutting resistance due to a higher percentage of quality coarse aggregate in the mix that ensures stone-on-stone contact. SMA mixtures have a high proportion of asphalt, mineral filler, fine aggregate, and a stabilizing agent to prevent asphalt cement drainage. In addition, SMA has been reported to provide high wear resistance and good durability.

Funding

If the predicted performance period and reliability is acceptable, an estimate of its initial construction cost is made to ensure that the initial cost is within the funds available for the project. If so, then the rehabilitation alternative is feasible. A comparison of life-cycle costs of all feasible alternatives to identify the most cost-effective alternative can then be made.

A brief summary of the feasibility for conventional HMA overlays is given in table 3-10.1.[3] The procedure for considering these factors in a selection process is described in module 3-11. A comprehensive advisory system for the selection of feasible rehabilitation strategies is found in reference 7.

8. PREOVERLAY TREATMENT AND REPAIR

The amount of repair and treatment that is performed to a pavement prior to overlay is probably the single most important factor that affects the future performance of the overlay. The amount and type of preoverlay restoration needed on an existing pavement must be carefully determined by considering the following factors:

- Type of overlay.
- Structural adequacy of the existing pavement.
- Distress type exhibited by the existing pavement.
- Future traffic loadings.
- Physical constraints such as traffic control.
- Overall costs (preoverlay repair and overlay).

Many of the modules covered earlier in this block constitute preoverlay treatments and repairs that can be applied to flexible pavements. Treatments to reduce reflection cracking should also be considered, and are discussed in module 4-14.

Influence of Repairs on Design Overlay Thickness

As previously discussed, the performance of an overlay (of a given thickness) is influenced by the amount of distress exhibited by the existing pavement. Obviously, one way to maximize the performance of an overlay is to exercise a certain amount of repair prior to overlay placement. For the case of HMA overlays, such repairs as full-depth patching of all the areas exhibiting medium to severe fatigue cracking and improving any poor drainage areas are sure to increase the life. This approach could also be viewed as a way of reducing the overlay thickness requirement, i.e., more preoverlay repair means less thickness.

Table 3-10.1 Feasibility guidelines for HMA overlays.[3]

CONSTRUCTIBILITY	
Vertical Clearance	Required thickness may pose a problem.
Traffic Control	Not difficult to construct under traffic. Can be opened to traffic quickly.
Construction	Common rehabilitation procedure. AC mix design and density critical.
PERFORMANCE PERIOD	
Existing Condition	The more deterioration present, the thicker the overlay for a given performance period.
Extent of Repair	Must repair deteriorated cracks and joints and restore load transfer.
Structural Adequacy	Thickness can be increased to provide structural adequacy, but may be substantial.
Future Traffic	High traffic level may result in permanent deformation.
Reliability	Fair (reflection cracking and permanent deformation may be a problem).
COST-EFFECTIVENESS	
Initial Cost	Depends greatly on preoverlay repair.
Life-Cycle Cost	Competitive, if future life is substantial.

A second approach to overlay design is to place an overlay of sufficient thickness to protect the weakened areas in the existing pavement. If it is determined that a stabilized subbase/base layer has deteriorated, instead of removing and replacing the layer, the reduced strength of that layer can be considered in the design of the overlay. The thickness of the overlay is then increased to account for the decreased strength of the deteriorated layer and to protect it from excessive stresses or deflections. With an increase in overlay thickness, applied loads will be distributed over a larger area in the lower pavement layers, decreasing the stresses and deflections imposed on the deteriorated layer. It should be noted that the designed thickness must be developed from a deflection-based or mechanistic overlay design procedure, as discussed later in this module. However, the additional thickness required to adequately protect weak layers will normally be so thick that this approach is not an economically feasible alternative. In addition, there may be other constraints.

A certain amount of preoverlay treatment and repairs should be performed as part of an overlay design. Generally, the second approach to overlay design is not recommended.

Types of Preoverlay Treatments

Localized Repair

Practically all pavements show areas of localized distress where for some reason (e.g., material and subgrade soil variability) structural deterioration originates. In flexible pavements, these areas will often exhibit alligator cracking and permanent deformation, and deflections at these locations will generally be much higher than those in other areas. If these areas are not repaired prior to overlay, thicker overlays will be required. However, since it may not be economical to repair all severity levels of alligator-cracked areas for example, some combination of increased thickness and full-depth patching may be the most cost-effective approach.

A cost analysis can be conducted to determine the optimum combination of repair and overlay that will be most cost-effective for a given project. Figure 3-10.2 illustrates the concept that can be followed, with repairing used as an example. For a pavement in need of localized repair, the cost of repair increases with the percent of area patched. As the distressed areas are patched, the residual strength of the pavement will be increased. Thus, more patching will decrease the required overlay thickness, with an increase in the area (or cost) of patching resulting in a decrease in overlay costs to some minimum level. By combining the cost of patching and overlay and plotting the total costs against the percent of area patched, an optimum total cost and combination of patching and overlay thickness can be determined.

Surface Leveling

Transverse surface irregularities in flexible pavements (permanent deformation, crown problems, and so on) can be corrected in some situations with an overlay. However, it has been found that permanent deformation will often reappear in an overlaid pavement that previously exhibited the distress. This has been attributed to either the difficulty in obtaining adequate compaction of asphalt concrete in rutted areas, or to the existence of low air voids contributing to a loss in stability in the asphalt concrete in the wheelpaths. In either case, the problem can be corrected by milling the surface to remove the irregularities, by removing the unstable asphalt concrete surface layer prior to overlay, or by filling the ruts with a stable leveling course that is properly compacted prior to placement of the overlay. Milling is by far the easiest and most commonly used method. Since permanent deformation can also develop in the base, subbase, and subgrade soil, the pavement must be trenched to determine which layer, or layers, is causing the permanent deformation so that appropriate rehabilitation measures can be applied. If trenching is not possible, coring may be conducted instead.

Long wavelength longitudinal profile irregularities (e.g., settlements and heaves) can be corrected with a leveling course overlay. An average HMA overlay thickness of about 20 mm is used for estimation purposes for a typical pavement to provide the required level up and surface profile.

Controlling Reflection Cracking

Although reflection cracking tends to be a more serious problem on overlays of existing rigid pavements, it can also be a significant problem in HMA overlays on flexible pavements with active transverse cracks. Typical treatments which have varying degrees include the use of:

- Geotextile or fabrics,
- Stress-relieving or stress-absorbing membrane interlayers (SAMIs), and
- “Band-Aid” type crack treatments.

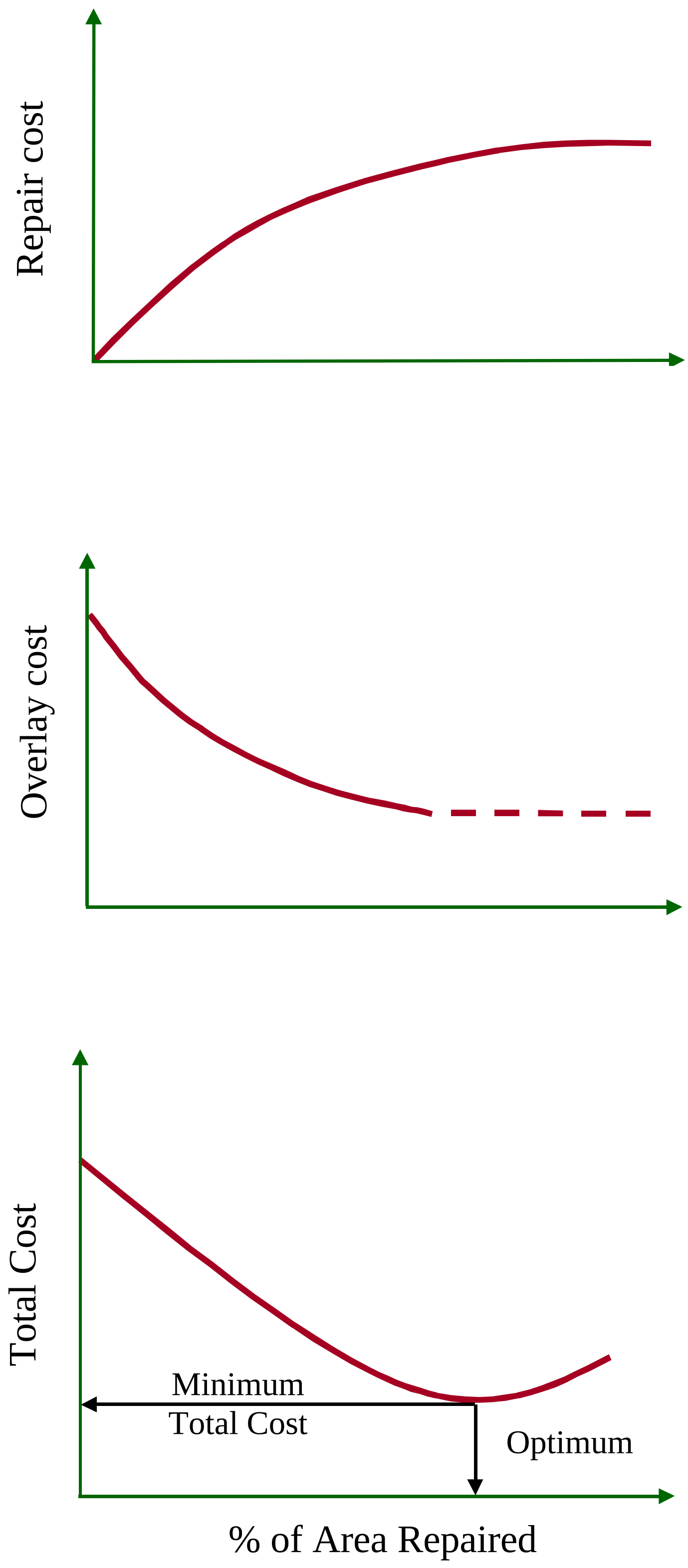


Figure 3-10.2. Patching, overlay, and total cost versus percent of area patched.

Module 4-14 provides more information on the control of reflection cracking.

Drainage Corrections

One factor that should always be addressed in overlay design is drainage, both of the surface and of the subsurface. If poor drainage conditions are contributing to the deterioration of the existing pavement, adding an overlay will not correct the problem. Most overlay design procedures assume that the existing pavement has good drainage. It is always advisable to conduct a thorough drainage survey, identify drainage-related distresses, and develop solutions that address these distresses as part of the overlay design process. Improving poor subdrainage conditions will have a beneficial effect on the performance of the overlay. Removal of excess water from the pavement cross section will reduce erosion and increase the strength of the base and subgrade soil, which in turn will reduce deflections. In addition, stripping in HMA layers may be slowed by improved drainage.

9. OVERLAY DESIGN

This section is only intended to provide some background on the elements of asphalt mixture design and thickness design associated with the design of HMA overlays. The National Highway Institute offers a number of other courses and materials that further elaborate on these key aspects of rehabilitation design. In addition, industry (the Asphalt Institute, the National Asphalt Pavement Association, the National State Association, etc.) and many State highway agencies or organizations (including AASHTO) have documented procedures that are available to the public (see references).

Asphalt Mixture

Module 3-1 provided background information and references on the latest technology for asphalt mixture design. With the emphasis now on laboratory mixture tests that simulate in-situ response/behavior of the pavement, there is an expectation that mixes in the future will perform better than those designed based solely on the current Marshall Stability or Hveem Stability testing. Application of the new technology will, therefore, help improve mix characteristics to better address problems with:

- Fatigue cracking,
- Permanent deformation (rutting),
- Thermal cracking, and
- Moisture susceptibility.

Consideration of these modern approaches to asphalt mixture design are definitely applicable and recommended for HMA overlay mixture design.

Overlay Thickness

There is no universally accepted overlay thickness design procedure. The four major approaches to overlay design are discussed below. Overlay thickness design procedures either assume that adequate preoverlay repair and reflective crack control actions are taken, or they permit different levels of repair to be considered. If preoverlay and reflective crack control treatments are not adequate, then the overlay will likely fail prematurely and not provide the designed structural service. Each agency is encouraged to calibrate or modify the design procedures to suit local conditions.

Engineering Judgment

A number of agencies rely on empiricism or the judgment and experience of their engineers in determining the required overlay thickness. Some agencies have monitored the performance of previous overlays and have an approximate estimate of how selected standard overlays will perform. (This is particularly true for HMA overlays of rigid pavements). Other agencies have set up standards such as 50 mm HMA overlays for certain classes of roads, 75 mm HMA overlays for other classes, and so on. There are obvious deficiencies to this approach, because very few engineers have adequate experience to determine the required overlay thickness for a given traffic and design life. In addition, the advent of vehicles with higher tire pressures, different axle configurations, and higher axle loads, along with new paving materials makes judgment based on past experience questionable. The development of an overlay design procedure that quantitatively considers the important design factors is strongly recommended. Engineering judgment is more acceptable when designing overlays to correct functional deficiencies.

Deflection Approach

The basic concept of the deflection approach is that, in general, the larger the deflection is, the weaker the pavement and subgrade soil must be. Overlays can be used to strengthen the pavement structure to an extent indicated by a certain desired reduction in deflection. Critical deflection levels are identified below which the pavement is expected to perform satisfactorily. Overlay thicknesses are based on reducing the pavement deflection to values less than the critical values. The thicker the overlay, the greater the reduction in deflection, and thus the longer the life of the overlay. The concept of the deflection approach to overlay thickness design is illustrated in figure 3-10.3. The Asphalt Institute, California, Texas, and several other agencies have used the deflection approach in the design of overlays of flexible pavements.

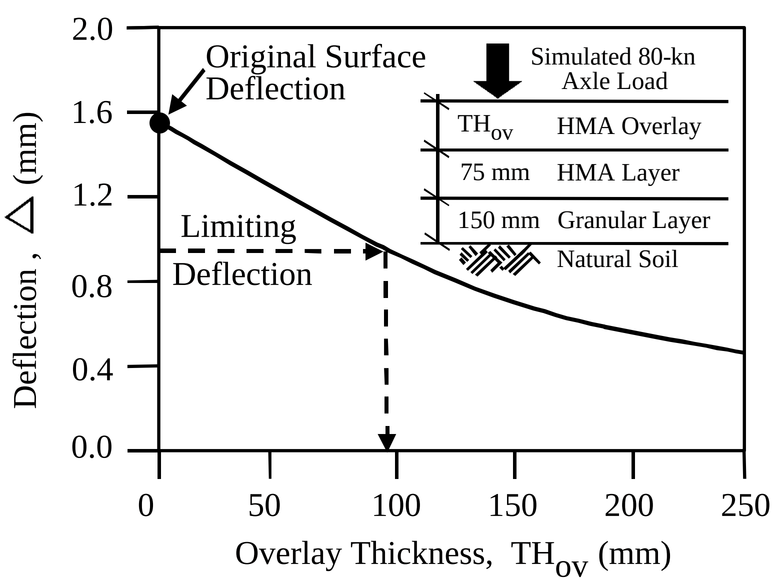


Figure 3-10.3. Illustration of concept for determining design overlay thickness based upon deflection approach.

Structural Deficiency

In the structural deficiency overlay approach, the basic concept is that an overlay is required that is equal to the difference between the structural capacity of a newly-designed pavement and the structural capacity provided by the existing pavement. The concept of this approach is best illustrated with the aid of figure 3-10.4.

The top half of the figure depicts the deterioration of an existing pavement with load applications (N) from some initial serviceability (P1) to an absolute terminal value of 1.5 at which reconstruction would be required. After N_P load applications, however, the pavement reaches some "trigger" level of serviceability (P2) at which a less-expensive overlay may be placed to improve functional characteristics and restore structural capacity. By the time the overlay deteriorates to the P2 level, it will have carried N_f load applications. The magnitude of N_f will depend upon the thickness of the overlay. The bottom half of figure 3-10.4 illustrates graphically what is happening to the pavement structure (in terms of structural capacity) as it is subjected to load applications.

Initially, the existing pavement starts out with a relatively high structural capacity (SC_o). As load applications increase, the structural capacity decreases to a level (SC_{eff}) corresponding to the "trigger" serviceability level (P2) at which the overlay is placed. If the structural capacity required for a new pavement to last the desired N_f load applications is SC_f, then the difference between SC_f and SC_{eff} is the increased structural capacity (SC_{ol}) that must be supplied by the overlay. The value of SC_{ol} is translated into design overlay thickness depending on the procedure. In the current AASHTO Guide (1993), SC is defined as structural number (SN) for flexible pavements and slab thickness (D) for rigid pavements. The Guide provides three procedures for assessing the most uncertain part of the process, the effective structural capacity of the existing pavement (SC_{eff}). The procedures are based on: 1) measurements of surface deflection, 2) observations of pavement condition, and 3) predictions of past load applications. The overall overlay design process in 1993 AASHTO Guide represents the sole improvement[8] over the 1986 version of the Guide.

Mechanistic Approach

The basic concept of the mechanistic approach to overlay design is that the design overlay thickness is one that will limit fatigue damage in the existing pavement and/or overlay to an acceptable level over the design period. The existing pavement and overlay are modeled using elastic layer theory, plate theory, or finite element analysis (as appropriate for the pavement type and overlay type) to estimate the critical fatigue responses associated with the design axle load. For flexible pavements with HMA original surface layers or HMA overlays, the critical response is the maximum tensile strain at the bottom of the original HMA surface layer or HMA overlay. These are depicted in the elastic layer theory pavement model shown in figure 3-10.5.

Once the maximum tensile strains are known, they can be used along with a suitable HMA fatigue equation (such as those depicted in figure 3-10.6) to estimate the fatigue life of the pavement. The simplest case involving the application of this approach is for an HMA overlay on an existing flexible pavement that has no remaining life or, by definition, zero structural capacity. Since, in this case, the original HMA surface layer has no load-carrying capacity, it is simulated as having an elastic (Young's) modulus value on the order of that of an aggregate base material, and the critical strain is determined at the bottom of the HMA overlay. Then, either by trial and error, or through a graphical method (see figure 3-10.7), a design overlay thickness is determined that corresponds to the design future load applications (usually 80-kN equivalent single-axle load applications).

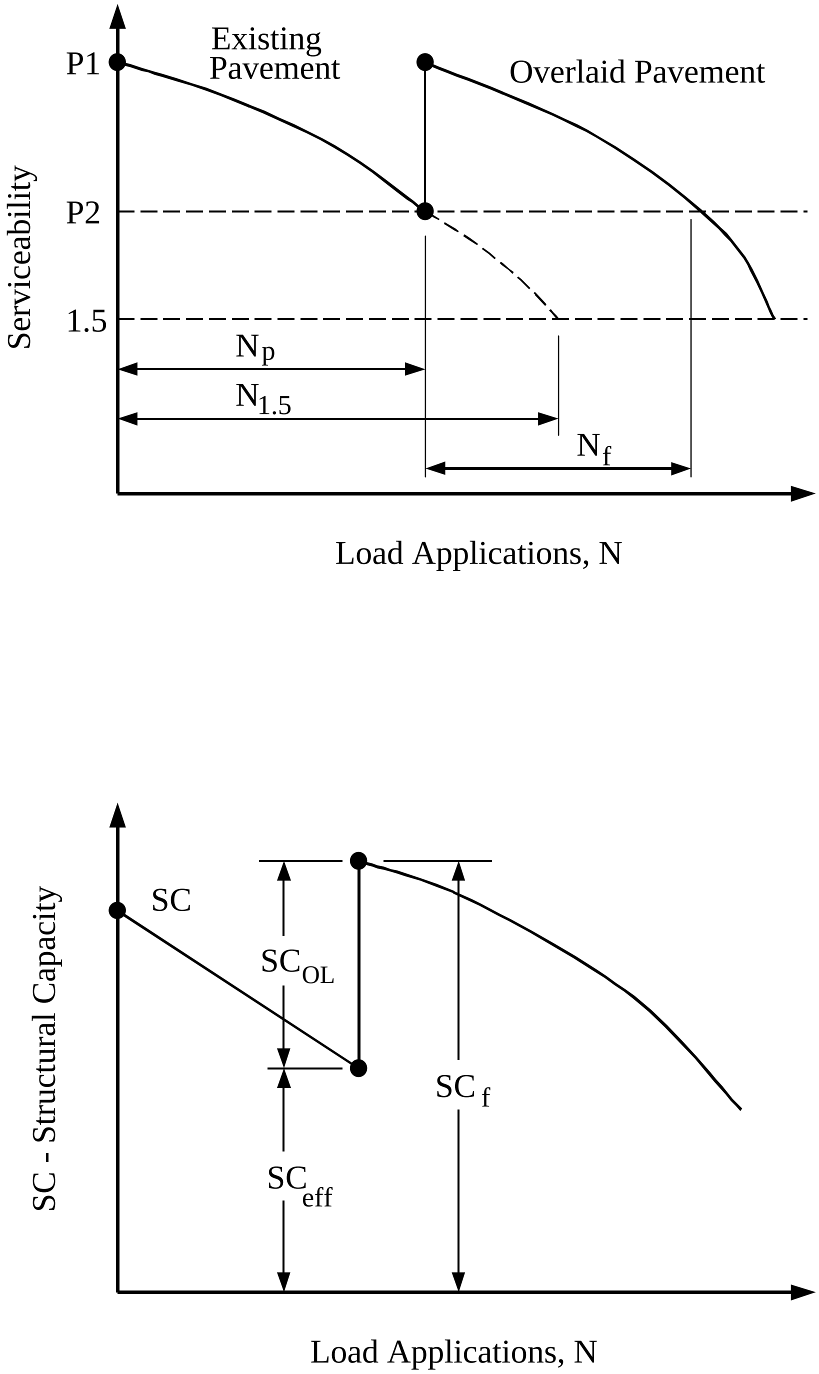


Figure 3-10.4. Illustration of structural capacity loss over time and with traffic.

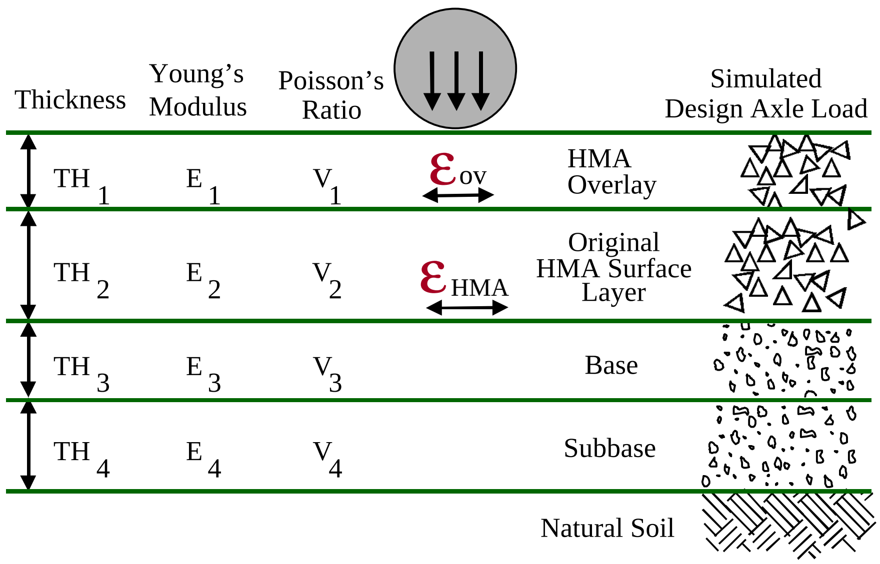


Figure 3-10.5. Illustration of elastic layer theory model used to estimate critical tensile strain in HMA overlay ($\varepsilon_{|OV}$) and/or original HMA surface layer (ε_{HMA})

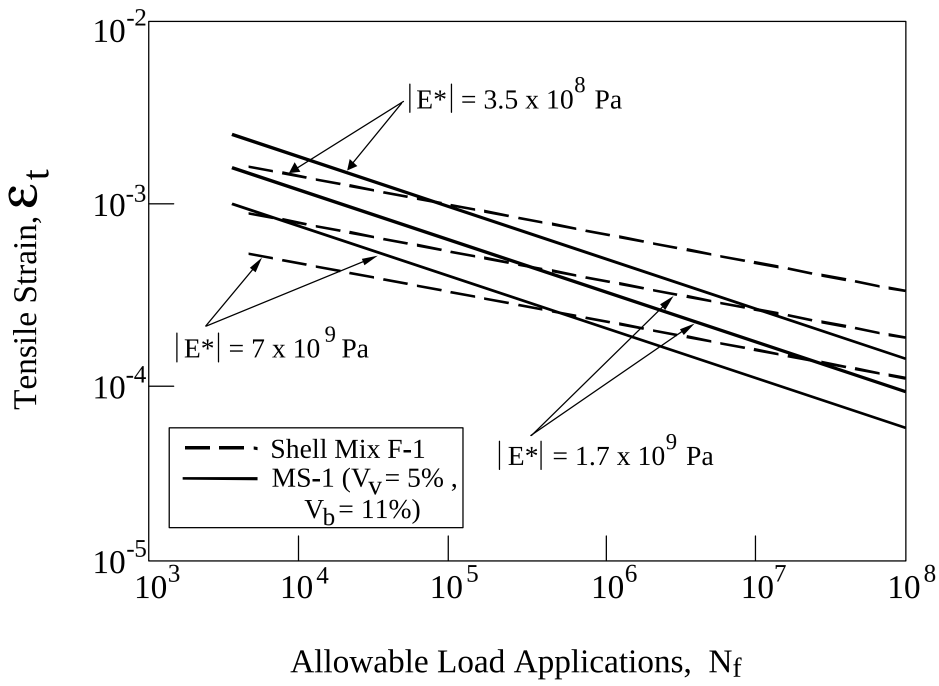


Figure 3-10.6. Typical HMA fatigue relationships from the Shell and Asphalt Institute design procedures.

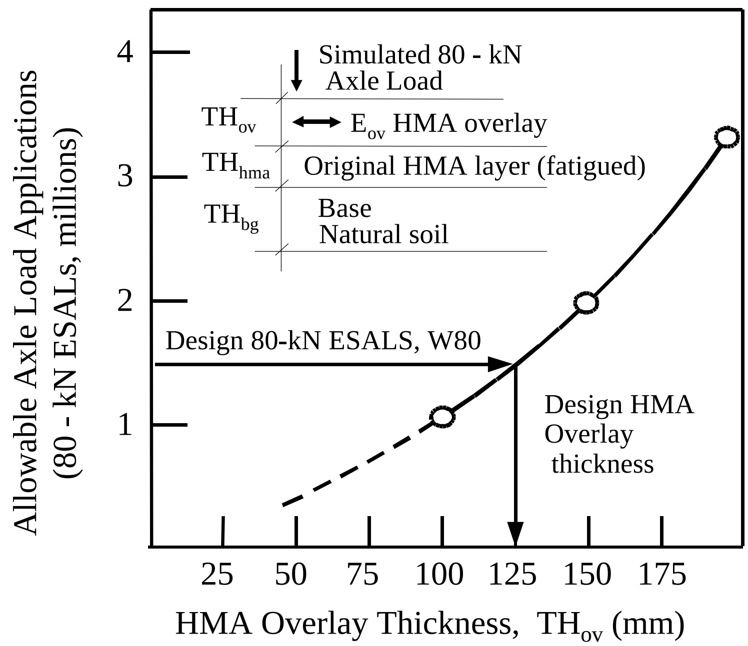


Figure 3-10.7. Graphical method for determining design HMA overlay thickness using the mechanistic (fatigue damage) approach.

For the case where the existing HMA surface layer has some remaining life (i.e., structural capacity greater than zero), the process becomes somewhat more complicated because consideration must be given to tensile strains in both the original HMA layer and the HMA overlay. Figure 3-10.8 illustrates one analytical methodology[9] for taking these into consideration. It involves the use of the Miner's linear damage hypothesis and the associated remaining life concept.

Given that remaining life is the proportion (expressed as a percent) of the original design load applications (or ESALs) the pavement can carry before it reaches zero structural capacity, then it is readily apparent that a graph of remaining life (R_L) versus axle load applications (W_{80}) will be linear, start at 100 percent and deteriorate to zero when the actual applied axle load applications reaches the original design axle load applications. For an existing pavement deteriorating along this line (stage 1 in figure 3-10.8), it becomes feasible (if not essential) to place an HMA overlay at some point, R_{Le}. Beyond this point, the net effect of the HMA overlay is to reduce the maximum tensile strain at the bottom of the original HMA surface layer, thereby changing the slope of the line and extending the life of the original HMA surface layer. During this period (stage 2), the HMA overlay may carry some tensile strains that will cause it to fatigue as well. Thus, by the time the original HMA surface layer loses its structural capacity (R_L = 0 percent), the HMA overlay will have deteriorated to a different remaining life level (R_{Lo}). Beyond that (stage 3), the HMA overlay must carry a higher level of tensile strain and, therefore, will wear out more rapidly. The axle load applications sustained during stages 2 and 3 now represent the allowable axle load applications for a given HMA overlay thickness. So, either by trial and error, or through using a graphical-based method such as that depicted in figure 3-10.7, a design overlay thickness is determined which corresponds the design axle load applications.

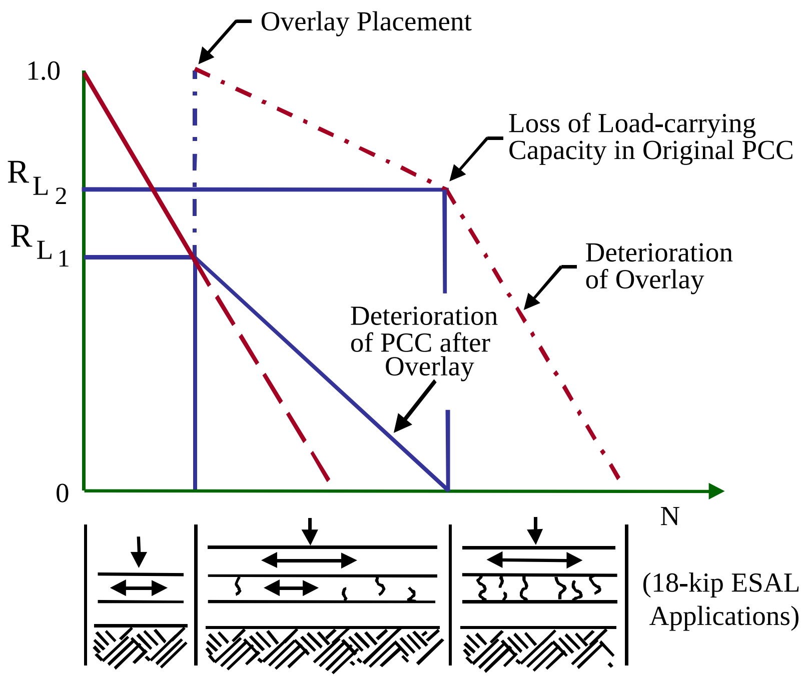


Figure 3-10.8. Graph of remaining life versus axle load applications depicting the approach to calculating the pavement fatigue life after the placement of an HMA overlay.

There is one additional point about the determination of an overlay thickness for a given section of pavement in which deflection measurements are used to characterize the support provided by the existing pavement structure. Actually, this applies equally to the deflection-based and structural deficiency approaches described earlier. If possible, it is preferable to determine an overlay thickness for each deflection data point individually and then average the overlay thicknesses to establish the design thickness than it is to average the deflections and establish a design overlay thickness based upon the average deflection. The reason for this is that the relationship between pavement surface deflection and design overlay thickness is nonlinear; thus, some error can be introduced when design is based on an average deflection.

The mechanistic fatigue damage approach to overlay design has evolved over the years in several research studies. Although these procedures have not been widely validated by field performance data and so far have seen only limited use by State highway agencies, they do provide a more fundamentally sound basis for thickness design than the other approaches. In fact, a project is currently underway (NCHRP 1-37) to implement mechanistic-based design procedures into the AASHTO Guide by 2002. One of the major limitations of the mechanistic approach at this time, however, is that it only considers one mode of failure (fatigue cracking). The design engineer should also consider overlay thickness requirements for other design factors, such as rutting and reflection cracking, especially if these are not addressed as part of the asphalt mixture design or preoverlay treatment plan.

10. SUMMARY

The preceding discussion shows that there are a number of possible overlay solutions for rehabilitating distressed pavements. Although certain overlays are well suited to correct certain problems, they may be less suited to correct others. Therefore, each pavement section must be considered as a separate rehabilitation project, for which the best solution is selected and customized rehabilitation design is performed. The results from a pavement evaluation, following the procedures discussed in block 2, are necessary to help select the best alternative solution.

The determination of whether it will be more cost-effective to conduct extensive repairs on the existing pavement and use a minimum thickness of overlay, or to conduct minimum repairs on the existing pavement and use thicker overlays, requires an economic analysis for the specific project under consideration. Factors such as the rate of deterioration must be considered, and repair amounts and associated costs must be determined for a specific pavement section after a thorough analysis.

For existing pavements with HMA surfaces, consideration should be given to the possibility and advantages of cold milling or recycling in conjunction with the overlay. The consideration of reflection cracking is also vital for all overlay selection and design alternatives. There are several alternatives available for controlling reflection cracking, and these need to be carefully evaluated for each project.

There are four approaches for determining the thickness required for a structural overlay, with the deflection-based and structural deficiency procedures most commonly used and the mechanistic-based procedures on the horizon. The recently revised AASHTO procedures provide field-tested methods for overlay design that are based on deflections, conditions surveys, or remaining life, and should be a useful tool for all types of overlay design. Each agency is advised to calibrate the design procedure to reflect local conditions. It is also suggested that the design engineer utilize more than one procedure to check the design, regardless of the original procedure used.

The results from the rehabilitation alternative selection process described in module 3-11 and module 5-1 should be used to make all final selections.

11. REFERENCES

1. "Distress Identification Manual for the Long-Term Pavement Performance Studies," SHRP-LTPP/FR-90-001, Strategic Highway Research program, National Research Council, 1990.

2. Finn, F.N. and C.L. Monismith, "Asphalt Overlay Design Procedures," NCHRP Synthesis of Highway Practice 116, Transportation Research Board, 1984.

3. Darter, M.I. and K.T. Hall, "Structural Overlay Strategies for Jointed Concrete Pavements, Volume IV—Guidelines for the Selection of Rehabilitation Alternatives," FHWA-RD-89-145, Federal Highway Administration, March 1990.

4. Voigt, G.F., S.H. Carpenter and M.I. Darter, "Rehabilitation of Concrete Pavements, Volume II—Overlay Rehabilitation Techniques," FHWA-RD-88-072, Federal Highway Administration, July 1989.

5. Sherman, G., "Minimizing Reflection Cracking of Pavement Overlays," NCHRP Synthesis of Highway Practice 92, Transportation Research Board, 1982.

6. Voigt, G.F., M.I. Darter and S.H. Carpenter, “Field Performance of Bonded Concrete Overlays,” Transportation Research Record 1110, Transportation Research Board, 1987.

7. Hall, K.T., J.M. Connor, M.I. Darter and S.H. Carpenter, “Rehabilitation of Concrete Pavements, Volume III—Concrete Pavement Evaluation and Rehabilitation System,” FHWA-RD-88-073, Federal Highway Administration, July 1989.

8. Darter, M.I., R.P. Elliott and K.T. Hall, “Revision of AASHTO Pavement Overlay Design Procedures—Final Report,” National Cooperative Highway Research Program, Transportation Research Board, September 1992.

9. Seeds, S.B., W.R. Hudson and B.F. McCullough, “A Design System for Rigid Pavement Rehabilitation,” Research Report 249-2, Center for Transportation Research, University of Texas at Austin, January 1982.

MODULE 3-11

IDENTIFICATION OF FEASIBLE ALTERNATIVES

1. INSTRUCTIONAL OBJECTIVES

This module brings together much of the information covered in this class concerning the basic design processes. The process described consists of the collection of the necessary information on a project, determining the distress mechanisms that apply to the particular project, and with the aid of decision charts, develop the most reasonable solutions (rehabilitation treatments) that would meet the needs of the project. This includes funding, expected traffic loads, resulting structural condition, and longest possible service life given these constraints. The participant shall be able to accomplish the following upon successful completion of this module:

1. Describe what a decision tree or chart is and how it is developed and used.
2. Analyze information from a project and develop a list of specific treatments for that project that will best meet the needs and constraints of that project.
3. Describe the proper uses of decision trees.
4. Describe the limitations and potential problems that are associated with the strict use of a decision tree.

2. INTRODUCTION

This class has now covered a general overview of the various pavement types commonly constructed in North America. The typical distress mechanisms that each pavement experiences, and the common description of the distresses caused by those mechanisms, have also been described. The class has also covered in some detail the type of information that should be collected in site specific project evaluation processes. This information is used to determine what distress and distress mechanisms are present throughout the length of a specific project, and which of these mechanisms, rather than just the distresses themselves must be addressed in the rehabilitation design. The information collected should also provide some indication as to the current level of the distress, and knowing the basic distress mechanisms that are involved, provide some idea as to what rate the distress can be expected to progress. A wide range of rehabilitation treatments has also been described that will correct, to varying degrees most pavement distress found in the different types of flexible pavements. The degree to which a specific treatment corrects, or merely covers the distresses, will determine the service life that particular treatment will give that project.

Many of the available treatments may not fit a specific project because of cost, effects on traffic, construction risk, or unreasonably short service anticipated service. Those who have been in the business of designing pavements for many years can weed through the large array of possible alternatives that do not fit a particular project and glean out the most appropriate treatments to be considered in the rehabilitation design processes. However, for those who are just learning the processes or where there are a large array of possible treatments, the use of decision trees or decision flow charts are a practical aid to sort out the best short list of feasible alternatives that should be considered for any specific project. In this module, the use of decision charts are introduced as a training aid to help lead the engineer through an example problem to determine the distress mechanism, and help develop a list of the best possible treatments that would provide the most cost-effective treatments for the example problem, given the funds available for the project.

Decision trees are developed as an aid to help one walk through the many items that should be considered and helps one account for the specific distress, the basic mechanisms at work within the pavement section, the project specific field conditions and constraints, as well as the available funding established for the project. Quite often the specific project's timing and funding level was established with the aid of a pavement management system, which also uses a decision tree to estimate the possible project timing, scope and, consequently, its allocated funds. The decision tree or chart used to help select the possible pavement rehabilitation treatments for a project level design should be consistent with the same, but more detailed decision trees and trigger levels that were used in the pavement management system, which helped place the project in the construction program.

The decision trees contained in this module are very generic and applicable to the wide range of conditions found throughout North America. Each agency should develop their own decision trees used in both pavement design and pavement management. Clearly, a decision tree that can be used over a large area that includes environmental conditions from Maine to Arizona, to Washington, to Mississippi; and material properties which are even more diverse, has to be very basic. Each agency or group of agencies within a uniform environmental zone should develop their own specific decision trees that account for their environmental conditions, as well as the specific material properties and treatments that are used in their area. The resulting agency decision trees will be more specific and more detailed than those used here, but they will be focused on the distress and distress mechanisms most commonly experienced by that particular agency.

3. DEFINITIONS

Pavement Condition

Pavement condition is usually the single most important piece of information to be considered when assessing a pavement and its potential serviceability and rehabilitation needs. For project planning and design purposes, a pavement is considered to be in good condition when there is little or no surface distress. It would ultimately lead to surface roughness and finally to complete pavement failure, which would result in a cracked and potholed pavement and would be difficult or even unsafe to use. Poor pavement condition is obviously then significant pavement distress that is or approaches an unsuitable surface for automobile or truck travel. For these reasons, pavement condition (i.e., distress) is used by most pavement management systems, and historically formed the basis for simple listings that are and were used to select the projects to be included in each agencies' construction and maintenance program. In the pavement rehabilitation design procedure, pavement condition (distress) is the principle information used to determine distress mechanisms and potential treatments.

A fairly complete description of the more common distresses found in flexible pavement were covered earlier in module 2-2.5.

The following table 3-11.1 includes only a few of the basic flexible pavement distresses that are included in the SHRP "Distress Identification Manual for the Long Term Pavement Performance Project."[1] The distresses are limited to those that are usually used to trigger rehabilitation projects and which are included because they indicate distinctly different distress mechanisms and also because they will be used later in this module as examples for typical decision charts.

Table 3-11.1. Pavement condition definitions.[1]

Distress	General Description	Low Severity Description	Moderate Severity Description	High Severity Description
Fatigue Cracking	"Can be a series of interconnected cracks in early stages of development. Develops into many-sided, sharp-angled pieces, usually less than 0.3 m on the longest side, characteristically with a chicken wire/alligator pattern evident in latter stages."	"An area of cracks with no or only a few connecting cracks; cracks are not spalled or scaled; pumping is not evident."	"An area of inter-connected cracks forming a complete pattern; cracks may be slightly spalled; cracks may be sealed; pumping is not evident."	"An area of moderately or severely spalled interconnected cracks forming a complete pattern; pieces may move when subjected to traffic."
Transverse Cracking	"Cracks that are predominately perpendicular to pavement centerline and are not located over portland cement concrete joints."	An unsealed crack with a mean width ≤ 6 mm.	"Any crack with a mean width > 6 mm and ≤ 19 mm; ..."	"Any crack with a mean width > 19 mm..."
Rutting	"A rut is a longitudinal surface depression in the wheelpath. It may have associated transverse displacement."	Not defined Use 3- to 6-mm	Not defined Use 6- to 12-mm	Not defined Use > 12 mm
Raveling or Weathering	"Wearing away of the pavement surface in high-quality hot-mix asphalt concrete. Caused by the dislodging of aggregate particles and loss of asphalt binder."	"The aggregate of binder has begun to wear away but has not progressed significantly. Some loss of fine aggregate."	"Aggregate and/or binder has worn away and the surface texture is becoming rough and pitted, loss of fine aggregate and some loss of coarse aggregate."	"Aggregate and/or binder has worn away and the surface texture is very rough and pitted; loss of coarse aggregate."

Distress Mechanisms

The basic distress mechanisms for flexible pavement were covered in detail in module 2-2.4.

1. Distresses 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 rehabilitation project, it is generally recommended that a follow-up survey be conducted just prior to advertisement 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. For example, a change in subgrade, traffic, or materials may result in a significant change in pavement performance (and hence

the occurrence of distress). In addition, the inner lanes of multi-lane facilities may exhibit significantly less distress or lower severity levels of distress than the outer lane. By recognizing these trends, rehabilitation designs can be varied along the project and/or across lanes to reduce costs.

3. The distress data provides permanent documentation of the condition of the existing pavement. This lends itself to several uses, including a comparison of pavement performance before and after rehabilitation, and also the development of performance prediction model and curves.

4. The data provides an excellent source of information with which to plan the structural and subgrade testing of the pavement. It is also useful in the evaluation of the pavement's drainability.

5. The distress data provides valuable insight into the mechanisms of pavement deterioration. As a first step, the distress can be identified as being either primarily load-associated or primarily climate / materials-associated. Table 3-11.2 is a general breakdown for the distress types defined in reference 2. If the distress is primarily load-associated, rehabilitation work should include a structural improvement. If climatic conditions or paving materials are contributing to the deterioration, appropriate measures should be identified to address those deficiencies or to lessen their impact or effect on pavement performance. If serious climatic or materials problems exist (e.g., severe D-cracking), the best solution may be complete reconstruction.

6. If time-series distress data is 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 distresses began to appear and their relative rates of progression. Such information can be extremely valuable in identifying causes of distress and in programming appropriate rehabilitation actions (e.g. determining if certain pavement can wait 3 years for an overlay or whether it then will be too deteriorated).

4. USE OF DECISION TREES TO DETERMINE FEASIBLE SOLUTIONS

Decision trees or tables are developed to help the engineer look at a large amount of information in the form of pavement types, distresses, strengths, deflections, etc., and focus on how that information leads to the solutions that fit the various conditions best. The complexity of the decision tree or table depends upon the intended use. They can be a very simple three-stage table like the following table 3-11.3 from the Washington State Pavement Guide that leads you from a very general distress group to what likely distress mechanisms may be considered. The most complex decision trees have been developed for use in advisory systems or "expert" systems.[3,4] An example of a more specific branch in a decision tree is shown in figure 3-11.1 from "Rehabilitation of Concrete Pavements Volume III - Concrete Pavement Evaluation and Rehabilitation System" FHWA-RD-88-073 by Hall et al.

Table 3-11.2. General categorization of pavement distresses by cause.

DISTRESS TYPE	PRIMARILY TRAFFIC/LOAD	PRIMARILY CLIMATE/MATERIALS
HMA-Surfaced Pavements		
Alligator (Fatigue) Cracking	T	
Bleeding		T
Block Cracking		T
Edge Cracking	T	
Lane to Shoulder Dropoff		T
Lane to Shoulder Separation		T
Longitudinal Cracking		T
Patch/Patch Deterioration	T	
Polished Aggregate	T	
Potholes	T	
Raveling and Weathering		T
Reflection Cracking at Joints		T
Rutting	T	
Transverse Cracking		T
Shoving	T	
Water Bleeding and Pumping	T	
Jointed Concrete Surfaces		
Blow-ups		T
Corner Break	T	
D-Cracking		T
Faulting of Transverse Joints and Cracks	T	
Joint Seal Damage of Transverse Joints		T
Lane to Shoulder Dropff		T
Lane to Shoulder Separation		T
Longitudinal Cracking		T
Map Cracking and Scaling		T
Patch/Patch Deterioration	T (M,H)	T (L)
Polished Aggregate	T	

Table 3-11.2. General categorization of pavement distresses by cause (cont'd).

DISTRESS TYPE	PRIMARILY TRAFFIC/LOAD	PRIMARILY CLIMATE/MATERIALS
Jointed Concrete Surfaces		
Popouts		T
Spalling of Longitudinal Joints		T
Spalling of Transverse Joints	T (H)	T (L,M)
Transverse Cracking	T (L,M,H)	T (L)
Water Bleeding and Pumping	T	
Continuously-Reinforced Concrete Surfaces		
Blow-ups		T
Construction Joint Deterioration		T
D-Cracking		T
Lane to Shoulder Dropoff		T
Lane to Shoulder Separation		T
Longitudinal Cracking		T
Map Cracking Scaling		T
Patch/Patch Deterioration	T	T
Polished Aggregate	T	
Popouts	T	
Punchouts	T	
Spalling of Longitudinal Joints	T (H)	T (L,M)
Transverse Cracking	T (M,H)	T (L)
Water Bleeding and Pumping	T	

Here the decision tree is quite specific, leading the engineer from rigid pavement cracks through climate considerations to slab thickness, and traffic levels to specific structural needs and treatments. The decision tree developed for a pavement design advisory system in Washington State[(4)] took an engineer through eight levels of decision branches for each pavement type and predominate distress groups to arrive at a very specific rehabilitation treatment. These latter types of decision trees are usually developed by individual agencies for their own use in pavement design. A similar system with pavement types, distress, road class, traffic, etc., with specific trigger levels to consider different rehabilitation treatments, is built into most pavement management systems (an advanced course in pavement management systems).

Table 3-11.3. Distress groups.[2]

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

Another more diagnostic type of table was developed by Fred Finn and Jon Epps[5] in their Research Report on “Guidelines For Flexible Pavement Failure Investigations” performed for the Texas Transportation Institute. It contained a listing of the distress, possible causes and potential rehabilitation alternatives. A copy of that table is shown as the following table 3-11.4.

A few basic decision tables have been developed as teaching aids to help lead the students through the thought processes as to what treatments may be applicable for certain pavement conditions. The following tables 3-11.5, 3-11.6, 3-11.7, 3-11.8, 3-11.9, and 3-11.10, were developed for use in this class and are not intended to represent accepted practice in any specific area. Any decision trees used by an agency should have been developed by that agency or adopted by that agency from other decision trees that reasonably represent pavement types, conditions, and accepted practice in their location.

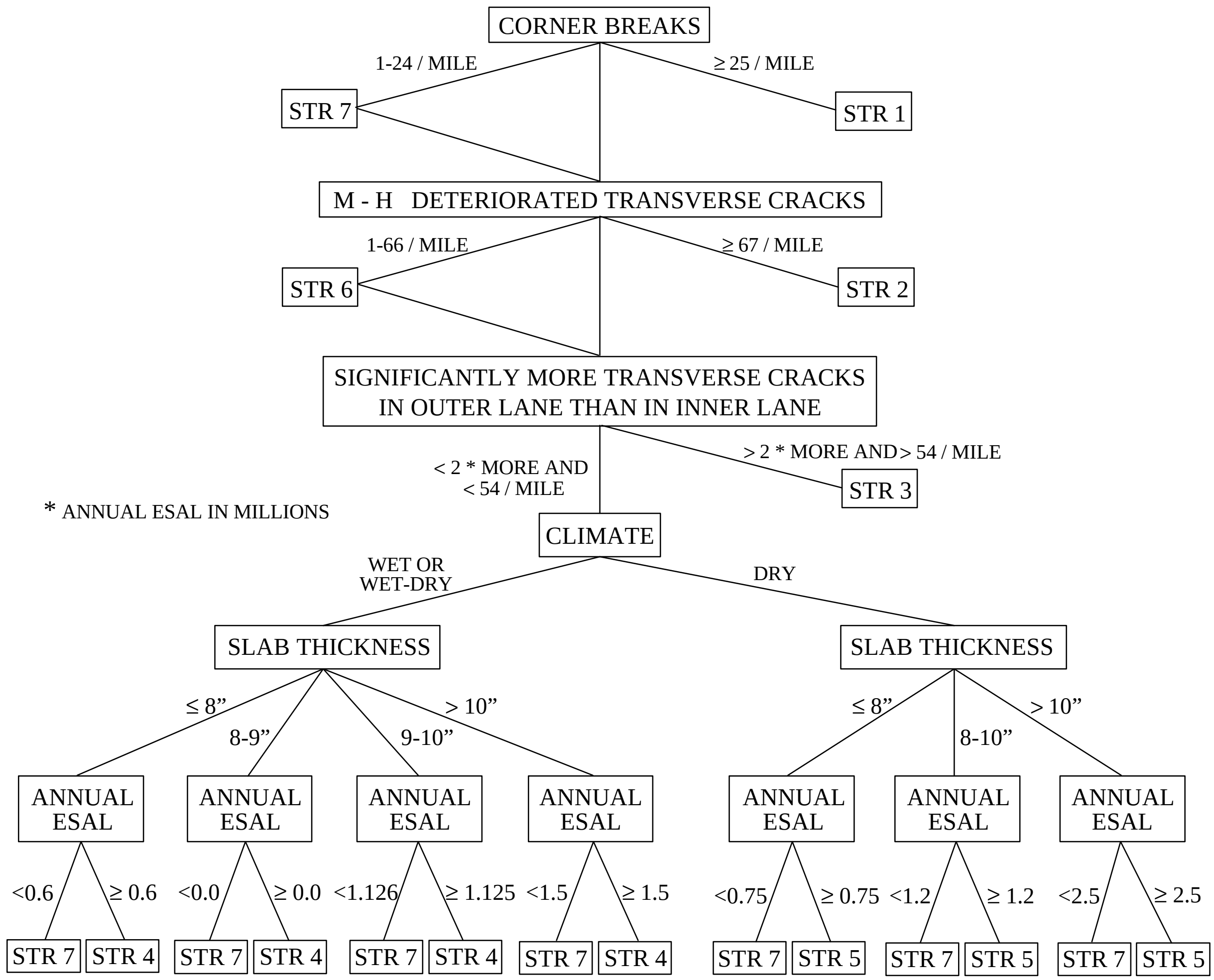


Figure 3-11.1 Decision tree for structural deficient jointed plain concrete pavements (JPCP).[3]

Table 3-11.4. Pavement distress and possible causes and treatments.[5]

Distress	Possible Causes	Rehabilitation Alternatives
Rutting	HMAC mix design, Structural deficiency, Stability of pavement layers Compaction (density) - all layers	Cold milling including profile requirements, With or without overlay Heater scarification or milling with overlay Replacement
Raveling	Low asphalt content Excessive air voids in HMAC Hardening of asphalt Water susceptibility (stripping) Aggregate characteristics Hardness and durability of aggregate	Dilute emulsion or rejuvenating fog seal Seal coat with aggregate Slurry seal Thin HMAC overlay
Flushing	High asphalt content Excessive densification of HMAC by traffic (low air void content) Temperature susceptibility of asphalt (soft asphalt at high temperature) Excess application of fog seal Water susceptibility of underlying asphalt stabilized layers with asphalt migration	Overlay of open-graded friction course Seal coat (well designed and constructed) Cold milling with or without seal coat or thin overlay Heater-scarification/milling with seal coat or thin overlay Heat surface and roll-in coarse aggregate
Alligator Cracking	Structural deficiency Excessive air voids in HMAC asphalt cement properties Stripping of asphalt from aggregate Construction deficiencies	Seal coat Replacement (full-depth patching with HMAC) Overlay of various thicknesses with or without crack treatment Recycle (central plant or in-place Reconstruction
Longitudi-nal Cracking	<u>Load Associated</u> Structural deficiency Excessive air voids in HMAC Asphalt cement properties Stripping of asphalt from aggregate Construction deficiencies <u>Non-load Associated</u> Volume change potential of foundation soil Slope stability of fill materials Settlement of fill or in-place materials Segregation due to laydown machine Poor joint construction Other construction deficiencies	Crack sealing Seal coat (applied to areas of cracking) Replacement (full-depth patching with HMAC) Thin overlay with special treatment to seal cracks and minimize reflection cracking Asphalt-rubber membrane with aggregate seal or thin overlay Heater-scarification/milling with thin overlay
Transverse Cracking	Hardness of asphalt cement Stiffness of HMAC volume changes in base and subbase Unusual soil properties	Crack sealing Seal Coat Overlay with special treatment to seal cracks and minimize reflection cracking Asphalt-rubber membrane with thin overlay Heater-scarification/milling with thin overlay

Table 3-11.4. Pavement distress and possible causes and treatments.[5] (cont'd)

Distress	Possible Causes	Rehabilitation Alternatives
Roughness	Presence of physical distress (cracking, rutting, corrugations, potholes) Volume change in fill/subgrade Non-uniform construction	Overlay Cold milling with or without overlay Heater scarification/milling with overlay Recycle (central plant or in-place)

Example Use of a Decision Table

General Example

For a typical example of the use of a decision table, consider the following conditions:

An existing two-lane HMA pavement surface rural roadway was constructed 12 years ago, consisting of 126 mm (5 in) of HMA over 150 mm of crushed rock and 150 mm of gravel. It has two 3.6 meter lanes with 2.4 meter shoulders.

The pavement is now experiencing medium severity fatigue cracking through about 50 percent of the length of the project with some minor rutting. The first signs of fatigue cracking was evident about 5 years ago. About 10 percent of the roadway is patched. The worst of the cracked pavement has been crack sealed.

Deflection testing was conducted throughout the project using a falling weight deflectometer (FWD). Deflection readings were taken at 0 mm, 200 mm, 300 mm, 450 mm, 600 mm, 900 mm, and 1200 mm from the center of the plate. Though all deflections are used in more rigorous backcalculation programs, only the 0 mm and 900 mm readings will be considered here to estimate subgrade stiffness. Deflections recorded for the 0 mm sensor average about 0.70 mm and ranged from 0.40 mm to 0.84 mm. Deflections recorded for the 900 mm sensor averaged 0.15 and ranged from 0.11- to 0.21-mm. The average daily traffic is 6,000 vehicles per day (vpd) with 20 percent trucks. The project has experienced an estimated 3.5 million ESALs over the last 12 years. It is predicted that the project will experience about 6 million ESALs over the next 10 years. What rehabilitation treatments may be reasonable for these conditions?

Pavement Section

The decision tables start with the consideration of the basic pavement section. In this example, we will choose to use table 3-11.3 < 150 mm (6 in) HMA with fatigue distress. The next consideration is what pavement distress is evident on the project. In the example table, three choices are given: low, medium, and high. Remember that by general consensus, low means that you should consider repairing the pavement, while medium means that you should now repair the pavement and high means that you should have fixed the pavement before this advanced distress. These general definitions also give some indication of the structural condition of the pavement.

Table 3-11.5. 150 mm ACP fatigue cracking decision table.

Distress Type	Fatigue Cracking											
Pavement Thickness	<150 mm ACP											
Distress Severity	Low Longitudinal Cracking in Wheelpaths				Medium Sporadic Alligator Cracking in Wheelpaths				High Heavy Alligator Cracking in Wheelpaths			
Deflection at 9 kips (Do D3)	<30mils < 7mils	<20mils > 7mils	>20mils < 7mils	>20mils > 7mils	<20mils < 7mils	<20mils > 7mils	>20mils < 7mils	>20mils > 7mils	<20mils < 7mils	<20mils > 7mils	>20mils < 7 mils	>20mils > 7 mils
Pavt. Struct. Condition	Good	Good	Poor	Fair	Fair	Fair	Poor	Poor	Poor	Fair	V.Poor	Poor
Subgrade Stiffness	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor
Possible Treatments Ranked in Ascending Cost and Descending Service Life Expected	Surface Treat-ment Crack Seal Do Nothing	Thin Overlay Surface Treat. Crack Seal Do Nothing	Struct. Overlay Double Seal Crack Seal Do Nothing	Nominal Overlay Double Seal Crack Seal Do Nothing	Nominal Overlay Double Seal Surface Treatme nt Crack Seal Do Nothing	Nominal Overlay Double Seal Surface Treat-ment Crack Seal Do Nothing	Recycle or Mill & Str. Overlay Struct. Overlay Double Seal Do Nothing	Recycle or Mill & Str. Overlay Struct. Overlay Double Seal Crack Seal Do Nothing	Recycle or Mill & Str. Overlay Struct. Overlay Double Seal Do Nothing	Mill and Overlay Struct. Overlay Double Seal Do Nothing	Recycle & Thick Str. Overlay Recycle & Str. Overlay Structural Overlay Double Seal Do Nothing	Recycle or Mill & Str. Overlay Thick Str. Overlay Double Seal Do Nothing

Note: Seals and other surface treatments are often considered the preferred rehabilitation treatment on 2 lane roads with < 2,000 ADT.
Seals and other surface treatments should not be routinely considered on 2 lane roads with > 5,000 ADT.
Seals and other surface treatments should not be considered on 2 lane roads with > 10,000 ADT without very careful planning and preparation.
In heavily fatigue cracked ACP in poor structural condition, the double chip seal serves as a crack seal.
Thin overlays are 1 inch or thinner ACP overlays such as thin maintenance hot-mix seals or open-graded friction seals.
Nominal overlays are considered 2 inches thick dense graded hot-mix pavement.
Structural overlays are considered to be thick dense graded hot-mix ACP greater than 2 inches thick, which may be 4 to 6 inches thick.

Table 3-11.6. 200 mm ACP original construction fatigue cracking decision table.

Distress Type	**Fatigue Cracking**											
Pavement Thickness	**> 200 mm ACP Original Construction**											
Distress Severity	Low Longitudinal Cracking in Wheelpaths				Medium Sporadic Alligator Cracking in Wheelpaths				High Heavy Alligator Cracking in Wheelpaths			
Deflection at 9 kips (Do / D3)	<12mils < 7mils	<12mils > 7mils	>12mils < 7mils	>12mils > 7mils	<12mils < 7mils	<12mils > 7mils	>12mils < 7mils	>12mils > 7mils	<12mils < 7mils	<12mils > 7mils	>12mils < 7 mils	>12mils > 7 mils
Pavt. Struct. Condition	Good	Good	Poor	Poor	Fair	Fair	Poor	Poor	Poor	Fair	V.Poor	V. Poor
Subgrade Stiffness	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor
Possible Treatments Ranked in Ascending Cost and Descending Service Life Expected	Surface Treat-ment Crack Seal Do Nothing	Thin Overlay Surface Treat. Crack Seal Do Nothing	Mill & Nom. Overlay Double Seal Crack Seal Do Nothing	Mill & Nom. Overlay Double Seal Crack Seal Do Nothing	Mill & Nom. Overlay Double Seal Surface Treatme nt Crack Seal Do Nothing	Mill & Nom. Overlay Double Seal Surface Treat-ment Crack Seal Do Nothing	Mill & Nom. Overlay Thin Overlay Double Seal Do Nothing	Mill & Nom. Overlay Thin Overlay Double Seal Crack Seal Do Nothing	Mill & Str. Overlay Mill & Nom. Overlay Double Seal Do Nothing	Mill & Nom. Overlay Nominal Overlay Double Seal Do Nothing	Recycle or Mill & Str. Overlay Mill & Nom. Overlay Double Seal Do Nothing	Recycle or Mill & Str. Overlay Mill & Nom. Overlay Double Seal Do Nothing

Note: Seals and other surface treatments are often considered the preferred rehabilitation treatment on 2 lane roads with < 2,000 ADT.
Seals and other surface treatments should not be routinely considered on 2 lane roads with > 5,000 ADT.
Seals and other surface treatments should not be considered on 2 lane roads with > 10,000 ADT without very careful planning and preparation.
In heavily fatigue cracked ACP in poor structural condition, the double chip seal serves as a crack seal.
Thin overlays are 1 inch or thinner ACP overlays such as thin maintenance hot-mix seals or open-graded friction seals.
Nominal overlays are considered 2 inches thick dense graded hot-mix pavement.
Structural overlays are considered to be thick dense graded hot-mix ACP greater than 2 inches thick, which may be 4 to 6 inches thick.

Table 3-11.7. 150 mm ACP temperature cracking decision table.

Distress Type	**Temperature Cracking**											
Pavement Thickness	**< 150 mm ACP**											
Distress Severity	Low >6 mm Wide Crack @ > 15 m Spacing				Medium >6 mm < 19 mm Crack @ 10 m to 15 m Spacing				High >19 mm Crack < 10 m Spacing or > 6 mm Depression			
Deflection at 9 kips (Do D3)	<30mils < 7mils	<30mils < 7mils	>30mils < 7mils	>30mils < 7mils	<30mils < 7mils	<30mils < 7mils	>30mils < 7mils	>30mils < 7mils	<30mils < 7mils	<30mils < 7mils	>30mils < 7 mils	>30mils < 7 mils
Pavt. Struct. Condition	Good	Good	Fair	Fair	Fair	Fair	Poor	Poor	Fair	Fair	Poor	Poor
Subgrade Stiffness	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor
Possible Treatments Ranked in Ascending Cost and Descending Service Life Expected	Rout & Cr. Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Mill & Cr. Seal & OL Cr. Seal & Overlay Do Nothing	Mill & Cr. Seal & OL Cr. Seal & Overlay Do Nothing	Recycle or Mill, Cr. Seal Structural OL Cr. Seal & Str. Overlay Cr. Seal & Overlay Do Nothing	Recycle or Mill, Cr. Seal Structural OL Cr. Seal & Str. Overlay Cr. Seal & Overlay Do Nothing

Table 3-11.8. 200 mm ACP temperature cracking decision table.

Distress Type		Temperature Cracking											
Pavement Thickness		> 200 mm ACP Original Construction											
Distress Severity		Low >6 mm Wide Crack @ > 15 m Spacing				Medium >6 mm < 19 mm Crack @ 10 m to 15 m Spacing				High >19 mm Crack < 10 m Spacing or > 6 mm Depression			
Deflection at 9 kips	Do D3	<10mils < 7mils	<12mils > 7mils	>12mils < 7mils	>12mils > 7mils	<12mils < 7mils	<12mils > 7mils	>12mils < 7mils	>12mils > 7mils	<12mils < 7mils	<12mils > 7mils	>12mils < 7 mils	>12mils > 7 mils
Pavt. Struct. Condition		Good	Good	Fair	Fair	Fair	Fair	Poor	Poor	Fair	Fair	Poor	Poor
Subgrade Stiffness		Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor
Possible Treatments Ranked in Ascending Cost and Descending Service Life Expected		Rout & Cr. Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Rout & Cr. Seal Crack Seal Do Nothing	Mill & Cr. Seal & OL Cr. Seal & Overlay Double Seal Do Nothing	Mill & Cr. Seal & OL Cr. Seal & Overlay Double Seal Do Nothing	Recycle or Mill, Cr. Seal, & OL Cr. Seal & Overlay Double Seal Do Nothing	Recycle or Mill, Cr. Seal, & OL Cr. Seal & Overlay Double Seal Do Nothing

Table 3-11.9. Weather or raveling decision table.

Distress Type		**Weathering or Raveling**											
Pavement Thickness		**Any Thickness**											
Distress Severity		Low Limited Loss of Fine Aggregate in WP				Medium Extensive Loss of Fine Aggregate in WP				High Ext. Loss of Fine and Some Course Aggregate			
Deflection at 9 kips	Do D3	<30mils < 7mils	<30mils < 7mils	>30mils < 7mils	>30mils < 7mils	<30mils < 7mils	<30mils < 7mils	>30mils < 7mils	>30mils < 7mils	<30mils < 7mils	<30mils < 7mils	>30mils < 7 mils	>30mils < 7 mils
Pavt. Struct. Condition		Good	Good	Poor	Poor	Fair	Fair	Poor	Poor	Poor	Fair	V. Poor	V. Poor
Subgrade Stiffness		Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor	Fair	Poor
Possible Treatments Ranked in Ascending Cost and Descending Service Life Expected		Surface Treat-ment Fog Seal Do Nothing	Surface Treat-ment Fog Seal Do Nothing	Single Chip Seal Fog Seal Do Nothing	Single Chip Seal Fog Seal Do Nothing	Single Chip Seal Slurry Seal Fog Seal Do Nothing	Single Chip Seal Slurry Seal Fog Seal Do Nothing	Mill & Nom. Overlay Double Seal Single Chip Seal Slurry Seal Fog Seal Do Nothing	Mill & Nom. Overlay Double Seal Single Chip Seal Slurry Seal Fog Seal Do Nothing	Mill & Str. Over-lay Structural Overlay Double Seal Do Nothing	Mill & Nom. Overlay Nominal Overlay Double Seal Do Nothing	Recycle or Mill & Str. Overlay Mill & Nom. Overlay Double Seal Do Nothing	Recycle or Mill & Str. Over-lay Mill & Nom. Overlay Double Seal Do Nothing

Note: Seals and other surface treatments are often considered the preferred rehabilitation treatment on 2 lane roads with < 2,000 ADT.
Seals and other surface treatments should not be routinely considered on 2 lane roads with > 5,000 ADT.
Seals and other surface treatments should not be considered on 2 lane roads with > 10,000 ADT without very careful planning and preparation.
In heavily fatigue cracked ACP in poor structural condition the double chip seal serves as a crack seal.
Thin overlays are 1 inch or thinner ACP overlays such as thin maintenance hot-mix seals or open-graded friction seals.
Nominal overlays are considered 2 inches thick dense graded hot-mix pavement.
Structural overlays are considered to be thick dense graded hot-mix ACP greater than 2 inches thick, which may be 4 to 6 inches thick.

Table 3-11.10. Deflection data rough interpretation.

Plate Deflection	900 mm Deflection	Comments
< 0.5 mm	< 0.18 mm	Low plate deflection and low 900 mm deflection indicates fair subgrade stiffness, and fair pavement structure.* Very low plate deflections would indicate a stiffer pavement structure.
< 0.5 mm	> 0.18 mm	Low plate deflection with high 900 mm deflection indicates a weak subgrade and, therefore, implies stiff pavement. As above lower plate deflection would indicate stiffer pavement.
> 0.5 mm	< 0.18 mm	High plate deflection with low 900 mm deflection indicates fair subgrade and, therefore, implies a weak pavement structure. Higher plate deflections would indicate very weak pavements.
> 0.5 mm	> 0.18 mm	High plate deflections and high 900 mm deflections indicate weak subgrade and possibly weak pavement structure. Much higher plate deflections would indicate very weak pavement.

* Weak pavement structure refers to the pavement stiffness. Low or weak pavement stiffness, precluding temperature effects, is usually caused by either fatigue cracking or moisture sensitivity or a combination of the two.

Deflection Data

To add to the information about the structural condition the deflection data is also considered. The chart used for this class breaks the deflection data in to two simple considerations: plate deflections higher or lower than 0.50 mm (20 mil), and whether the 900 mm (3-ft) sensor deflections are higher or lower than 0.18 mm (7 mil). Plate deflections by themselves do not provide much information about the structural condition of the pavement, except in very general categories (i.e., very high deflections are bad and very low deflections are good). Sensor deflections at 600- to 1200-mm from the center of the plate provide a good estimate of subgrade stiffness.[6] Sensor deflections of 0.18 mm (7 mil) at 900 mm (3 ft) from the center of the load indicates that the subgrade has a stiffness of around 69 MPa (10,000 PSI). Subgrade stiffness less than 69 MPa is generally considered poor, while greater than 69 MPa is considered fair. From these two deflection readings, one can make the general assumptions shown in table 3-11.10.

Decision Table

The example problem indicated that the average plate deflection was 0.70 mm and the 900 mm deflection averaged 0.15 mm. This indicated that the subgrade stiffness was higher than 69 MPa (10,000 PSI) which, combined with higher plate deflections, indicates a weak pavement section. With fatigue cracking evident through much of the project and the deflection data also indicating a weak pavement structure, one can be reasonably sure that the fatigue cracking extends through the pavement and is probably greater at the bottom of the pavement. From this information, structural reconstruction or resurfacing is obviously called for. Following the same trail through the decision chart shown in table 3-11.3, recycling the fatigued pavement, milling one layer with a structural overlay, or a structural overlay by itself is indicated. Decisions to mill or recycle the existing pavement depend upon pavement height restriction, pavement rutting, etc.

Traffic Consideration

Since the traffic exceeds 5,000 vpd, surface seals or treatments should only be considered as temporary treatments that require additional construction management and control.

5. LIMITATIONS OF DECISION TREES

Decision trees are developed as an aid to help one walk through the information that should be considered. It provides a guide to sort out the specific distress and consider the basic mechanisms at work within the pavement section, while also accounting for the limitations placed on the solution such as traffic geometrics and other construction restrictions.

Decision trees are used in this class to provide some structure to look at the possible alternatives after studying all of the rehabilitation treatments that could be considered. They are used by many agencies in their pavement management systems to limit or constrain the amount of combinations of timing conditions and treatments that the program analyzes to those that are the most logical and fit the agencies' processes. They may also be used by agencies as design aids, as a means to provide uniformity between regional design groups, and to help promulgate policy.

If the decision trees are not continually updated, or if they are rigidly adopted, they may serve more as design constraints than as design aids.

To be of the greatest use and least hindrance to progressive cost-effective treatment, selection must be considered as a treatment selection aid, not mandatory rules. When a new or emerging and cost-effective treatment will satisfy the needs and constraints of a project, it should be considered along with the more standard treatments included in the decision tree. As new or emerging treatments become fairly standardized, they should be incorporated into the decision tree. [In the example, decision charts used in this module, there are several treatments that include milling and or recycling.] Ten years ago, these treatments were considered new or emerging. They are now considered standard practice.

6. REFERENCES

1. "Distress Identification Manual for the Long-Term Pavement Performance Project," SHRP-P 338 Strategic Highway Research Program Washington, DC, 1993.

2. Washington State Pavement Design Guide, Volume 2, Washington State Department of Transportation, Olympia, WA 98501, February 1995.

3. Hall, K., J.M. Connor, M.I. Darter, and S.H. Carpenter, "Rehabilitation of Concrete Pavements Volume III - Concrete Pavement Evaluation and Rehabilitation System," FHWA-RD-88-073, FHWA, McLean, Virginia 22101-2296, July 1989.

4. Ritchie, S.G., Che-I Yeh, J.P. Mahoney, and N.C. Jackson, "Development of an Expert System for Pavement Rehabilitation Decision Making," Transportation Research Record 1070, Washington , DC, 1986.

5. Finn, F.N. and J.A. Epps, "Guidelines for Flexible Pavement Failure Investigations," Research Report 214-16 Texas Transportation Institute, July 1980.

6. "AASHTO Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials, Washington, DC, 1993.

BLOCK 4

RIGID PAVEMENT REHABILITATION TECHNIQUES

This block provides detailed information on the design and construction related issues for the most widely-used rigid pavement rehabilitation techniques. These techniques all fall under the standard 3R definitions of restoration, recycling and resurfacing. They are covered in a logical sequence of minimal to maximum impact on the existing pavement.

In addressing issues related to the most common method of rehabilitation (i.e., resurfacing), it should be understood that the information is more of a general nature and that the emphasis is more on the practices that surround the overlay design and construction operations. NHI offers several other courses that are targeted at addressing specific structural design, mix design and construction related issues.

The block concludes with a module on how to identify the most appropriate (candidate) rehabilitation treatments for a given project.

MODULE 4-1

RIGID PAVEMENT OVERVIEW

1. INSTRUCTIONAL OBJECTIVES

Rigid pavements, those constructed using portland cement concrete (PCC), differ substantially from flexible pavements. In addition to the obvious difference of materials, rigid pavements differ in their design, response to traffic and environmental loadings, overall performance capabilities, and rehabilitation requirements.

This module presents a brief introduction to PCC materials and rigid pavements. This information is by no means comprehensive, but should serve as a useful background to the participant interested in learning more about a commonly used material. Upon successful completion of this module, participants will be able to accomplish the following:

1. Identify the typical layers of a rigid pavement.
2. Understand the response of a rigid pavement to applied loads.
3. Describe the fundamental materials that are used in rigid pavement construction and rehabilitation.
4. List the common rigid pavement types and their unique characteristics.

2. INTRODUCTION

The use of concrete materials dates back thousands of years. Well known examples include gypsum mortars used by the Egyptians in the construction of the Pyramid of Cheops (around 3000 B.C.) and the lime mortar used by the Romans in the construction of the Colosseum in Rome. The Parthenon, built by the Greeks in the second century, made use of hydraulic limes as well.

The durability and long-term performance characteristics of cementitious materials used in structures are well known. The modern origins of the material that is used in concrete pavements today dates back to the early 1800s. In 1824, the first patent on portland cement was issued. The name came from the similarity of the hardened cement to locally used building stone found near Portland, England. The first American patent for the product was issued in 1871, and in 1892 the first concrete road was built in the United States, in Bellefontaine, Ohio. That road recently celebrated its one-hundredth anniversary and is still carrying traffic today.

3. DEFINITIONS

Portland Cement

Portland cement (as suggested previously, was once a brand name but now is used as a generic reference to a category of materials) is made up of lime, iron, silica, and alumina. These materials are broken down, blended in the proper proportions, and then heated in a furnace at a high temperature to form a product called "clinker." The clinker, when cooled and pulverized, is then ready for use as cement. By varying the materials that are used in the production of cement as well as the fineness of the grinding, different cement "types" are created.

Portland Cement Concrete

Portland cement concrete consists of a blend of commonly found materials, including coarse-grained materials (aggregates), fine-grained materials (sand and smaller particles), and cement. After mixing in the presence of water, a hard cement matrix is formed. Additional materials, called admixtures or additives, are added to PCC to enhance its properties. For example, accelerators are a class of admixture that speed up the hardening rate of the concrete mix.

Aggregates

Aggregates include both gravels (naturally occurring) and crushed stone (quarried). PCC is made up of coarse aggregates, those retained by a 4.75-mm sieve, and fine aggregates. Aggregates typically make up between 60 percent and 80 percent by volume of a PCC mix. The type, gradation, shape, and hardness are among the properties of aggregates that affect the quality of the concrete.

Water-Cement Ratio

The water-cement ratio is the ratio of the weight of total water in the PCC mix to the weight of cementitious material in the mix. Cementitious materials include cement and any other pozzolans, such as flyash, that are incorporated into the mix. The water-cement ratio is the most important factor contributing to the strength of the concrete. Typical water-cement ratios for paving concrete are between 0.40 and 0.50.

Admixtures

A number of admixtures are added to plastic (still wet) concrete in order to obtain specific desirable characteristics. These include air entraining agents, water reducing agents, set accelerators, and set retardants. Each of these admixtures alters a specific property of the plastic mix. Some admixtures, such as accelerators, retardants, and water reducing agents, are used to obtain specific results during placement. These materials are added to increase concrete's workability or to improve its handling under otherwise adverse conditions. Other admixtures, such as air entraining agents, are used to enhance concrete's long-term properties. Air entraining admixtures introduce a matrix of air bubbles into concrete so that water trapped in the pavement has room to expand when frozen. Their use is essential to sound concrete constructed in areas subjected to freezing.

Rigid Pavement

A rigid pavement is one that uses a PCC layer to carry traffic loads. The concrete layer may be located at the surface or may be found under one or more hot-mix asphalt (HMA) overlays (sometimes referred to as a composite pavement). Rigid pavements may include one or more base courses beneath the pavement, or may be constructed directly on the subgrade soil. Base courses are made from a range of materials, including both dense-graded and open-graded granular materials, cement- and asphalt-stabilized materials, and various by-products such as slag. The purpose of a base course in a rigid pavement varies. They may be used to:

- Provide subdrainage.
- Reduce the effect of damage due to frost heaving.
- Limit pumping and the associated development of subsurface voids.
- Serve as a construction platform.

Several different types of rigid pavements are constructed. The more common types include jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), and continuously reinforced concrete pavement (CRCP). Characteristics of these different rigid pavement types are described below, with schematic drawings of each shown in figure 4-1.1.

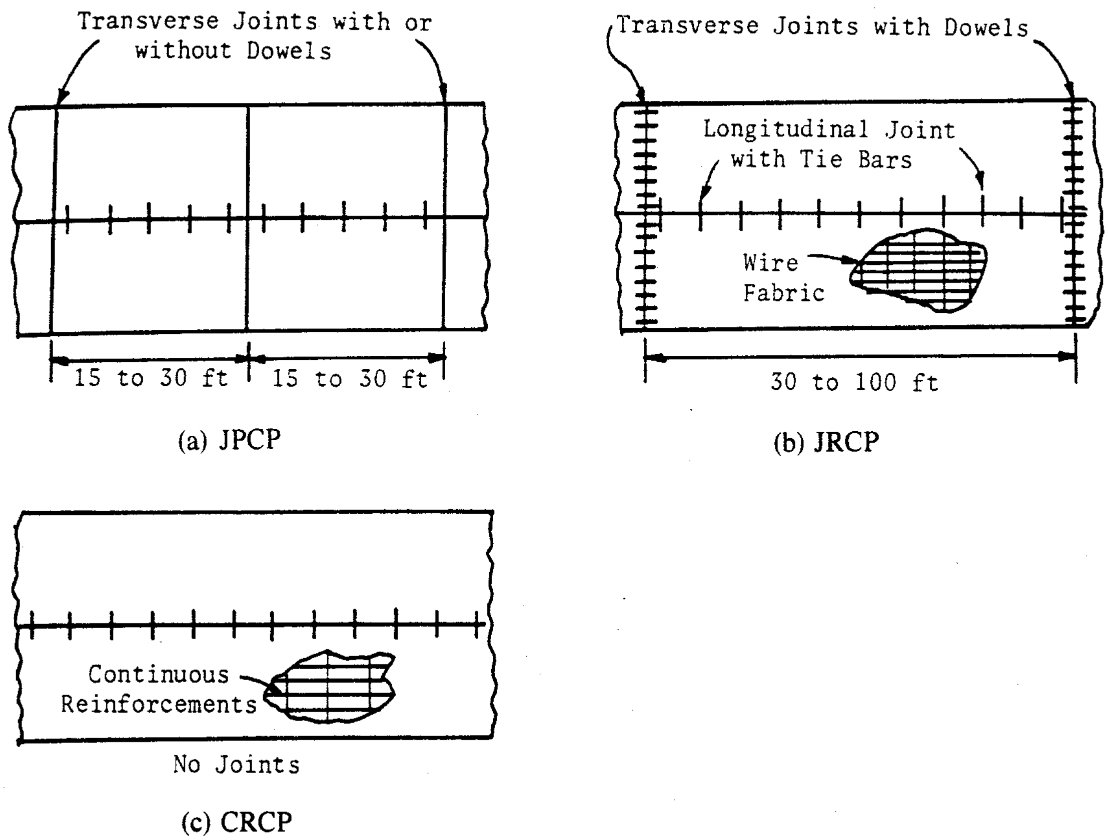


Figure 4-1.1. Common rigid pavement types.[1]

<u>Jointed Plain Concrete Pavement</u>

Jointed plain concrete pavements contain no reinforcement distributed throughout the pavement slabs. However, they do contain tie bars across longitudinal lane-lane joints and may contain dowel bars across transverse joints. Most modern designs do include dowel bars, especially where heavy traffic loadings are expected. This design is characterized by short joint spacing, generally between 4 and 6 m.

<u>Jointed Reinforced Concrete Pavement</u>

Jointed reinforced concrete pavements contain steel reinforcement (either wire or mesh) distributed throughout the interior portion of the slabs. JRCP have longer joint spacing than non-reinforced PCC pavements, typically from about 7- to 30-m. Because of these longer joint spacings, it is expected that transverse cracks will develop but that the steel reinforcement will hold those cracks tightly together; however, the steel is not intended to contribute structurally to the load-carrying capacity of the pavement. Steel contents are typically around 0.2 to 0.3 percent of the cross-sectional area. Dowels should always be used at the transverse joints of JRCP.

Continuously Reinforced Concrete Pavement

Continuously reinforced concrete pavement is distinctive in that its only joints are made at the end of a day's construction or at structures. In the absence of joints, continuous longitudinal reinforcement is used to both induce the formation of closely spaced (1- to 2.5-m) transverse cracks and to keep those cracks together. The longitudinal reinforcing bars are larger than those found in JRCP, and typically constitute 0.6 to 0.8 percent of the cross-sectional area. As with the steel on JRCP designs, the steel in CRCP is not intended to contribute to the load-carrying capacity of the pavement.

Other Rigid Pavements

A number of other types of rigid pavements are occasionally encountered. These include roller-compacted concrete, prestressed (post-tensioned) concrete pavements, and fiber-reinforced concrete pavements. Their performance and rehabilitation is not expected to be like any of the other more conventional rigid pavement types, and such designs require special care in their maintenance and rehabilitation.

4. BASIC RIGID PAVEMENT RESPONSES

Rigid pavements perform by distributing applied vehicle loads over a wide area. Due to the comparatively high modulus and rigidity, the PCC portion of the pavement structure alone supports most of the load. Nonetheless, rigid pavements, whether placed directly on the subgrade soil or over one or more bases and subbases, do deteriorate. Deterioration, whether due to traffic, environment, materials problems, or construction, can be said to be a result of excessive stresses.

The development of stresses that exceed the structural capacity of the concrete are the primary cause of concrete pavement deterioration. As shown in figure 4-1.2, tensile stresses, which concrete does not withstand as well as compressive stresses, are caused by many different factors. However, it must be stressed that the stresses shown in this figure represent a major simplification. None of these will occur independently of the others; considering slab weight and the additive effect of these stresses, the magnitude and even the sign of the stresses can change.

Traffic Loading

Traffic loads are a major source of stresses in a pavement. A load that passes at the pavement edge can create critical stresses if located at mid-panel or at the slab corner. If the load is applied to the slab at mid-panel, the bottom of the slab deflects slightly, creating a tensile stress at the bottom of the slab and a compressive stress at the top. If the load is at the slab corner, it can induce a tensile stress at the top of the slab and a compressive stress at the bottom. When the tensile stress is greater than the tensile strength of the concrete, a transverse crack will begin to form.

Temperature

As ambient temperatures change throughout the day, the temperature of concrete pavement also changes. This temperature cycling creates a temperature gradient in the slab; during the day the temperature at the top of the slab will be greater than the temperature at the bottom of the slab, while at night the opposite is true. The temperature gradient causes the slab to curl downward (daytime) or curl upward (nighttime), either of which can induce stress in the pavement. Depending upon the time of day, these curling stresses can either add to or subtract from the effect of the load-induced stresses.

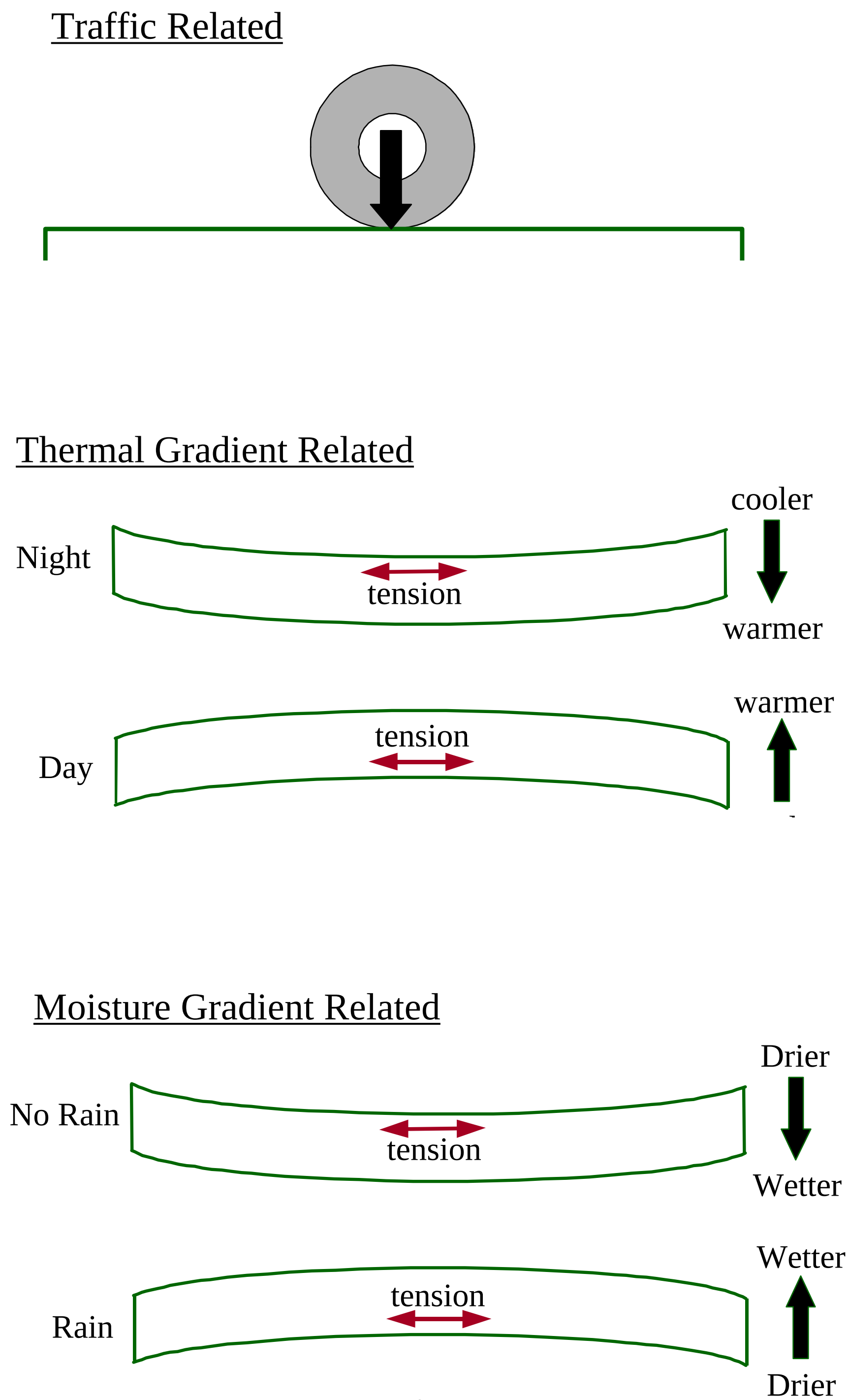


Figure 4-1.2. Various causes of stress in a rigid pavement slab.

Moisture

Warping stresses also occur in PCC due to variations in the moisture content from the top to the bottom of the slab. These variations usually cause the pavement to curl upward as a result of a moisture gradient. While environmental stresses alone may be sufficient to cause distress, the greatest damage is caused in conjunction with traffic loadings.

Other Stresses

As concrete cures, it loses volume due to water loss. This volume change creates shrinkage stresses which may lead to pavement deterioration. Even material-related distresses, such as D-cracking or alkali-silica reactivity, are ultimately linked to stress failures.

5. CONCRETE PAVEMENT MATERIALS

The most commonly used types of cement are shown in table 4-1.1. In most pavement applications a Type I cement will be used. However, there are several applications in which specialty mixes are increasingly common. For example, a range of high early strength (HES) mixes are used when conventional concrete curing times are too long. These mixes may be used in patching, full-depth repair, and whitetopping applications.

Table 4-1.1. Conventional types of cement.

Cement Type	Differentiating Characteristic(s)
Type I	Normal
Type Ia	Type I with air entraining agent
Type II	Moderate heat of hydration, moderate sulfate resistance
Type IIa	Type II with air entraining agent
Type III	High early strength
Type IIIa	High early strength with air entraining agent
Type IV	Low heat of hydration, low strength gain
Type V	High sulfate resistance

6. SUMMARY

Cementitious materials have been used for thousands of years and the existence today of ancient structures in which these materials were used either as a mortar or as a structural block is a testament to its strength as a construction material. Concrete pavements were first constructed in the United States over 100 years ago, and are widely used today where heavy loads are anticipated and long-term reliability is sought. Good concrete pavement performance begins with the selection of appropriate materials, including high quality aggregates and admixtures that are appropriate for the paving conditions.

There are many different types of concrete pavements, but the most commonly constructed are jointed plain, jointed reinforced, and continuously reinforced concrete pavement. These may or may not include a base or subbase. The pavement types are differentiated by the joint spacing and the amount of reinforcing steel.

7. REFERENCES

1. Huang, Y.H. 1993, "Pavement Analysis and Design," Prentice-Hall, Inc., Englewood Cliffs, NJ.

MODULE 4-2

JOINT SEALING FOR RIGID PAVEMENTS

1. INSTRUCTIONAL OBJECTIVES

The sealing and resealing of joints in rigid pavements is an important part of preserving an agency's investment that is often not adequately considered by those responsible for pavement maintenance and rehabilitation. If performed effectively and in a timely manner, joint sealing can help reduce pavement deterioration and thereby prolong pavement life.

This module describes recommended procedures for joint sealing operations. Upon successful completion of this module, participants will be able to accomplish the following:

1. Identify the major factors that affect joint sealant performance.
2. Describe the steps involved in resealing rigid pavement joints.
3. Identify the primary sealant types, appropriate specifications, and sealant properties.
4. Understand the factors to consider in designing an effective joint sealant installation.

2. INTRODUCTION

Joint sealing is a commonly performed pavement maintenance activity that serves two primary purposes. One objective is to reduce the amount of moisture that can infiltrate a pavement structure, thereby reducing moisture-related distresses such as pumping. A second objective is to prevent the intrusion of incompressible materials into joints so that pressure-related distresses (such as spalling) are prevented.

Sealants become ineffective anywhere from 1 to 4 years after placement.(1,2) However, improvements in sealant materials, an increased recognition of the importance of a proper reservoir design, and an emphasis on effective joint preparation procedures are expected to increase the expected life of sealant installations.(3) This is indeed necessary, as joint sealing constitutes a major maintenance activity among highway agencies. A 1985 summary of State experience with various rehabilitation techniques showed 45 States conduct joint sealing operations on rigid pavements.(4) In a 1996 report on preventive pavement maintenance, results of a nationwide survey distributed to 65 State and Provincial highway agencies reported that 35 agencies reseal rigid pavement joints as part of their maintenance strategy.(5) At the same time, there is a persistent controversy over whether joint sealing is needed at all.(6,7,8)

In 1990, the Strategic Highway Research Program (SHRP) initiated a nationwide study (SHRP Project H-106) of the performance of joint and crack sealing materials placed in different configurations. Test sections were installed in "22 sites throughout the United States and Canada between March 1991 and February 1992".(3) From this study, a manual was developed that addresses the effectiveness of various sealing materials and procedures for pavements under different conditions, but evaluation of the sites is ongoing.(9) Another ongoing study that has a significant impact on joint sealing (and resealing) is the Long-Term Pavement Performance (LTPP) SPS-4 studies, in which the effect of joint sealing on pavement performance is being evaluated.

3. DEFINITIONS

Sealing Operations

In rigid pavements, most resealing is conducted at transverse joints, although often the longitudinal joints (lane-shoulder or lane-lane) are sealed at the same time. The sealing of joints, particularly transverse joints, is generally regarded as a more critical operation than crack sealing, primarily because more careful preparation and higher-quality sealants are needed to ensure good performance.

Sealant Materials

There are many different sealant types available; some are designed for specific applications, but for the most part they are all intended for the same purpose. The general categories used by the American Concrete Institute (ACI) to differentiate among sealing materials are:[10]

- Thermoplastic materials
  - Hot-applied
  - Cold-applied

- Thermosetting materials
  - Chemically cured
  - Solvent release

- Preformed compression sealants

Table 4-2.1 provides a summary of the different sealant materials by category, including applicable specifications and typical costs.

<u>Thermoplastic Materials</u>

Thermoplastic sealants are bitumen-based materials that soften upon heating and harden upon cooling, usually (and preferably) without a change in chemical composition. These sealants vary in their elastic and thermal properties and are affected by weathering to some degree. Many thermoplastic sealants are applied in a heated form, although some are diluted and can be installed without heat.

Hot-Applied

Among the more widely used hot-applied thermoplastic sealants are asphalt cement, asphalt rubber, rubberized asphalt, fiberized asphalt, and polyvinyl chloride (PVC) coal tar. Asphalt cement has been used for many years as a sealant material, although with very limited success due to its poor elastic properties. While inexpensive, it is not generally used for sealing medium and high-volume roadways.

Asphalt rubber sealing materials have been widely used on highways, airfields, and city streets. These sealants are a blend of asphalt and suspended, unmelted rubber particles that results in improved elasticity, greater cohesiveness, and an increased softening point in comparison to asphalt cement. The quality of the asphalt rubber varies with the quality of rubber incorporated. A standard specification for asphalt rubber materials is ASTM D 5078.

Table 4-2.1. Typical sealing and filling materials.[9]

SEALANT/FILLER MATERIAL	EXAMPLE PRODUCT	APPLICABLE SPECIFICATION(S)	COST RANGE ($/kg)
Thermoplastic Materials			
Asphalt Cement	AC-10, AC-20	ASTM D 3381	0.11 – 0.33
Asphalt Emulsion	CRS-2, HFMS	ASTM D 244, ASTM D 977, ASTM D 2397	0.11 – 0.33
Polymer-Modified Asphalt Emulsion	Witco CRF, Hy-Grade Kold Flo	ASTM D 244, ASTM D 977, ASTM D 2397	0.88 – 1.22
Asphalt Rubber (hot-applied crack filler)	Koch 9000, Crafco AR2	ASTM D 5078	0.44 – 0.66
Fiberized Asphalt	Kapejo Bonifibers + AC Hercules FiberPave + AC	---	0.33 – 0.55
PVC Coal Tar	Crafco Superseal 444, Meadows Gardox	ASTM D 3406	1.44 – 2.10
Rubberized Asphalt	Koch 9005, Crafco 221, Meadows Hi-Spec	ASTM D 1190, D 3405 AASHTO M 173, M 301	0.44 – 1.10
Low Modulus Rubberized Asphalt	Crafco 231, Koch 9030, Meadows Sof-Seal	Modified ASTM D 1190, D 3405 Modified AASHTO M 173, M 301	0.77 – 1.44
Thermosetting Materials			
Polysulfide	Koch 9015	Fed. Spec. SS-S-200E	2.21– 2.76
Polyurethane	Vulchem, Sikaflex, Burke U-Seal	Fed. Spec. SS-S-200E	6.08 – 7.18
Silicone	Dow 888, Mobay 960, Crafco RS	Fed. Spec. TT-S-001543A, TT-S-00230C	5.52 – 7.73
Preformed Compression Seals			
Neoprene	D.S. Brown "Delastic"	AASHTO M 220, ASTM D 2628	

In the past 15 years, 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. Most of the rubberized asphalt materials fall under ASTM D 3405.

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.

Polyester, polypropylene, and polyethylene fibers have also been added to asphalt cement in an effort to enhance sealant performance, since fibers improve the tensile strength characteristics of the resulting material. However, extensibility and resistance to softening are only slightly improved and adhesion is reduced in relation to the original asphalt cement. Currently, there is not any standard specification that directly addresses fiber-modified sealants.

Several agencies have used PVC coal tar sealant materials. These materials are fuel and jet blast resistant, have a high softening point, and bond well to concrete. Installation requires accurate temperature control, care in adding fresh sealant, and careful observation of continuous heating limits. However, due to variable performance in the field and the potential health problems associated with coal tars, PVC sealants are not widely used. These materials are covered by ASTM D 3406.

Cold-Applied

Cold-applied asphalt cutbacks and emulsions have been used as joint sealant materials with limited success. Cutbacks consist of asphalts that have been thinned with light petroleum solvents. As the solvents evaporate, the asphalt cures, thus eliminating the need for heating, as with other asphalt-based sealants. However, poor performance and environmental restrictions have substantially reduced the use of asphalt cutbacks. ASTM D 1850 is occasionally used to specify these sealant materials.

An asphalt emulsion is a mixture of small asphalt particles suspended in water and an emulsifying agent (such as soap). Such emulsions can be applied with or without heating to wet or dry joints. They maintain fair elastic properties, but are temperature-sensitive and prone to tracking.[11] Modifying agents such as rubber and polymer are added to some emulsions to improve performance.

<u>Thermosetting 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 four to ten times greater than standard rubberized asphalt. However, thermosetting sealants often are placed thinner and may have lower labor and equipment costs.

Chemically Cured

Chemically cured sealants are the predominant type of thermosetting materials used in highway sealing applications. They include polysulfides, polyurethanes, silicones, and epoxies. The advantage of many of these materials is their durability and ease of handling, and in particular the fact that they are not heated during application. On the other hand, the material costs are higher as a group than the thermoplastics.

The performance of polysulfide and polyurethane sealants has been variable. While these materials seem to retain elasticity fairly well, their adhesive capabilities (and in particular those of the polysulfides) are questionable. In addition, most of these materials are two-component materials; the need to mix the two parts introduces an additional step in the sealing operation, which increases sealing time and introduces another source for error.

Silicone sealants are one-part, cold-applied materials that have been used in the paving industry since the 1970s. Their properties include good extensibility, resistance to weathering, and temperature susceptibility resistance. These sealants have good bonding strength in combination with a low modulus that allow them to be placed thinner than the thermoplastic sealants.[12] Due to silicone's thin layer application and lower associated equipment costs, the ratio of in-place cost compared to rubberized asphalt is not nearly as high as the ratio of material cost (for a given volume).

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. Recently developed self-leveling silicone sealants can be placed in one step since they freely flow to fill the joint reservoir without tooling. Performance of silicone sealants is typically tied to joint cleanliness and tooling effectiveness. Some States, such as Georgia and Kentucky, have developed their own silicone specifications, but there are currently no ASTM or AASHTO standards.

Solvent Release

Included in this category are those thermosetting sealant materials that cure through the release of solvent. Examples of these materials include chlorosulfonated polyethylene and certain butyl and neoprene products.[10] However, because the extension-compression range of these materials does not exceed $\pm$ 7 percent, they are not used on pavements.

Preformed Compression Seals

Preformed compression seals are premolded strips of styrene, urethane, neoprene, or other synthetic materials that are designed to be placed in PCC pavement joints in a state of compression. Generally, these seals are designed to be compressed 20 to 50 percent of their uncompressed width. Thus, the opening for a joint sealed with a 25-mm compression seal should remain between 13 and 20 mm. If the seal is too narrow or if the joint opens excessively, the seal, not being compressed, will fall to the bottom of the joint or be pulled out by traffic. If the seal is subjected to compressions greater than the 50 percent level for extended periods of time, the seal may take a "compression set." When this happens, the seal will not open to follow the movement of the joint, and the seal will no longer be effective. The forces that act on a preformed sealant in a pavement joint are shown in figure 4-2.1. In figure 4-2.1(a), the forces on the installed sealant are shown. As the slabs expand, the sealant compresses, as shown in figure 4-2.1 (b).

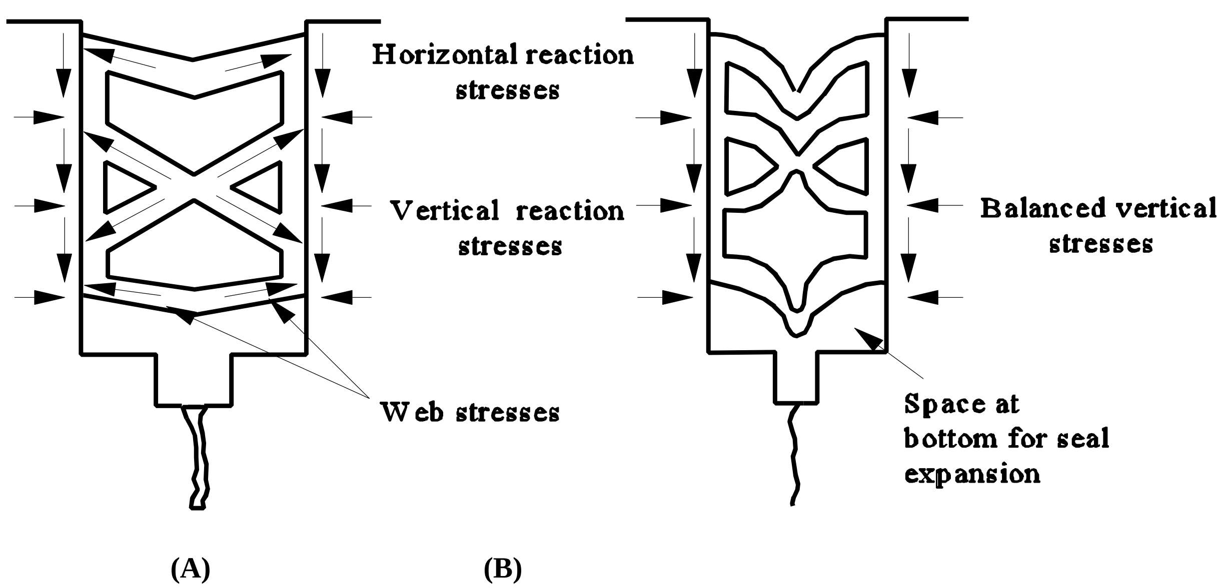


Figure 4-2.1. Preformed compression seal.

Preformed sealants have a good history of performance when used with new rigid pavements. However, for several reasons they are not commonly used on rigid pavement restoration projects. These include lack of uniformity of joint openings over time (and the resultant need for different sized sealants), different joint shape factors, high costs, and the need for vertical sidewalls. Standard specifications for preformed compression seals are found in AASHTO M 220 and ASTM D 2628.

4. PURPOSE AND APPLICATIONS

In rigid pavements, free water entering a joint 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 in rigid pavements interfere with normal opening and closing movements, causing compressive stresses in the slab and increasing the potential for spalling. If the compressive stress exceeds 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.

Resealing joints in rigid pavements is necessary when the existing sealant has deteriorated to the extent that incompressibles and water are able to infiltrate the pavement structure. 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, slab stabilization, and diamond-grinding.

5. LIMITATIONS AND EFFECTIVENESS

Joint resealing should be performed when the existing sealant material is no longer performing its intended function. This is indicated by missing sealant, in-place sealant that is not bonded to the joint faces, or sealed joints that contain incompressibles. The optimum time to perform joint resealing is in the spring or the fall when moderate installation temperatures are prevalent.

If effectively sealed, cracks in rigid pavements will also deteriorate less and contribute less to the overall deterioration of the pavement. Crack sealing is most effective when performed on rigid pavements that exhibit minimal structural deterioration and in which the cracks are tight and have minimal spalling.

Many sealant failures have been attributed to poor or inadequate preparation of the joint and to poor material handling procedures.[1,2,13] Proper preparation of the joint is essential to ensure that the sealant material can effectively bond to the sidewalls. If laitance, dust, water, or other debris remains on the joint faces, a good bond will not be achieved.

Many of the newer sealant materials are sensitive to heating and application temperatures, and narrower temperature ranges are being recommended by manufacturers and noted in the appropriate ASTM sealant specifications.[14] 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.

Some specific problems have been reported with silicone sealant performance. These may be due to an incompatibility between the sealant and certain aggregates (e.g., carbonates), to the presence of residue in the reservoir, or to moisture in the reservoir.[2,8]

Joint Sealing/Resealing and Pavement Performance

As mentioned earlier, the effectiveness of joint sealing has been questioned by some agencies. For example, one agency contends that the purported benefits derived from transverse joint sealing do not offset the costs of sealing and resealing operations.[6,7] This has helped spur another look at the conditions in which sealing and resealing rigid pavement joints are appropriate, and it may be the case that there are some conditions in which joint sealing does not provide a cost-effective extension of the pavement's service life. As part of SHRP's SPS-4 studies, the effect of sealing activities on rigid pavement performance is being examined. While this debate may never be completely resolved, those efforts should go a long way toward identifying whether sealing activities are effective and under what conditions they should be applied. Nonetheless, the overwhelming majority of States' experiences support the contention that sealing cracks and resealing joints is a meaningful rehabilitation activity, within the constraints discussed elsewhere in this module.

Sealant Material Performance

As noted previously, there are many factors that affect performance of a sealant installation. In Geoffroy's nationwide survey of preventive maintenance effectiveness, he notes that sealants are applied an average of every 7 to 8 years, with an observed increase in pavement life of 5 to 6 years.[5] However, these average results are being exceeded in many cases. The performance of sealant materials is being studied nationwide under research initiated as part of SHRP Project H-106 and continued by the FHWA under a long-term monitoring project. At five sites around the country, the performance of 15 different materials and up to four different application methods has been monitored since their installation in 1991. This research does not address whether sealing rigid pavements improves or maintains the overall condition of the pavement, but rather which materials are most cost-effective. The most recent findings suggest that silicone sealants are the most cost-effective.[15] Conclusive results, however, must await the completion of the study so that the actual performance lives of the different applications can be used in the evaluation.

The lack of conclusive results on the performance of sealants is also found in the SPS-4 studies of joint sealants, which cover five sites, three different sealing configurations, and 20 different sealing materials. A field trial of 18 different concrete joint sealants is also being conducted in the United Kingdom.[16] In a 1992 report that followed 5 years of service, only five sealants showed over 10 percent adhesion failure. Where joint movements were greater than expected, failure usually occurred, but overall there were not yet enough failures to permit conclusions about long-term performance.

6. DESIGN CONSIDERATIONS

There are several factors that influence the performance of a concrete pavement sealant. These include joint movement, the reservoir shape factor, and the properties of the sealant itself, and are best considered carefully in a sealant design process that is performed prior to the actual joint sealing operation.

Transverse Joint Movement

Contraction joints in rigid pavements experience more movement than tied construction joints and longitudinal joints, which typically undergo little movement. The types of movements that commonly occur at a transverse contraction joint include:

- Vertical movements due to differential deflection.
- Vertical movements due to differential faulting.
- Horizontal and vertical movements caused by changes in pavement temperature and moisture content.

Differential vertical deflection occurs as heavy wheel loads pass over joints in curled or poorly supported pavements. In cases of high vertical deflection, load transfer and/or base support should be improved prior to any resealing operation.

Permanent settlement, or faulting, results from pumping of erodible base material. Sealant remaining in a faulted joint experiences extreme vertical shear stresses that can lead to failure.[17] As in the case of high vertical deflection pavements, vertical faulting must be properly corrected by grinding prior to sealing and the causes of the faulting should be addressed to promote good long-term pavement performance.

Despite the importance of vertical movements, only horizontal movements are generally considered in determining joint movements for design. The amount of horizontal opening is a function of the joint spacing, the friction between the slab and the base, the concrete's coefficients of thermal expansion and of drying shrinkage, and the temperature range to which the pavement is subjected. In lieu of seasonal measurements of actual joint movements, the following equation may be used to obtain an estimate of the average horizontal joint opening:[18]

$$\Delta L = CL(\alpha \Delta T + e) \tag{4-2.1}$$

where:

ΔL = Joint opening, mm
α = Thermal coefficient of contraction for portland cement concrete (PCC),typically 9 to 11 x 10^{-6} mm/mm/ °C
e = Drying shrinkage coefficient of the PCC, typically 0.00127 to 0.00635 mm/mm, but ignored for resealing projects (i.e., e = 0)
L = Joint spacing, mm
ΔT = Temperature drop (temperature at placement minus the lowest mean minimum monthly temperature)
C = Subbase/slab friction resistance adjustment factor (0.65 for stabilized subbase, 0.80 for granular subbase, 1.0 for subgrade soil)

This equation provides a fairly good estimation of the expected movement at joints. Of course, not every joint will open the same amount due to variations in material properties and in the frictional restraint conditions. Also, on new pavements every joint will not crack initially, leaving intermediate joints uncracked (typically one in four to one in seven joints will crack initially). The joints that crack first usually remain wider throughout their lives.

Using equation 4-2.1, figures 4-2.2 and 4-2.3 illustrate the effects of temperature, joint spacing, and base type on calculated joint movements. Figure 4-2.2 shows the effect of temperature change and joint spacing on the opening in rigid pavement joints. The movement occurring at the joint increases not only with an increase in the temperature differential, but also with an increase in joint spacing. Figure 4-2.3 shows the effect of frictional resistance on the calculated joint movements, and indicates that pavements constructed on stiffer base course materials (such as cement- or asphalt-treated bases) exhibit fewer joint openings due to the increased friction between the base and the slab.

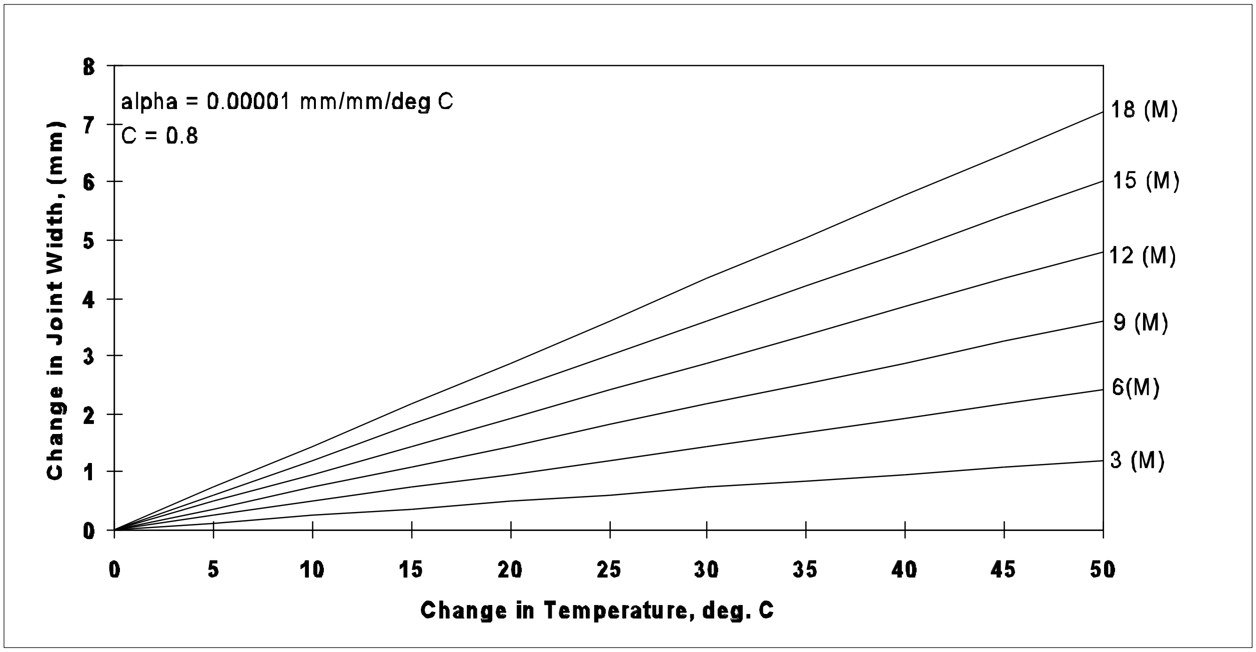


Figure 4-2.2. Joint opening as a function of temperature change and joint spacing.

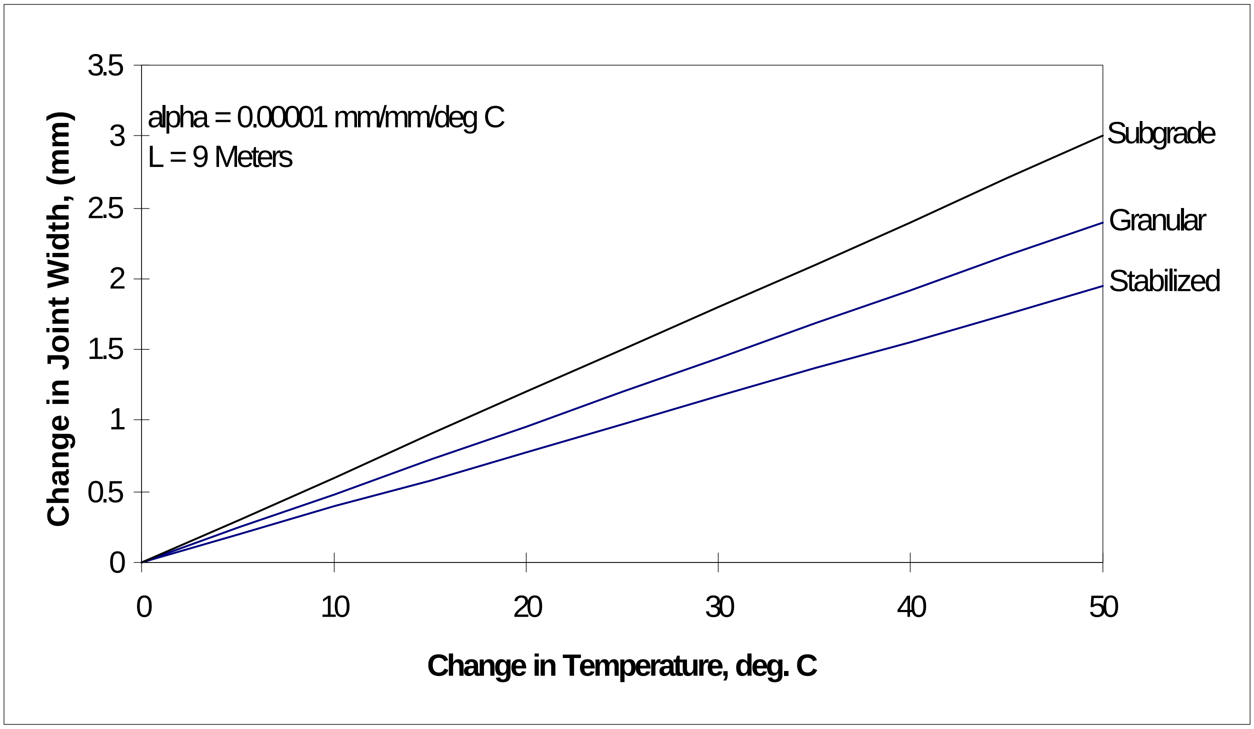


Figure 4-2.3. Joint opening as a function of temperature change and base type.

Vertical movements are a primary design concern for sealing the longitudinal joint between a mainline rigid pavement and a flexible shoulder. 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.[19] 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.

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 conducted by Tons nearly 40 years ago indicated 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.[20] Figure 4-2.4 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.

In 1959, Tons introduced the concept of a shape factor, in which the dimensions of a sealant material are expressed in terms of the ratio of its width (W) to its depth (D).[20] This is illustrated in figure 4-2.5. A proper shape factor minimizes the stresses that develop within the sealant and along the sealant/pavement interface as the joint opens.

For good performance, the sealant must also be kept from bonding to the bottom of the reservoir. A backer rod, shown in figure 4-2.5, 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.

The depth to which a joint is sawed should include allowances for the sealant material, the backer rod (if used), and the sealant recess (if any). Excessively deep sawing should be avoided because it not only slows productivity and costs more, but also decreases the joint surface area available for aggregate interlock load transfer. However, in new construction deeper sawcuts are essential to prevent random cracking if the slab bonds to a stabilized base or subbase.

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 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.

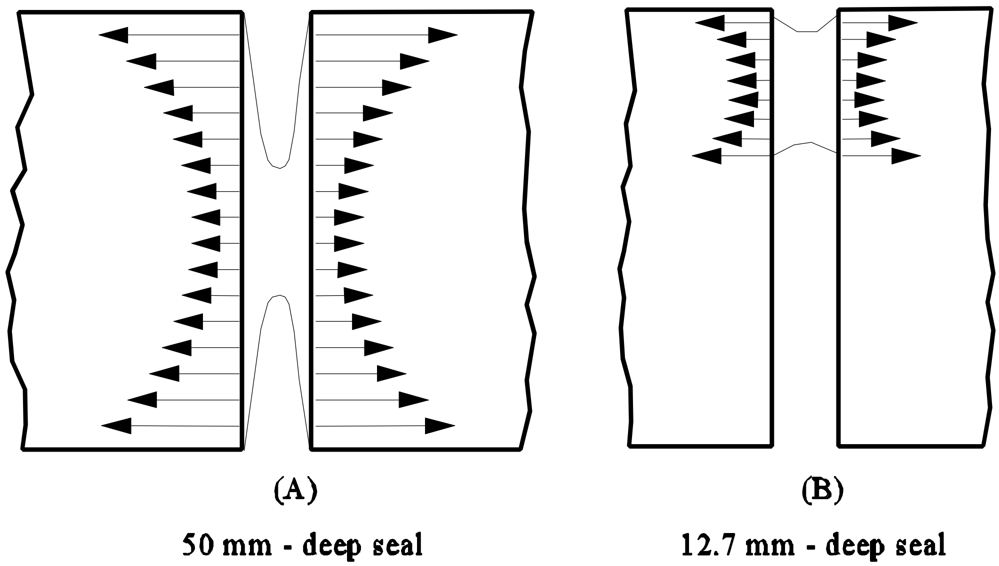


Figure 4-2.4. Relative effect of shape factor on sealant stresses.

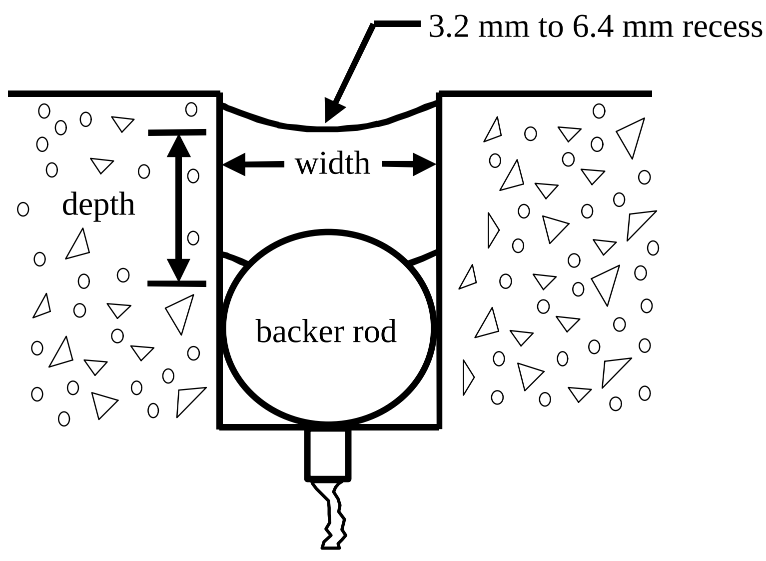


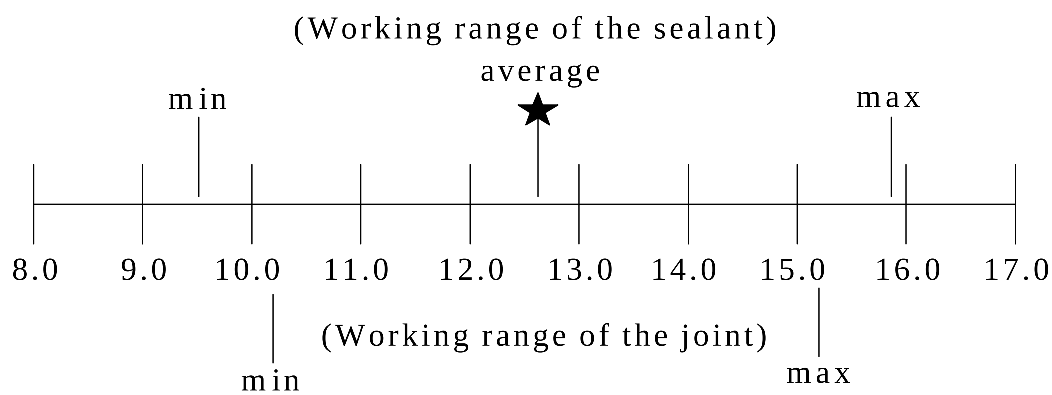
Figure 4-2.5. Illustration of sealant shape factor.

As an example, a thermoplastic material placed in a 13-mm wide joint can withstand an opening of 3.25 mm (13 mm x 25 percent) before exceeding a strain of 25 percent. A silicone material placed in a 13-mm wide joint can withstand an opening of 6.5 mm (13 mm x 50 percent) before exceeding a strain of 50 percent.

Due to strain limitations, the shape factor (W/D) generally recommended to control stresses in hot-poured thermoplastic sealants is 1:1 to 1:2, while a 2:1 shape factor is suggested for silicone sealants. It is also generally recommended that the sealant be recessed between 3.2 and 6.4 mm below the surface of the pavement. These recommendations assume that the joints are opened to a uniform width. In pavements in which the joints did not all crack uniformly, joint openings, and therefore sealant reservoirs, may need to be sawed to different widths.

<u>Design Example</u>

To illustrate the process of determining the working ranges of a joint and a sealant material, consider the example shown in figure 4-2.6. Assuming a 12.2-m long slab, a stabilized base, and a temperature change of 33 °C, equation 4-2.1 estimates a joint opening of 2.5 mm. Using this value for design purposes, and assuming that the average joint width at the time of sealing is 13 mm, then the width of the joint will vary between 11.4 and 14.0 mm throughout the year. A thermoplastic joint sealant material with an allowable strain of 25 percent will tolerate movements of 3.2 mm, meaning that it can withstand joint openings from 9.5 to 16 mm. Thus, the sealant material should accommodate the anticipated joint movements. Had the sealant material not been acceptable, either a more elastic material would have to be selected or a wider joint width would be needed.



Average joint width at time of sealing = 12.7 mm
Working range of sealant = 12.7 mm ± (0.25 x 12.7 mm)
= 9.5 mm to 15.9 mm
Working range of joint, L = 12.7 mm ± 2.54 mm
= 10.2 mm to 15.2 mm

Figure 4-2.6. Working range of transverse joint and thermoplastic sealant material.

Sealant Configurations

Joints in rigid pavements are typically sealed in the configuration shown in figure 4-2.5. However, some manufacturers of hot-poured thermoplastic materials recommend that the recess be eliminated and that the joint be filled to 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. The effect of sealant configurations on joint performance is currently being studied in great detail in the sealant long-term monitoring studies and the long-term pavement performance (LTPP) SPS-4 study.

Sealant Properties

Critical sealant properties that affect the performance of the sealant material include the following:

- Durability.
- Extensibility.
- Resilience.
- Adhesiveness.
- Cohesiveness.

Durability refers to the ability of the sealant to withstand the long term 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 nondurable 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 is, the lower are its 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 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 incompressibles 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 movement and the presence of incompressibles for specific climatic regions.

As a sealant material in a joint 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 sidewalls determines the sealant's bonding ability. Cohesive failures are more common in sealants that have hardened significantly over time.

Typically, higher-quality sealants are specified for joint resealing operations than for crack sealing operations. However, it may be more cost-effective to use lower quality sealants in joints that experience little movement or that will be overlaid or otherwise rehabilitated in the near future.

7. COST CONSIDERATIONS

While all sealant types are used in new concrete construction, concrete joint resealing and crack sealing operations generally employ hot-applied thermoplastics materials and chemically cured thermosetting materials. Table 4-2.1 shows various sealant types and typical cost ranges. Note that these costs are material costs only; labor, traffic control, and equipment costs vary considerably and are not included. 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 be less than less expensive sealants. In the final analysis, however, the true measure of cost effectiveness is that the sealant must extend the service life of the pavement by an amount that at least exceeds the discounted value of its originally installed cost.

8. CONSTRUCTION SEQUENCE

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 rigid pavements consists of the following steps:

- Removal of the old sealant.
- Refacing of the joint sidewalls.
- Cleaning of the joint reservoir.
- Installation of the backer rod.
- Installation of the new sealant.

Each of these steps is discussed in the following sections.

Sealant Removal

The old sealant must be completely removed from the joint sidewalls prior to refacing concrete joints. 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 water blasting.

Diamond-bladed sawing as a means of sealant removal has gained acceptance because it combines the sealant removal and refacing steps into 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.

Sealant Reservoir Refacing

Refacing is the creation of the specified joint width through diamond sawing of the existing joint sidewalls. 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.

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. Refacing is generally done using a 26 to 46 kW 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 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 3.2 mm; 1.6 mm on each face.

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.

Joint 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 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.

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 (> 620 kPa) clean, dry air to remove sand, dust, and other incompressibles that remain in the joint. A backpack blower typically cannot generate sufficient air 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.

Power-driven wire brushes should never be used to remove old sealant or to clean a joint in a rigid 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.

Backer Rod Installation

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.

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. Take care to ensure that the rod is installed to the proper depth and that no gaps exist at the intersections of backer rod strips. Stretch the rod as little as possible to reduce the likelihood of shrinkage and the resultant formation of gaps.

Sealant Installation

As soon as possible after backer rod placement, install the sealant material. This helps to avoid problems, such as condensation on the backer rod and debris collecting in the reservoir, that occur when the backer rod is left in place too long before the sealant is placed.

Hot-Poured Thermoplastic Sealant Materials

Hot-poured thermoplastic sealant materials should be placed only when the air temperature is at least 8 ^{o}C and rising. 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 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, traffic should not be allowed on the newly sealed joints for about 30 minutes to 1 hour after sealant placement.

It is important to follow the manufacturers' 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. A study in Utah indicated that the temperatures shown on the thermometers of the DOT’s melter/applicators were at times more than 10 ^{o}C different from the actual sealant temperature.(14) Using an additional thermometer to monitor sealant temperatures can help eliminate damage due to sealant overheating.

Low-Modulus Silicone Sealants

Silicone sealants should not be placed at temperatures below 4 ^{o}C. As with the thermoplastic materials, apply silicone sealants 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 2.0 or slightly higher. It is not recommended that they be placed any thinner than half the width of the joint, with a minimum thickness of 6 mm. Traffic should not be allowed on the pavement for about 1 hour after sealant placement.

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 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.(21)

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.

Compression Seals

Preformed compression seals may be used for joint resealing, but their use requires that extra attention be given to joint preparation. The joint sidewalls must be perpendicular and have a very uniform width. Spalls that extend downward below the top of the compression seal must be repaired no matter how minor they appear. These spalls cause the seal to work its way out of the joint, resulting in failure of the seal. Lack of this extra preparation for resealing with compression seals has produced poor results in some preformed resealing projects. Compression seals have generally performed very well, however, in new sealing operations.

Compression seals are generally designed to be compressed between 20 and 50 percent of their uncompressed width. The sealant manufacturer should be consulted to select appropriate sizes for a particular pavement. Certain pavements may have widely varying joint widths, in which case several different sizes of compression seals must be available on the job site.

The sealing operation with compression seals requires that a lubricant/adhesive be applied to the joint sidewalls immediately prior to insertion of the compression seal. This material eases the insertion of the seal and, with time, cures to form a weak adhesive.

The compression seal is inserted vertically into the joint such that it is about 6 mm below the pavement surface. Extreme care must be taken when inserting the seal into the joint to avoid twisting or stretching the seal. If the seal stretches more than 2 to 3 percent, the seal may break or shrink significantly after placement. Most manufacturers of compression seals provide installation machines that eliminate this problem.

Longitudinal Joint Sealing

Two types of longitudinal joints in rigid pavements may also be addressed in a resealing operation: longitudinal joints between adjacent PCC slabs, and the longitudinal joint between the mainline rigid pavement and the flexible shoulder. While the procedures are essentially the same as transverse joint resealing, some additional considerations should be noted.

PCC to PCC Joints

Longitudinal joints between adjacent PCC slabs are found between adjacent traffic lanes or between a PCC mainline pavement and a PCC 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.

Rigid Mainline/Flexible Shoulder Joint

The longitudinal joint between a rigid mainline pavement and a flexible shoulder is 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. Unlike adjacent PCC lanes, a PCC lane and a flexible shoulder are not tied together so there is often horizontal movement, or separation, that 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 flexible shoulder to allow for the anticipated vertical movements. Barksdale and Hicks[19] recommend a minimum 25-mm wide reservoir, wider if larger vertical movements are expected. The depth of the reservoir should also be about 25 mm. 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. Most agencies use hot-poured thermoplastic materials to seal this joint, although there are newer silicone materials that have been specifically developed for this application.

9. EQUIPMENT

In a joint sealing project, a variety of equipment is needed to perform the sealant removal, joint cleaning, and sealant installation activities. A brief description of the equipment used in joint sealing operations is described in the following sections.

Equipment for Sealant Removal and Joint Refacing

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 joint sidewalls. 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 26 to 46 kW water-cooled saws 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.

Routers

Routers are more commonly used on flexible pavements, although they occasionally may be used on rigid pavements, especially for crack sealing or for preparing the lane-shoulder joint. The most common types are the vertical spindle and rotary impact. Vertical spindle routers, like wood routers, consist of a clustered cutting head which rotates about a vertical shaft to remove concrete to the desired depth and width. To maintain the router position over the crack, a skilled operator is required. If old sealant is present in the crack, it may be smeared on the crack face by this routing operation.

A rotary impact router consists of multiple cutter blades mounted about horizontal shafts that are spaced along the periphery of a circular cutting head. The cutting head rotates about a primary horizontal shaft. As a result, the cutter blades experience two types of motion: circular and translational. Rotary impact routers are not recommended for concrete crack refacing, as they tend to spall the reservoir edges.

Equipment for Joint Cleaning

Obtaining clean, dry sidewalls prior to sealant placement is critical to the performance of the sealant installation. Equipment used for the cleaning operation is described below.

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 of pressure and 4.3 m^3/min of oil- and moisture-free air should be provided. Additionally, nozzles should be modified with deflectors to direct sand against the sidewalls to provide more efficient cleaning.

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 and a blast volume of 4.3 m^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 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. Another way of applying a sealant material, although not commonly used, is with a cornucopia pour pot, which is a hand-held, conical-shaped pot used to apply unheated or partially-heated emulsions into joints.

10. SUMMARY

This module presents information on joint sealing in rigid pavements. The need for sealing operations is discussed, including guidelines for identifying candidate projects. Various sealant materials that are available are presented, along with their properties, applicable specifications, and typical costs.

Important design considerations for joint sealing are described. These include the anticipated joint movements, the shape factor (W:D) of the sealant material as installed, the sealant configuration, and the properties of the sealant.

Procedures for the sealing of both transverse and longitudinal joints in rigid pavements are described. In almost every project, a successful joint resealing operation includes the following steps: removing the old material, refacing the existing joint reservoir, cleaning, installing backer rod, and installing the new sealant material. Extensive guidelines for joint sealing operations are found in references 9, 22, 23, and 24.

11. REFERENCES

1. Peterson, D.E. 1982, “Resealing Joints and Cracks in Rigid and Flexible Pavements,” NCHRP Synthesis of Highway Practice 98, Transportation Research Board, Washington, DC.

2. Permanent International Association of Road Congresses (PIARC). 1992, “Evaluation and Maintenance of Concrete Pavements,” Permanent International Association of Road Congresses, Paris, France.

3. Smith, K. L., D. G. Peshkin, E. H. Rmeili, T. Van Dam, K. D. Smith, and M. I. Darter. 1991, "Innovative Materials and Equipment for Pavement Surface Repairs, Volume I: Summary of Material Performance and Experimental Plans," SHRP-M/UFR-91-504, Strategic Highway Research Program, Washington, DC.

4. Federal Highway Administration (FHWA). 1985a, "Pavement Rehabilitation Design and Techniques Study, Summary of State Experience with 63 Rehabilitation Techniques," National Pavement Initiative No. 1, Federal Highway Administration, Washington, DC.

5. Geoffroy, D.N. 1996, "Cost-Effective Preventive Pavement Maintenance," NCHRP Synthesis of Highway Practice 223, Transportation Research Board, Washington, DC.

6. Shober, S.F. 1986, "Portland Cement Concrete Pavement Performance as Influenced by Sealed and Unsealed Contraction Joints," Transportation Research Record 1083, Transportation Research Board, Washington, DC.

7. Shober, S.F. 1997, "The Great Unsealing: A Perspective on Portland Cement Concrete Joint Sealing," Transportation Research Record 1597, Transportation Research Board, Washington, DC.

8. McGhee, K. H. 1995, "Design, Construction, and Maintenance of PCC Pavement Joints," NCHRP Synthesis of Highway Practice 211, Transportation Research Board, Washington, DC.

9. Evans, L.D. and A. Romine. 1993, "Materials and Procedures for the Repair of Joint Seals in Concrete Pavements," SHRP-H-349, Strategic Highway Research Program, Washington, DC.

10. American Concrete Institute (ACI). 1990, "Guide to Joint Sealants for Concrete Structures," ACI 504R-90, American Concrete Institute, Farmington Hills, MI.

11. Chehovits, J. and M. Manning. 1984, "Materials and Methods for Sealing Cracks in Asphalt Concrete Pavements," Transportation Research Record 990, Transportation Research Board, Washington, DC.

12. Zimmer, T.R., S.H. Carpenter, and M.I. Darter. 1984, "Field Performance of a Low-Modulus Silicone Highway Joint Sealant," Transportation Research Record No. 990, Transportation Research Board, Washington, DC.

13. Blais, E. J. 1984, "Value Engineering Study of Crack and Joint Sealing," FHWA-TS-84-221, Federal Highway Administration, Washington, DC.

14. Belangie, M.C. and D.I. Anderson, 1985 (appendix revised 1987). "Crack Sealing Methods and Materials for Flexible Pavements," FHWA/UT-85/1, Utah Department of Transportation, Salt Lake City, UT.

15. Romine, A.R., et al. 1996, "Long-Term Monitoring of Pavement Maintenance Materials Test Sites," Interim Report of data analysis submitted to FHWA, Federal Highway Administration, Washington, DC.

16. Franklin, R.E., and H. Savage. 1992, "The Performance of Joint Sealants in Concrete Pavements," Research Report 349, Transport Research Laboratory, Crowthorne, Berkshire, Great Britain.

17. Spells, S. and J.M. Klosowski. 1985, "The Importance of Sealant Modulus to Performance of Concrete Highway Joint Sealants Under Vertical Shear," Transportation Research Record 1041, Transportation Research Board, Washington, DC.

18. Darter, M.I. 1977, "Design of Zero-Maintenance Plain Jointed Concrete Pavement, Volume I—Development of Design Procedures," FHWA-RD-77-111, Federal Highway Administration, Washington, DC.

19. Barksdale, R.D. and R.G. Hicks. 1979, "Improved Pavement-Shoulder Joint Design," NCHRP Report 202, Transportation Research Board, Washington, DC.

20. Tons, E. 1959, "A Theoretical Approach to Design of a Road Joint Seal," Highway Research Board Bulletin 229, Highway Research Board, Washington, DC.

21. Federal Highway Administration (FHWA). 1985b, "Pavement Rehabilitation Manual," FHWA-ED-88-025, Federal Highway Administration, Washington, DC.

22. Darter, M. I., E.J. Barenberg, and W.A. Yrjanson. 1985, "Joint Repair Methods for Portland Cement Concrete Pavements," NCHRP Report 281, Transportation Research Board, Washington, DC.

23. American Concrete Pavement Association (ACPA). 1993, "Joint and Crack Sealing and Repair for Concrete Pavements." Technical Bulletin TB-012.0. American Concrete Pavement Association, Skokie, IL.

24. Federal Highway Administration (FHWA). 1996, "Pavement Notebook for FHWA Engineers." FHWA-PD-96-037. Federal Highway Administration, Washington, DC.

MODULE 4-3

PRESSURE RELIEF JOINTS

1. INSTRUCTIONAL OBJECTIVES

This module describes design considerations for the installation of pressure relief joints (PRJ) in rigid pavements. Upon completion of this module, the participants will be able to accomplish the following:

1. Identify the causes of expansive pressures in rigid pavements and the resulting pressure-related distresses.
2. Recognize situations where the installation of pressure relief joints may be warranted.
3. Describe procedures for properly constructing pressure relief joints.
4. Recognize the potential problems associated with the use of pressure relief joints.

2. INTRODUCTION

Due to the accumulation of incompressibles in transverse joints and to critical thermal and moisture conditions, significant expansive pressures can build up in rigid pavements over time. These internal expansive pressures can lead to the development of severe distresses that detract from the performance of the pavement and, in extreme cases, compromise the safety of the roadway. Joint spalling, blowups, and damage to adjacent structures (bridges, manholes) and roadway appurtenances (traffic islands, curbs) are just a few of the distresses that can develop if these expansive pressures continue to develop.

As a means of relieving these internal expansive pressures, pressure relief joints are sometimes installed in existing rigid pavements. These are transverse joints placed the full depth and width of the pavement that provide an open area into which the slab may expand, thereby dissipating the expansive pressures and reducing the associated pavement damage. However, these joints often have a detrimental effect on other rigid pavement features, and they should be used only on pavements that are exhibiting severe pressure-related distresses.(1)

3. DEFINITIONS

General Definition

Pressure relief joints are transverse joints installed after initial construction to relieve expansive stresses in the pavement that otherwise could lead to joint spalling, blowups, or bridge pushing. Pressure relief joints are placed across the full width and depth of the pavement, and are typically 25 to 50 mm wide. They are usually filled with a compressible filler material, such as styrofoam or sponge rubber, to prevent the intrusion of incompressibles. Asphalt concrete and large preformed compression joint seals have also been used as filler materials by some agencies.

Types of Pressure Relief Joints

A wide variety of PRJ types and designs have been used by highway agencies over the years, with the primary function being to provide relief of built-up pressures in the rigid pavement to reduce pressure-related damage. A few of the more common pressure relief joint designs are described below.

Narrow Pressure Relief Joints

Narrow pressure relief joints, typically less than 100 mm wide, are installed using either two cuts of a diamond blade saw or with a single pass of a carbide tooth wheel saw. After removal of the sawed material, filler material (generally a compressible material such as styrofoam or sponge rubber) is placed in the joint to prevent the intrusion of incompressibles. Preformed joint seals have also been used as a filler with some success. After installing the filler place the sealant on top of the filler, slightly recessed below the pavement surface. The sealant material serves to help keep the filler material in the joint and to keep incompressibles from infiltrating. Figure 4-3.1 presents a cross-section of a typical narrow relief joint.(2)

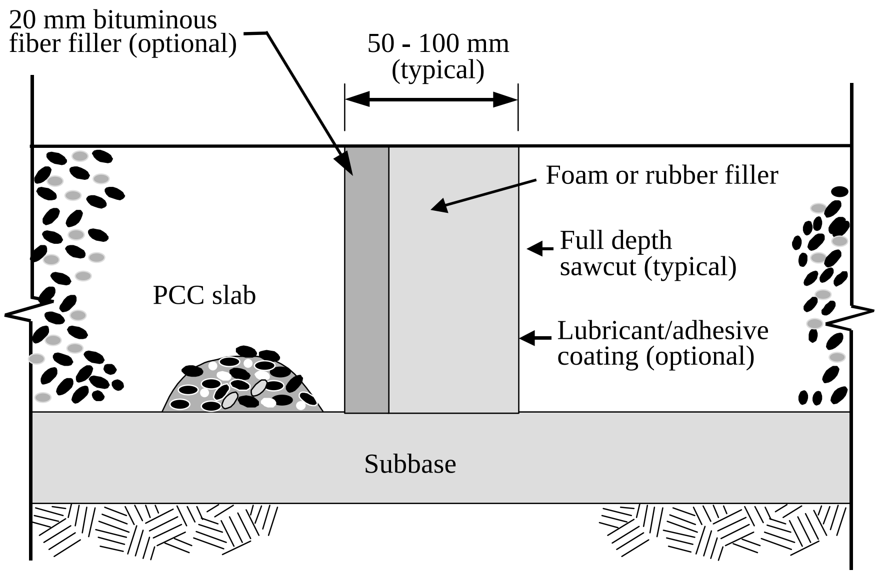


Figure 4-3.1. Cross-section of typical narrow pressure relief joint.(2)

Wide Pressure Relief Joints

Some agencies have constructed full-depth hot-mix asphalt (HMA) patches in rigid pavements to serve as wide relief joints, as shown in figure 4-3.2.(2) These patches are full-lane width and 0.9 to 1.2 m long, and are often placed in deteriorated areas that would otherwise require a full-depth concrete repair. Pavement removal and construction methods are similar to those used in normal bituminous patching operations.

Pressure Relief Joints Within Full-Depth Repairs

A few highway agencies have constructed relief joints at one or both edges of full-depth repairs. These are placed as part of the full-depth repair and are typically 25 to 50 mm wide. If the joints are not doweled, this design has often produced rocking, premature cracking, and spalling of the repair because of poor transverse joint load transfer.

Special heavy-duty expansion joints with dowels for load transfer have been used on highways with heavy traffic, particularly where a bituminous overlay is to be placed. An example of such a design used by the Illinois Department of Transportation (IDOT) is shown in figure 4-3.3.(2) Its performance in Illinois has been fairly good under very heavy traffic loadings.

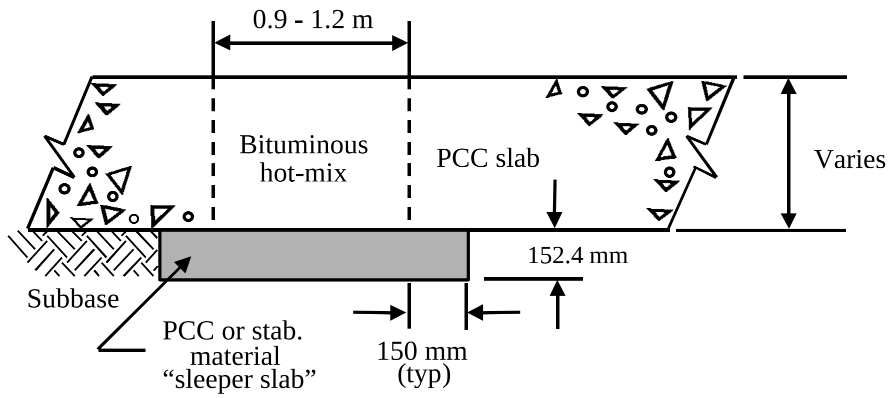


Figure 4-3.2. Cross-section of wide pressure relief joint.[2]

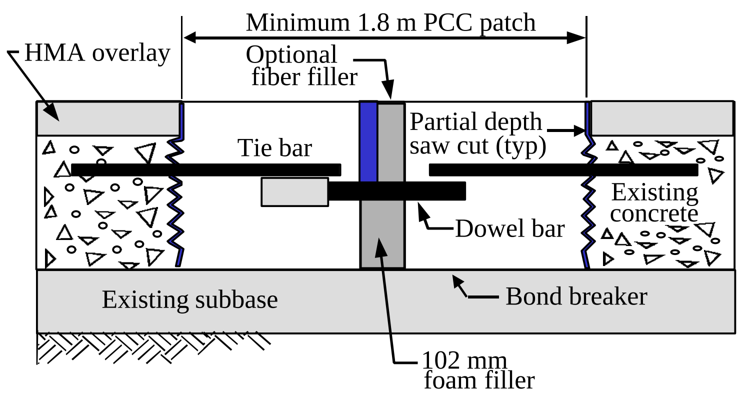


Figure 4-3.3. Illinois DOT heavy duty pressure relief joint design.[2]

In the IDOT design, the pressure relief joints are usually placed in the center of a full-depth repair, with the repair being tied to the original slab. The relief joints are prefabricated assemblies that include dowels, chairs, and a fibrous filler material. The dowels are capped and coated in a similar manner to those used in new construction. While this design does provide load transfer and reduces the occurrence of localized distress in subsequent bituminous overlays placed over the expansion joint, it is expensive and may limit or impair future rehabilitation options (e.g., milling of the asphalt concrete overlay).

4. PURPOSE AND PROJECT SELECTION

Purpose

As previously discussed, pressure relief joints are intended to relieve built-up expansive pressures that may develop in a rigid pavement over time. If these expansive pressures exceed the compressive strength of the portland cement concrete (PCC), joint spalling, longitudinal compression cracking, and blowups of the pavement can result.

The build-up of expansive pressures in a rigid pavement can also affect bridges and other adjacent structures. Incidents of cracked abutments and bridge decks being pushed nearly off of the abutments have been documented for both short- and long-jointed pavements. See references 3, 4, 5, and 6. In addition, continuously reinforced concrete pavements (CRCP) has been found to displace heavy anchor lugs due to high expansive stresses.(7)

Pavement expansion can also damage other secondary structures. These include manholes and other drainage and access structures in the pavement surface. They can be crushed, collapsed, or rendered nonfunctional as they are moved by the pavement. Curbs and traffic islands are also subject to shattering, breakup, upheaval, and failure as the surrounding pavement expands.(3)

<u>Causes of Pavement Expansion</u>

The expansive pressures that build up in rigid pavements over time may be created by one or more of the following factors:

- Intrusion of incompressibles into joints and cracks.
- Expansion of reactive aggregates in the PCC.
- Extremely high pavement temperatures and moisture conditions relative to those that produce a neutral (no stress) condition.

Incompressible materials (e.g., sand, cinders, underlying base course materials) can accumulate in poorly sealed transverse joints and cracks, especially during the cold winter months when the joints and cracks are open the widest. This infiltration may also occur from beneath the slab, where pumped base course materials accumulate in the joint or crack.

The presence of these incompressibles prevent the pavement from expanding during the summer months. The result is the build-up of expansive pressures in the slab over time, which can become even greater with the onset of critical thermal or moisture conditions. If the compressive pressures in the slab exceed the compressive strength of the PCC, joint spalling or blowups may occur.

Greater movement is associated in rigid pavements with longer slab lengths. This is illustrated in table 4-3.1, which shows the computed joint movements for various slab lengths and for temperature differentials of 16 °C and 38 °C. Greater temperature differentials and longer joint spacings produce larger joint movements that must be accommodated by the joint sealant material. If the transverse joints and cracks are contaminated with incompressibles, they cannot effectively close and large compressive pressures can be created. This can take up to 15 years to fully develop to the point when blowups or spalling occurs.

Table 4-3.1. Computed maximum joint movement for various slab lengths.

	CALCULATED JOINT OPENING, mm			
	Stabilized Base		**Granular Base**	
Joint Spacing, m	**Temperature Range**		**Temperature Range**	
	15.6 C°	37.8 C°	15.6 C°	37.8 C°
4.6	0.47	1.13	0.57	1.39
6.1	0.62	1.50	0.76	1.84
9.1	0.92	2.24	1.14	2.75
12.2	1.24	3.00	1.52	3.69
15.2	1.54	3.74	1.90	4.60
30.5	3.09	7.49	3.81	9.22

Joint opening computed using $\Delta L = L[c(\alpha \Delta T)]$, where:

α = PCC thermal coefficient of expansion (10×10^{-6} mm/mm/°C assumed)
c = Adjustment factor for base friction (0.65 for stabilized base, 0.80 for granular base)
ΔL = Joint opening, mm
L = Slab length, m
ΔT = Temperature range, difference between minimum annual temperature and temperature at time of PCC placement, °C.

Pavement expansion can also be due to cement-aggregate reactions, such as alkali-silica reaction (ASR) or alkali-carbonate reaction (ACR). These reactions occur between alkalies in the cement paste and the siliceous or carbonate components of certain aggregates. The product of the reaction is a gel that absorbs water and swells, cracking the cement matrix. With additional moisture, this gel can flow into the cracks and continue to swell, causing progressive expansion and cracking. If the expansion is unrestrained, it can cause a significant volume increase in the PCC, thereby closing transverse joints and pushing bridge structures. If the expansion is restrained, longitudinal cracking, spalling, or shattering of the PCC at the joints can occur over time. Reactive aggregate distress appears as fine, longitudinal cracks located throughout the slab, and may emanate from tightly closed joints. Additional information on the identification of ASR distress is found in the handbook by Stark.[8]

Another source of pavement expansion is the increase in pavement temperature above the neutral temperature. The neutral temperature is defined as the temperature at which the axial force in a pavement is equal to zero; for new pavements, the neutral temperature is approximately the temperature at which the PCC solidified. Kerr and Shade[9] have theorized that pressure damage occurs as the temperature increases above the neutral temperature. Thus, the placement of rigid pavements at higher temperatures may reduce the likelihood of blowups. Also, since blowups are more likely to occur when the PCC is placed at higher moisture contents, minimizing the moisture content of the PCC may reduce the likelihood of blowups. However, since concrete shrinks as it cures, the joints are open somewhat shortly after curing, which provides some expansive space unless the infiltration of incompressibles occurs.

Even changes in the moisture content of the PCC can cause significant pavement expansion.[10] At the AASHO Road Test, length changes as high as 14.0 x 10^{-6} mm/mm per 1 percent decrease in moisture content were reported. Changes in moisture content can occur during curing, as well as during changes in seasons and as the relative humidity in the air changes.

Regardless of the cause of the pavement expansion, one factor that influences the development and magnitude of pavement expansion pressures is the PCC coefficient of thermal expansion, α. This factor can range from 7.1 *x* 10^{-6}/°C to 12.3 *x* 10^{-6}/°C, depending primarily upon the type of coarse aggregate, but also upon the type of fine aggregate and the characteristics of the cement paste.

Mechanisms of Blowups

When the expansive pressures in the slab exceed the compressive strength of the PCC at a given point, spalling, shattering, or blowup of the slab occurs.[11,12] Spalling near the joints reduces the stiffness of the joints and introduces axial force eccentricities into the slabs, making them more susceptible to buckling or "lift-off" blowups.[9,12] Joint spalling also increases the likelihood of shattered slab blowups because slab expansive pressures at joints are resisted by a smaller cross-sectional area of PCC at the joint. Restraint by adjoining structures (e.g., bridge abutments) can also contribute to the development of blowups.[13]

Giffin[14] proposed one theory of blowup development. According to this theory, the first stage of failure occurs prior to the actual blowup when compressive forces in the slab become severe enough to fracture the PCC below the surface. The disintegrating PCC forms an inclined plane below the undamaged PCC. As the compressive forces increase over time, one slab moves up the inclined plane with sufficient force to shear the edge of the adjacent slab (see figure 4-3.4). This explains the observed blowup characteristic of one slab overriding the other.

Deterioration of the lower portion of the slab (due to D-cracking, for example) also shifts the point of application of the compressive forces from mid-depth to some higher point. This results in the development of an effective moment at the joint (see figure 4-3.5) that causes a classic, or lift-off, blowup.[9]

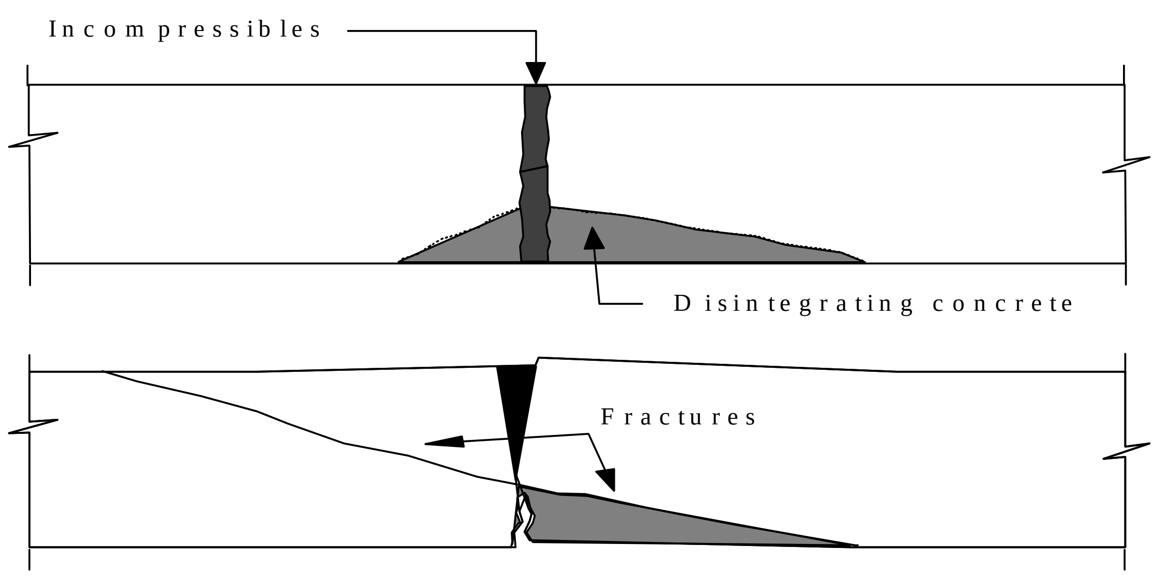


Figure 4-3.4. Development of a blowup along an inclined plane of fracture.[14]

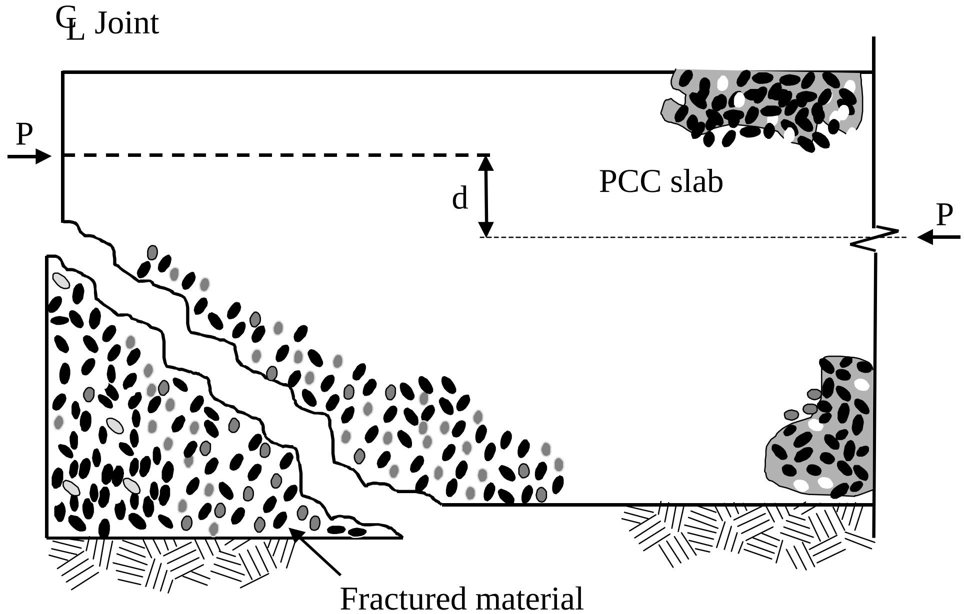


Figure 4-3.5. Development of an effective moment that can result in a lift-off blowup.[9]

Kerr and Shade[9] developed analysis procedures to predict allowable safe temperature increases over a pavement's neutral temperature. These analyses consider the pavement thickness, sliding frictional resistance between the slab and subbase, effective flexural stiffness of the pavement, coefficient of linear thermal expansion, and rotational and axial stiffness of the joints and cracks.

Project Selection

Pressure relief joints are generally recommended only for long-jointed rigid pavements that have a history of blowups or pressure-related damage. In most cases, proper maintenance and regular resealing of transverse joints are the best preventive measures for reducing the build up of expansive pressures in rigid pavements.

Pressure relief joints should not be used in CRCP because they destroy the continuity of the pavement by allowing previously tight transverse cracks to open, which can result in loss of support. Pressure relief joints should be used on these types of pavements only near bridges, when shoving is a problem or if blowups have occurred.[15]

Nondoweled jointed plain concrete pavements (JPCP) are also poor candidates for the use of pressure relief joints, except at bridges. For these pavement types, pressure relief joints will allow the adjacent joints to open, resulting in loss of aggregate interlock load transfer, which can lead to pumping and faulting of those joints. Figure 4-3.6 illustrates the movement of intermediate joints within a typical pavement section with pressure relief joints. The opening of the adjacent joints also allows water to enter the pavement structure, resulting in deterioration of the subbase, pumping, and rocking of the slab. If used at bridges on nondoweled pavements, it is recommended that load transfer be re-established within six to ten contraction joints of the pressure relief joint.[2,16]

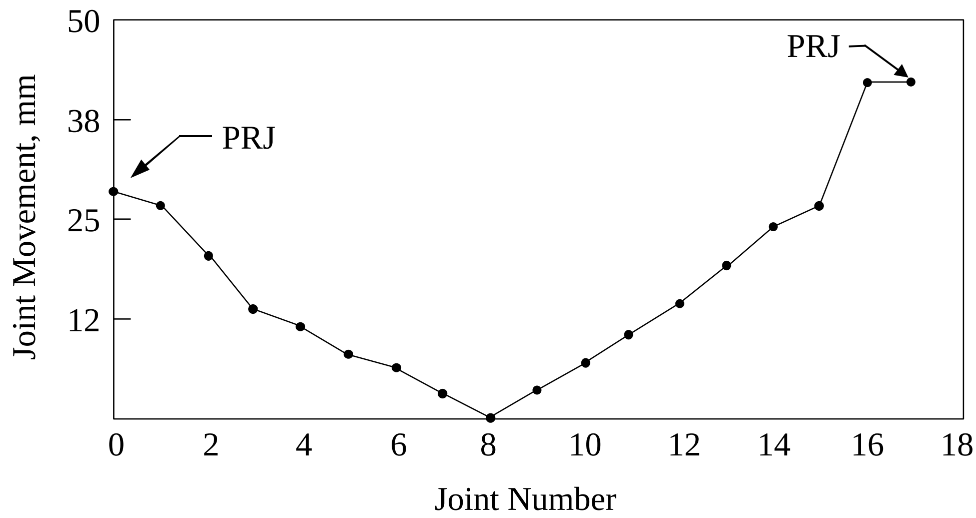


Figure 4-3.6. Illustration of movement of intermediate joints.[17]

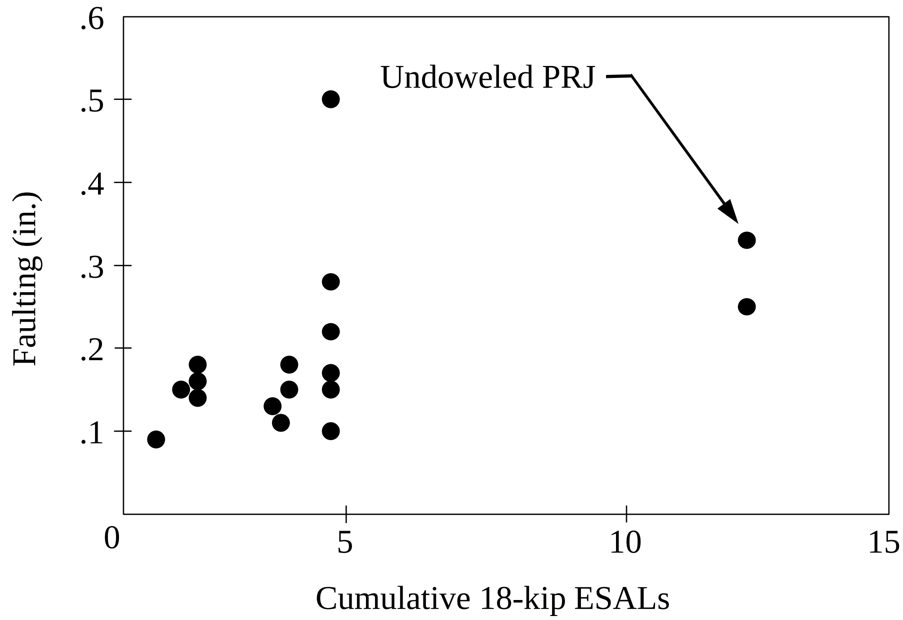
The effect of pressure relief on existing joint seals must also be considered in evaluating the need for pressure relief joints. Because pressure relief joints cause the adjacent contraction joints to open, sealant materials located in those adjacent joints could become overstressed and lose adhesion to the joint face. This is particularly true for compression seals. Once sealants fail, water and incompressibles can infiltrate the joint, inhibiting joint closure and creating additional compressive stress in the pavement. Therefore, prior to their installation, it must be confirmed that pressure relief joints will not impair the effectiveness of adjacent transverse joint sealants.

Finally, the presence of other pressure-relieving features must be considered when contemplating the installation of pressure relief joints. For example, bridge approach expansion joints will usually provide sufficient pressure relief within 150 m, and additional relief is not needed within that distance. Pavements that have sustained recent full-width blowups may not need pressure relief joints within 150 m of the blowups, especially if the blowups were patched with bituminous materials. Large joint openings in the area of a blowup indicate that further relief is not needed. In addition, if full-depth repairs have been recently constructed on the project, pressure relief joints are probably not needed, since the installation of the full-depth repairs serves to relieve any built-up pressure that may have been present in the pavement.[2,16]

5. LIMITATIONS AND EFFECTIVENESS

Since pressure relief joints are typically constructed without positive load transfer devices, poor load transfer is a major problem plaguing the performance of these joints. Because they are usually constructed with no load transfer, pressure relief joints should be used only on pavements that have exhibited severe blowups or bridge pushing problems.

Deflection testing from Ohio on several PRJ installations yielded an average load transfer of 51 percent for a 100 mm wide, nondoweled PRJ, compared to an average load transfer of 74 percent for a 25 mm wide, doweled PRJ.[2] Poor load transfer results in large corner deflections that can lead to pumping and faulting of the PRJ. Figure 4-3.7 shows the faulting of several in-service, nondoweled pressure relief joints as a function of 80-kN ESAL applications.[2] The faulting for most of these joints is approaching unacceptable values (say, greater than 3.8 mm).

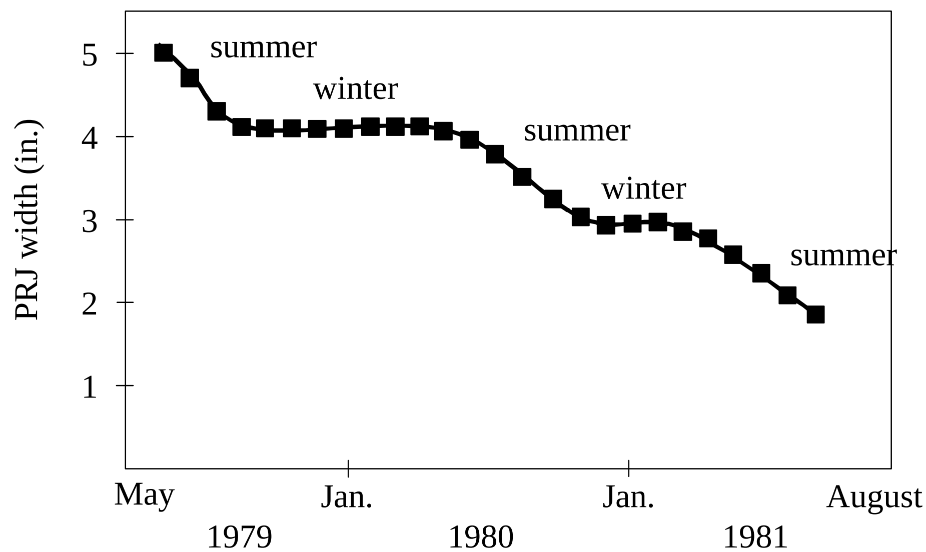


1 in= 25.4 mm
18 kip= 80 kN

Figure 4-3.7. Faulting of in-service, nondoweled pressure relief joints.[2]

Pressure relief joints may close completely over time, making the pavement again susceptible to blowups and bridge pushing. If intrusion of incompressibles into the joints is not stopped or if reactive aggregate expansion is progressing, the construction of pressure relief joints will provide only a temporary solution. When the joints have closed, new pressure relief joints will be required.

The rate of the closure of pressure relief joints depends upon the magnitude of the expansive pressures that exist in the pavement. Pressure relief joints installed in pavements experiencing appreciable expansive pressures will begin to close almost immediately. Figure 4-3.8 illustrates PRJ closure rates on jointed reinforced concrete pavements (JRCP) designs in Nebraska, where ASR was causing the expansion.[17] Most of the closure experienced during the first year occurred before the summer months, when unrelieved stresses would be highest.



1 in= 25.4 mm
18 kip= 80 kN

Figure 4-3.8. Pressure relief joint closure rates in Nebraska.[17]

As previously mentioned, the unwarranted use of pressure relief joints in nondoweled slabs will result in loss of load transfer in adjacent joints, further intrusion of incompressibles, widening of joints/cracks, slab faulting, and overall accelerated pavement deterioration.[2,16] And, as has been shown, deflections at the relief joint are high because no load transfer is provided. Thus, to reiterate, the construction of pressure relief joints is recommended only when major blowups or other significant pressure-related damage has occurred.

The use of asphalt concrete-filled relief joints and patches in expanding rigid pavements often results in "humping" of the asphalt patch as the rigid pavement expands into the patch area. Humping, loss of load transfer, adjacent slab rocking, and settlement or heaving of asphalt patch areas have resulted in rough pavements and loss of pavement serviceability. This design also permits substantial slab movement that may result in increased deterioration of the adjacent pavement. When HMA overlays are placed above these joints, reflection cracks appear and deteriorate rapidly.[2] For these reasons, the use of bituminous pressure relief patches is not recommended.

Finally, installation of pressure relief joints has been accompanied by accelerated pavement deterioration on some projects. For example, when pressure relief joints were installed in one direction of a divided, JRCP section in Michigan, a condition survey conducted 13 years later revealed that the side with pressure relief joints had deteriorated to a greater extent.[16] Also, a project in Louisiana showed that the pressure relief joints were effective in preventing blowups of JRCP, but did not reduce the amount of joint spalling.[16]

6. DESIGN CONSIDERATIONS

As the previous discussion has indicated, the performance of pressure relief joints has been variable. While they have worked well in some cases, they have often resulted in major problems in other instances. Described below are several key design and construction considerations that each agency should examine prior to installing pressure relief joints.

General Design Recommendations for Pressure Relief Joints

Work documented by Smith et al.[2] and Snyder et al.[16] present the findings of a detailed study conducted on the effectiveness of pressure relief joints. That study provided many recommendations regarding the use of pressure relief joints, including the following:

- Pressure relief joints should be used only on rigid pavements with a history of blowups, on PCC pavements containing reactive aggregate, or on rigid pavements that are to be overlaid that have demonstrated pressure buildup problems in the past.

- When appropriate, pressure relief joints should be installed approximately 305 m from one another and from other pressure relieving features (such as concurrently or recently placed full-depth repairs or expansion joints near secondary structures).

- Most new pressure relief joints should be limited to widths of 25 to 50 mm to reduce the possibility of over-relieving the pavement. A greater width promotes excessive widening of adjacent joints and cracks. However, pavements with reactive aggregates may require greater pressure relief joint widths.

- The continued use of pressure relief joints to protect bridges and other secondary structures is recommended for all pavement types.

- Existing drainage conditions and the need for drainage improvement should be considered in conjunction with pressure relief work. Improved drainage may improve pressure relief joint performance by reducing pumping and faulting.

The optimum time for construction of pressure relief joints is not known. Experience has shown that many rigid pavements exhibit blowups after 6 to 12 years of service.[6,18] However, many rigid pavements perform for over 30 years without experiencing blowups.

Studies by several agencies have concluded that pressure relief joints tend to relieve stresses for about 150 m on either side of the joint. See references 6, 17, 19, and 20. In most cases, pressure relief joints are typically installed at intervals of 150 to 300 m.[1] Because of the difficulty in sawing through dowels or other load transfer devices and the danger of encountering unstable subbase conditions near old joints, pressure relief joints are generally placed near mid-slab. When bridge pushing is the only problem, pressure relief joints should be located only near the bridge approach slabs.

7. PAVEMENT SURVEYS

Pressure relief joints are recommended for installation only in areas where compressive stresses in the slabs are known to have increased dramatically during warmer weather, or where blowups have occurred previously. Generally, only long-jointed pavements should be considered as candidates for the installation of pressure relief joints.

Pavement surveys should be conducted and signs of pressure-related damage (blowups, spalling, compression cracking, cracked bridge abutments) should be noted. The existing joint sealant should be inspected to determine its condition and to note the presence of incompressibles. The presence of ASR or D-cracking are signs that expansive pressures may exist in the pavement, necessitating the placement of pressure relief joints.

8. COST CONSIDERATIONS

The cost of pressure relief joints varies considerably, depending upon whether they are installed as part of a planned rehabilitation, placed by maintenance forces in response to the occurrence of blowups, or incorporated as part of a full-depth repair. The costs for the installation of a 100 mm wide pressure relief joint may range from $400 to $800 per joint, including labor, material, and equipment.

9. CONSTRUCTION

Construction

The pressure relief joint can be placed either in mid-slab or at the location of an existing transverse joint, with the mid-slab location preferred. Due to the pressure release which accompanies a blowup, a pressure relief joint will not be needed within the first 150 to 305 m on either side of the blowup.

Generally, the relief joint is created using either a carbide-tipped wheel saw or two cuts from a diamond-bladed saw. The wheel saw can cut the joint to the desired width (generally between 25 and 100 mm) in one pass. The diamond-bladed saw is used to make two cuts, with the area between the cuts removed with jackhammers. Diamond-bladed saws create a smoother joint than wheel saws, the latter of which have a tendency to spall the joint at the surface of the slab.

After the creation of the joint, the joint is cleaned and the compressible filler material is installed. The filler material is generally a compressible material, although preformed compression seals have also been used. These compressible filler materials are often coated with a lubricant-adhesive that facilitates the installation of the filler and holds it in place after installation. Special hydraulic equipment is often used to compress and install the filler. After installation, the filler should be capped with a sealant material to reduce the infiltration of water.

Pressure Relief Joints and Multi-Lane Pavements

Pressure relief joints are normally installed on pavements with more than one traffic lane, and it is sometimes impossible to install the joint across the full pavement width in one day. When relief is provided for one lane only, the other lane can be subjected to higher compressive stresses, and blowups or shearing of the longitudinal tiebars can result. Thus, it is necessary to install pressure relief joints in all adjoining lanes as soon as possible. If the joints are placed during seasons with small daily temperature variations, a maximum period of 48 hours between installation of relief joints in adjacent lanes should not cause a blow up in the adjacent lane.[(6)]

In cases where the unrelieved lane is of good-quality concrete, restraint between the lanes has been observed to prevent functioning of the relief joint so that the joint filler material is not held tightly in position and can float out during a heavy rain. This can be avoided by installing the joint full width within 48 hours, which is strongly recommended.

Hot Weather Installation

During the warmest time of the year, or in pavements with reactive aggregates, pressures in the pavement may be of sufficient force to pinch or bind the saw blades during the sawing operation. In addition, the problem of unequal pressure between adjacent lanes is often aggravated during warm weather. For this reason, installation within a temperature range of 4 ^{o}C to 21 ^{o}C is recommended. Some agencies install relief joints during the summer months by sawing at night or early in the morning to avoid high temperatures.

10. EQUIPMENT

Although exact construction procedures vary among agencies, two methods are generally used for the installation of narrow relief joints. When sawcuts are made with a diamond-bladed saw, the material between the cuts is removed using light jackhammers. The faces of the joint are cleaned and the filler material is installed. This must be done during periods of cool temperatures to prevent the saw from binding in the cut.

With carbide-tipped wheel saws, make only one cut to the prescribed width. After sawing, clean the faces as necessary, and then install the filler material. The use of these saws should be considered carefully, since they may produce a large amount of spalling along the pressure relief joint at the pavement surface.

11. SUMMARY

Expansive pressures in rigid pavements may build up over time due to the intrusion of incompressibles into poorly sealed joints, pumping of base materials into joints, the use of expansive or reactive aggregates during original construction, and extremely high temperatures and moisture conditions in the slab relative to the neutral (no stress) condition. If untreated, these pressures result in compressive forces in the rigid pavement that can cause joint spalling, pavement blowups, and damage to adjacent structures and appurtenances. The installation of pressure relief joints is one method of reducing the build-up of expansive pressures and preventing the development of these distresses.

Pressure relief joints are full-width, full-depth cuts in a rigid pavement slab. They are typically made near the center of the slab, are 25 to 50 mm wide, and are filled with a compressible material such as sponge rubber or styrofoam. These joints allow nearby slabs (within several hundred feet) to move toward the joint area, thereby relieving compressive stresses. Special, heavy-duty relief joints incorporated into full-depth repairs have also been used with success on heavily trafficked highways. The use of full-depth asphalt concrete patches as expansion joints is discouraged because of poor performance.

The use of pressure relief joints should be carefully considered and placed only on pavements exhibiting pressure-related distresses. The unwarranted use of pressure relief joints can cause significant problems to the existing pavement. For example, loss of load transfer, increased intermediate joint openings and deflections, patch failure, joint sealant failure, loss of serviceability, and increased roughness can occur if pressure relief joints are installed in pavements where excessive expansive pressures do not exist.

Where warranted, pressure relief joints should be limited to a width of 25 to 50 mm, and should generally be placed no less than 300 m apart. Special care must be taken when constructing pressure relief joints in hot weather or on multi-lane facilities. Construction during periods of moderate temperature variation will prevent many problems (e.g., bound or stuck saw equipment, stress transferral resulting in blowups in adjacent lanes, and so on). Placing pressure relief joints in nondoweled JPCP and CRCP is not recommended except where required near bridges to prevent shoving of abutments.

12. REFERENCES

1. McGhee, K.H. 1995, "Design, Construction, and Maintenance of PCC Pavement Joints," NCHRP Synthesis of Highway Practice 211, Transportation Research Board, Washington, DC.

2. Smith, K.D., M.B. Snyder, M. I. Darter, M. J. Reiter, and K. T. Hall. 1987, "Pressure Relief and Other Joint Rehabilitation Techniques," FHWA/RD-88/111. Federal Highway Administration, Washington, DC.

3. Gordinier, D.E. and W.P. Chamberlin. 1968, "Pressure Relief Joints for Rigid Pavements," Research Report 68-12, New York State Department of Transportation, Albany, NY.

4. Stott, J.P. and K.M. Brook. 1968, "Report on a Visit to U.S.A. to Study Blowups in Concrete Roads," Report LR128, Road Research Laboratory, Great Britain.

5. Shah, G.N. 1974, "Rigid Pavement Investigation—Growth Characteristics and Blowups," Maryland Department of Transportation, Baltimore, MD.

6. McGhee, K.H. 1977, "Effectiveness of Pressure Relief Joints in Reinforced Concrete Pavement," Transportation Research Record 632, Transportation Research Board, Washington, DC.

7. LaCoursiere, S.A., M.I. Darter, and S.A. Smiley. 1978, "Performance of Continuously Reinforced Concrete Pavements in Illinois," FHWA-IL-UI-172, Illinois Department of Transportation, Springfield, IL.

8. Stark, D. 1991, "Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures," SHRP-C/FR-91-101, Strategic Highway Research Program, Washington, DC.

9. Kerr, A.D. and P.J. Shade. 1984, "Analytical Approach to Concrete Pavement Blowups," Transportation Research Record 930, Transportation Research Board, Washington, DC.

10. American Association of State Highway Officials (AASHO). 1962, "The AASHO Road Test, Report 5— Pavement Research," Special Report 61E, National Academy of Sciences, National Research Council, Washington, DC.

11. Kerr, A.D. and W.A. Dallis, Jr. 1985, "Blowup of Concrete Pavements," Journal of Transportation Engineering, Volume 111, American Society of Civil Engineers, Washington, DC.

12. Kerr, A.D. 1993a, "The Assessment of Concrete Pavement Blowups—A User Manual," FHWA-RD-91-056, Federal Highway Administration, Washington, DC.

13. Kerr, A.D. 1993b, "Blowup of a Concrete Pavement Adjoining a Rigid Structure," FHWA-RD-90-111, Federal Highway Administration, Washington, DC.

14. Giffin, H.W. 1943, “Transverse Joints in the Design of Heavy Duty Concrete Pavements,” Proceedings, Highway Research Board Annual Meeting, Volume 23, Highway Research Board, Washington, DC.

15. Federal Highway Administration (FHWA). 1990, “Continuously Reinforced Concrete Pavement,” FHWA Technical Advisory T 5080.14, Federal Highway Administration, Washington, DC.

16. Snyder, M.B., K.D. Smith, and M.I. Darter. 1989, “An Evaluation of Pressure Relief Joint Installations,” Transportation Research Record 1215, Transportation Research Board, Washington, DC.

17. Ramsey, W.J., R.J. Wedner, and J.E. Anderson. 1982, “Concrete Pavement Joint Repair in Nebraska,” Report No. NE-DOR-R-82-2, Nebraska Department of Roads, Lincoln, NE.

18. Gress, D.L. 1977, “Blowups on Resurfaced Concrete Pavements,” Report No. JHRP-76-25, Indiana State Highway Commission, Indianapolis, IN.

19. Rasoulian, M. and C. Burnett. 1983, “Evaluation of Relief Joints—Brush Fire C-51, A Summary Report,” Louisiana Department of Transportation and Development, Baton Rouge, LA.

20. Simonsen, J.E. 1980, “Preventive Maintenance of Concrete Pavements—US 127,” Report No. R-1141, Michigan Department of Transportation, Lansing, MI.

MODULE 4-4

PARTIAL DEPTH REPAIRS

1. INSTRUCTIONAL OBJECTIVES

This module describes procedures for partial-depth repair of concrete pavements. The participant will be able to accomplish the following upon successful completion of this module:

1. Identify distress types that can be corrected with partial-depth repairs.
2. Be familiar with various materials used for partial-depth repairs.
3. Describe successful construction procedures for partial-depth repairs.

2. INTRODUCTION

Partial-depth repairs extend the life of rigid pavements by restoring ride quality to pavements that have spalled or that have distressed joints. Partial-depth repairs of spalled joint areas also restore a well-defined, uniform joint sealant reservoir prior to joint resealing. When properly placed with durable materials and combined with a good joint resealing program, these repairs can perform well for many years.

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 where 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 (e.g., when replacing an entire joint to address small spalls). 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.

Many aspects (e.g., materials, procedures, equipment, and long-term performance) of partial-depth repair have recently been studied nationwide and are reported elsewhere.[(1)] There is also a SHRP manual on concrete pavement rehabilitation that includes a chapter on partial-depth repair.[(2)]

3. DEFINITIONS

Spalls. Broken or chipped off areas of concrete that leave an uneven surface or edge. Spalls can occur at joints or in the interior portion of the slab; interior spalling is almost always caused by either high reinforcing steel or the use of inferior materials in the concrete.

Partial-depth repair. The placement of a patch on concrete pavements, usually to a depth of less than one-third of the slab thickness, that addresses surface defects and shallow joint spalling. Partial-depth repairs should not be used for deep spalls (greater than one-third of the slab thickness) or for spalls at working cracks. Distresses that extend more than one-third of the slab thickness or that occur at cracks are candidates for full-depth repairs (see module 4-5).

4. PURPOSE AND PROJECT SELECTION

Partial-depth repair is the removal of small, shallow areas of deteriorated concrete and replacement with a suitable repair material. It is important that the repair material be compatible in strength and volume stability with the concrete in the existing slab. Ideally, the repair material bonds to the sound concrete and becomes an integral part of the slab.

The need for partial-depth repair should be evaluated whenever joint resealing is planned for a project, as effective joint resealing requires repair of the adjacent distress. Partial-depth repair should also be considered when preparing a pavement to receive a hot-mix asphalt (HMA) or bonded portland cement concrete (PCC) overlay. Failure to repair spalled areas prior to placement of an overlay may contribute to the appearance of reflected distresses that break down rapidly, causing premature failure of the overlay. On comprehensive concrete pavement restoration projects, partial-depth repairs should be completed after any undersealing and/or slab jacking, but prior to diamond-grinding and joint sealing.

5. LIMITATIONS AND EFFECTIVENESS

Partial-depth repairs replace concrete only, and cannot accommodate the movements of working joints and cracks, load transfer devices, or reinforcing steel without experiencing high stresses and material damage. Thus, they are appropriate only for certain types of concrete pavement distresses that are confined to the top of the slab. Distresses that have been successfully corrected with partial-depth repairs include:

- Spalls caused by the use of joint inserts.
- Spalls caused by intrusion of incompressible materials into the joints (typically associated with long-jointed slabs).
- Spalls caused by localized areas of scaling, weak concrete, clay balls, or high steel.

Types of distresses that are not candidates for partial-depth repair include:

- Cracking and joint spalling caused by compressive stress buildup in long-jointed pavements.
- Spalling caused by dowel bar misalignment or lockup.
- Transverse or longitudinal cracking caused by improper joint construction techniques (late sawing, inadequate saw cut depth, or inadequate insert placement depth).
- Working transverse or longitudinal cracks caused by shrinkage, fatigue, or foundation movement.
- Spalls caused by D-cracking or reactive aggregate.

Joint spalling in concrete pavements may be caused by poor joint maintenance practices, the use of metal or plastic joint inserts during construction to initiate the formation of the joint, material durability problems, an inadequate concrete air void system, or improper placement of load transfer devices or tiebars. If the spalling distress caused by any of these factors is confined to the upper one-third of the slab, partial-depth patching may be the ideal solution.

Surface scaling or deterioration may be caused by reinforcing steel that is too close to the surface or by an inadequate air void system. Again, if limited to the upper one-third of the slab, partial-depth repairs can be used to address these problems.

The performance of partial-depth repairs has been excellent on many projects. Studies on one heavily traveled toll road where several thousand partial-depth repairs had been installed showed that over 80 percent of the repairs were in excellent condition after 5 years of service.[3] Inspection and quality control were very stringent on this job. A New York study comparing polymer concrete systems found that after 3 years of field testing, all of the repairs were performing satisfactorily and other polymer repairs were performing satisfactorily after 5 years with no signs of wear.[(4)] A comprehensive survey of partial-depth repair throughout the United States found a wide range of performance: many partial-depth repairs were performing well after 8 to 10 years of service, while others exhibited poor performance.[(5)]

On many projects where quality control and inspection have been less stringent, performance has been unsatisfactory. On some such projects, as many as 50 percent of the repairs failed in about 2 years.[(6)] The most frequent causes of failure include inappropriate use (when deterioration is too deep), lack of bond, compression failure of the patch (due to failure to re-establish the joint), variability in effectiveness of repair material, improper use of repair materials, insufficient consolidation, and incompatibility in thermal expansion between the repair material and the original slab.

One study comparing 11 repair materials and three hole preparation techniques concluded that the most important factor affecting performance was squaring of the hole, because few of the materials could be feathered and held in place.[(7)] A later study reviewing partial-depth repair construction and performance noted that most failures resulted from improper construction and placement techniques, not from material deficiencies.[(8)] In a study in Indiana in which partial-depth repairs were performed as part of a comprehensive CPR project, failures after 1 year were common.[(9)] However, it appears that these repairs failed due to adjacent crack or joint movement, and thus it is likely that the repair was not well isolated.

The largest and most comprehensive study of partial-depth repair performance is an ongoing evaluation of up to 10 materials and four preparation/application methods at four test sites around the country.[(10)] At the time of the most recent evaluation, the test sites were between 4 and 5 years old, and there were a number of materials and placement procedures that had yet to show any failures. Ultimately, it is expected that long-term monitoring of the remaining test sites and materials will provide important information about which materials and placement procedures are cost-effective, and under what conditions.

6. DESIGN CONSIDERATIONS

Repair Locations and Size

Partial-depth repairs, though generally placed along transverse joints, can be placed along longitudinal joints or anywhere in the slab. On any project where partial-depth repair is being considered, it is highly recommended that coring be performed at representative joints to determine the depth of the deterioration and assess the appropriateness of partial-depth repair.

If several spalls are present on one joint, it is usually more economical to repair the entire length of the joint than to repair individual spalls. Areas less than 150 mm long and 40 mm wide at the widest point are normally not repaired, but are filled with a sealant (unless a preformed compression seal is to be used in the joint). If a preformed compression seal is to be used, all spalls must be repaired to provide a uniformly-shaped reservoir and to prevent the seal from working out of the joint.

Repair Materials

A wide variety of materials is available for use in a partial-depth repair. Material selection depends on available curing time, ambient temperature, cost, and the size and depth of the repairs. 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 have been developed.(11,12)

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 can the patch be exposed 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 far greater than normal concrete, and when restrained can induce a tensile stress as high as 6,900 kPa.(13) The amount of drying shrinkage can be reduced by providing continuous moist curing, such as by covering with wet burlap. Differential expansion between the repair material and the surrounding concrete can also be detrimental. The problem is that limiting values for these properties cannot be established because a standard, comparable test method does not yet exist. Several studies have been or are currently being conducted to establish such limits based on performance.(12,14,15)

Another important property of the repair material is freeze-thaw durability. A recent study on the properties of repair materials found that the freeze-thaw durability of many materials is unacceptable, especially under severe exposure conditions.(12) Specimens were tested according to ASTM C-666-92, Procedure A, and were subjected to a maximum of 300 cycles of freezing and thawing. This procedure is recommended for testing repair materials before widespread use.

In general, the repair materials can be classified into three categories: cementitious, polymeric, and bituminous. Each category and their corresponding properties are described in the following sections.

Cementitious Materials

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

Portland Cement Concrete

High-quality PCC is generally accepted as the most compatible material for the repair of rigid pavements. Typical mixes combine Type I, II, or III portland cement with coarse aggregate not larger than one-half the minimum repair thickness (9.5 mm maximum size is often used). The mix should be a low-slump mixture of air entrained concrete having a water-cement ratio not exceeding 0.44. Either type III portland cement or the use of a set-accelerating admixture (e.g., 2 percent $CaCl_2$ by weight of cement) in a Type I mix is often specified if the concrete repair must be reopened to traffic quickly.

Type I portland cement, with or without admixtures, has been used for permanent repairs longer and more widely than most other materials because of its relatively low cost, availability, and ease of use. Rich mixtures (up to eight bags of cement, or 450 kg/m^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.

Gypsum-Based Concrete

Gypsum-based (calcium sulfate) repair materials (e.g., Duracal, Rockite) gain strength rapidly and can be used in any temperature above freezing. However, gypsum concrete does not appear to perform well when exposed to moisture and freezing weather.[16] Additionally, the presence of free sulfates in the typical gypsum mixture may promote steel corrosion in reinforced pavements.[1]

Magnesium Phosphate Concrete

Magnesium phosphate concretes (e.g., Set 45, Eucospeed MP, Propatch MP) 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 cause severe strength reduction). Furthermore, magnesium phosphate concrete is very sensitive to aggregate type (limestones are not acceptable).[1] In hot weather (i.e., above 32 ºC), commonly available mixes will set in less than 10 minutes, often providing insufficient time for proper placement. These limitations make it difficult to use and have led to variable field performance.[16,17]

High Alumina Concrete

Calcium aluminate cements (e.g., Five Star HP) 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.[5,16,18]

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 (large differences in the coefficient of thermal expansion can cause 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 in certain field conditions. Polymers used for pavement repairs can be classified into four categories: epoxies, methacrylates, polyester-styrenes, and urethanes.

Epoxy Concrete

Epoxy resins are impermeable and excellent adhesives. Available epoxy resins (e.g., Burke 88/LPL, Mark 103 Carbo-Poxy) have a wide range of setting times, application temperatures, strengths, and bonding conditions. The epoxy concrete mix design must 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. Deep epoxy repairs must frequently be placed in lifts to control heat development. Use of epoxy concrete to repair spalls caused by reinforcing steel corrosion may accelerate deterioration of unrepaired areas adjacent to the epoxy-concrete repair by creating a strongly cathodic area.[19]

Methyl Methacrylate Concrete

Methyl methacrylate (MMA) concretes and high molecular weight methacrylate (HMWM) concretes (e.g., SikaPronto 11, Degadur 510) have long working times, high compressive strengths, and good adhesion. Many methacrylates are volatile and may pose a health hazard to those exposed to the fumes for prolonged periods.[20]

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.[20]

Polyurethane Concrete

Polyurethane repair materials generally consist of a two-part polyurethane resin mixed with aggregate. 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.[6,20]

Bituminous Materials

Bituminous materials are widely used as a repair material for both flexible and rigid pavements. While bituminous materials are often perceived as temporary repair materials on rigid pavements, they often are left in place for many years. They have the advantage of being relatively low in cost, widely available, easy to place with small crews, easy to handle, and generally need little, if any, cure time. However, because the joint cannot be re-established when using bituminous mixtures, they are not recommended for permanent repairs. HMA patches are often used instead of PCC patches prior to overlaying, particularly when the existing rigid pavement is too D-cracked or otherwise deteriorated to permit full-depth repairs.

Bonding Agents

PCC repair 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 PCC 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.[17] 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. Many of the proprietary mixes, when they do require one, will specify what can be used.

Cost Considerations

Material costs, mechanical properties, workability, and performance vary greatly between the different repair materials. Table 4-4.1 describes typical properties for a few of the many repair materials available.[20,21] As observed from this table, the more rapid-setting materials are more expensive than the more traditional repair materials. Results from the FHWA's long-term monitoring of partial depth repairs suggest that the bituminous repair materials have the shortest life.[10] Their use may be appropriate, however, if the pavement is to be rehabilitated in less than 3 years. A cooperative effort between the U.S. Army Engineer Waterways Experiment Station and the U.S. Bureau of Reclamation conducted extensive laboratory testing to determine pertinent properties of eleven repair materials.[12] Further evaluation of the field performance of the repair materials is currently being conducted.

Table 4-4.1. Typical repair material properties.[1]

Product	**Category**	**Working time (min)**	**Temp range, °C**	**Time to traffic at 21 °C(hr)**	**Moisture sensitive**[1]	**Cost**[2] **($/m^3)**
Type III PCC	Portland Cement	20	>0	5 - 6	No	375
Duracal	Gypsum	20	>0	1.5	No	280[3]
Set 45	Magnesium Phosphate	10	0-38	1.5	Yes	1,300
Five Star HP	High Alumina	20	0- 38	1.5	No	1,100
MC 64	Epoxy	10	>14	2.0	Yes	8,500
SikaPronto 11	Modified Methacrylate	30	>2	1.5	Yes	5,900
Blend[4]	Polyester-Styrene	15		2.0	Yes	760
Percol FL	Polyurethane	1	-18	0.17 - 0.33	No[5]	3,550
UPM	Bituminous	N/A	N/A	immediate	No	185

[1]Substrate and aggregate must be dry.
[2]Includes the cost of bagged aggregate (materials extended to maximum degree recommended by manufacturer), bonding agent if required, and admixtures if required.
[3]Does not include the cost of bonding agent. Bonding agent is recommended if used in shallow repairs.
[4]Nonproprietary blend containing 12 percent polyester resin by weight of dry aggregate.
[5]Aggregate must be dry; manufacturer claims that substrate may be wet.

7. CONSTRUCTION CONSIDERATIONS

The steps for the construction of partial-depth repairs are presented in this section. A simplified overview of this process is shown in figure 4-4.1. Manuals that describe the construction procedures in great detail are also widely available.[11,18,22] The Federal Highway Administration also provides general specifications for partial-depth repairs, although they may require some revisions to reflect local conditions.[18]

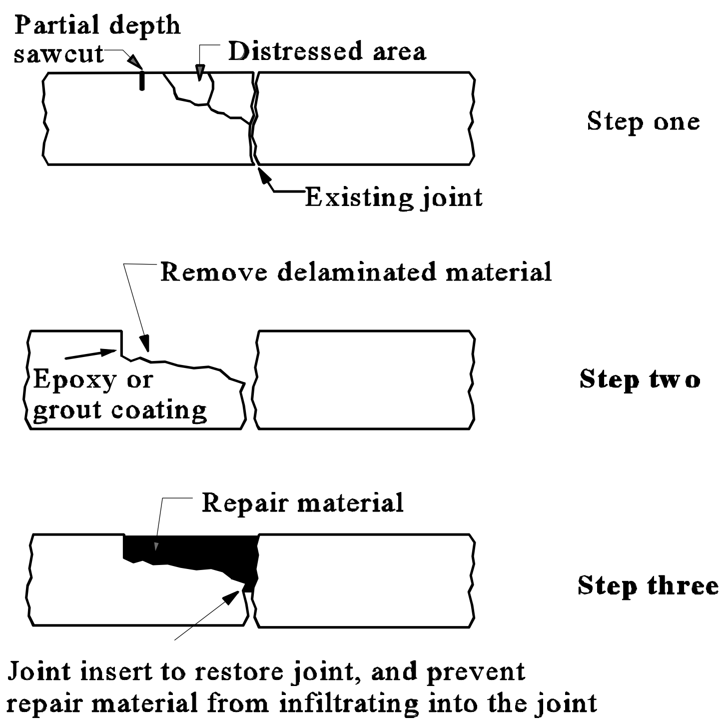


Figure 4-4.1. Placement of partial-depth repairs.

Location of Repair Boundaries

The actual extent of deterioration in a rigid pavement may be greater than 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. Often the extent of deterioration can be determined by “sounding” the concrete with a solid steel rod, chains, or a ball peen hammer. Areas yielding a clear ringing sound are judged to be acceptable, while those emitting a dull sound are considered weak. Sophisticated sounding equipment (such as the DelamTech) is also commercially available for the identification of deteriorated areas.

All weak and deteriorated concrete must be located and removed if the repair operation is to be effective. Normally, the area marked for removal is located 50 to 150 mm outside the defective area. A minimum repair length of 250 mm and width of 100 mm is recommended.

Removal of Deteriorated Concrete

Partial-depth removal of the deteriorated concrete is accomplished by several methods. The most frequently used method employs a diamond-bladed saw to outline the repair boundaries and light jackhammers to remove the concrete inside the repair area. Other methods include jackhammering solely, cold milling, or waterblasting.

If a saw is used, the saw cut should be 50 mm deep (figure 4-4.1). The cut boundary should be straight and vertical to provide a vertical face and square corners. Cutting repair boundaries with jackhammers results in "scalloped" boundaries into which repair materials must be "feathered." Vertical boundaries reduce the spalling associated with thin or feathered concrete along the repair perimeter.

After sawing, removal of unsound concrete is usually accomplished with jackhammers. The initial breakup can be done with hammers weighing up to 13.6 kg. 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 9.0 kg) 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.

For large repairs, the pavement marked for removal may be sawed in a shallow criss-cross or waffle pattern to facilitate concrete removal. A few States have successfully used carbide-tipped milling machines to remove these larger repair areas.[23] Standard milling machines with cutting heads of 300 to 450 mm have proven efficient and economical, particularly when used for larger areas, such as for full-lane-width repairs. The milling operation leaves a dish-shaped cavity that may be made vertical by additional jackhammering or sawing. This removal method produces a very rough irregular surface that promotes a high degree of mechanical interlock between the repair material and the existing slab.

Several States have also tried waterblasting, the use of a high pressure water jet to remove the deteriorated concrete[24]. The waterblasting equipment should be controlled by a mobile robot and be capable of producing a stream of water at a minimum of 100,000 to 200,000 KPa. The operating variables, such as speed and pressure, should be set to remove only the unsound concrete. Repair surfaces produced by waterblasting are rough and irregular and promote a high degree of mechanical interlock between the repair material and the existing slab. However, it is very important to remove the debris and slurry caused by this process immediately before the slurry sets. If this is not done, extensive cleaning operations will be needed to remove it from the existing concrete.

The typical depth of concrete removal varies from 25 to 100 mm. Partial-depth repairs should be limited to the top one-third of the slab and should not extend to a depth that allows dowel bars or reinforcing steel in the slab to bear directly on the repair material. If sound concrete cannot be reached (e.g., the area is unsound through the depth of the slab or unsound material cannot be removed because of reinforcing or load transfer devices), a full-depth repair is required. Small areas of full-depth repair have been combined with partial-depth repairs, but these generally do not perform as well as regular full-depth repairs.

After removal and cleaning, the bottom of the repair area is checked by "sounding" to ensure that all deteriorated concrete has been removed. Any remaining areas of unsound concrete must be removed.

Joint Preparation

The most frequent cause of failure of partial-depth repairs at joints is excessive compressive stresses. 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 (figure 4-4.2) will reduce the risk of such failures. 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 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 below and 75 mm beyond the repair boundaries. The joints must later be sawed and sealed.

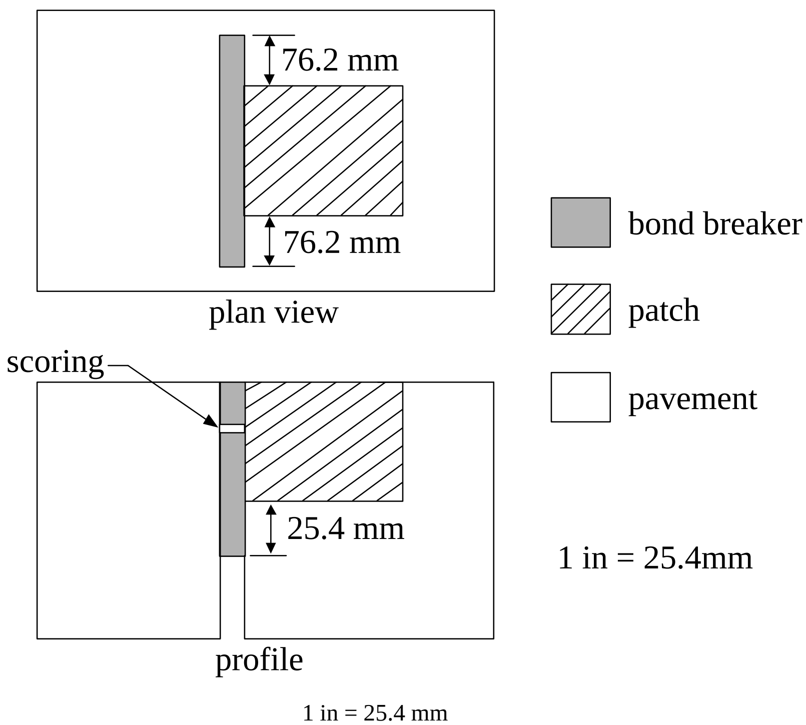


Figure 4-4.2. Compressible insert placement.

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.

Where spalling has been caused by a metal insert, the spalls usually start at the bottom fin of the insert (about 60 mm below the surface). When repairing this type of spall, it is recommended that the insert be sawed out along the entire length of the joint to avoid further deterioration. The joint can then be repaired and resealed.

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" cements may have sufficient compressibility to accommodate joint movements without the need for a compressible insert. Consult the manufacturers of these products for appropriate joint treatment.

Cleaning the Repair Area

Following removal of the concrete, the surface of the repair area must be cleaned. Dry sweeping, sandblasting, and compressed airblasting are normally sufficient to provide a clean, irregular surface for the development of a good bond between the repair material and the existing slab. Sandblasting is highly recommended for cleaning the surface. Sandblasting removes 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.

Application of Bonding Agent

Portland Cement Concrete Repair Materials

After the surface of the existing concrete has been cleaned, and just prior to placement of the repair material, it should 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, follow the manufacturer's directions 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; 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.

Repair Material Mixing

The volume of material required for a partial-depth repair is usually small (0.014 to 0.056 m^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 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 °C. Additional precautions, such as the use of warm water, insulating covers, and longer cure times, may be required at temperatures below 13 °C. 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 (McGhee 1981).

Some epoxy concretes may require that the material be placed in lifts not exceeding 50 mm 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 °C 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 releases trapped air from the fresh mix and contributes 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 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.

The repair surface may be hand-troweled to remove any remaining minor irregularities. The edge of a repair located adjacent to a transverse joint should be tooled to provide a good reservoir for joint sealant. Excess mortar from troweling can be used to fill any saw cuts extending into the adjacent pavement at repair corners.

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.

Curing

Curing is as important for partial-depth repairs as it is for full-depth repairs. Since partial-depth repairs often have large surface areas in relation to their volumes, moisture can be lost quickly. Inadequate attention to curing can result in the development of shrinkage cracks that may cause the repair to fail prematurely.

Portland Cement Concrete Repair Materials

All of the standard curing methods used for full-depth repairs may be considered for partial-depth repairs as well. The most effective curing procedure in hot weather 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. Moist burlap and polyethylene can also be used, but they must be removed when the roadway is opened to traffic. In cold weather, insulating blankets or tarps can be used to provide more rapid curing and earlier opening to traffic. The required curing method should be stated in the project plans and specifications.

In lieu of specifying the curing method and time, some agencies prefer to specify a strength requirement that must be attained before the repair can be opened to traffic. Discussion on the strength requirements is provided in subsequent sections.

Proprietary Repair Materials

Epoxy and proprietary repair materials should be cured as recommended by their manufacturers.

Joint Sealing

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 module 4-2.

Opening to Traffic

It is important that the partial-depth repair attain sufficient strength before it is opened to traffic. Generally, compressive strengths of 20.7 MPa are required by most agencies before the partial-depth repair is opened to traffic. However, in order to minimize lane closures, traffic loadings are often allowed on a repaired area when the repair concrete has attained the minimum strength needed to assure its structural integrity. Factors that favor the lowering of compressive strength requirements for the release of partial-depth repair to traffic are the lateral confinement and shallow depth of such repairs.

8. SUMMARY

Partial-depth repairs can be used to address certain types of distresses that affect only the upper one-third of the slab, such as joint spalls and localized areas of scaling. Inappropriate use on projects where full-depth deterioration exists has caused many problems on past projects. Cementitious, polymeric, and bituminous patching materials have all been used, with varying degrees of success.

All weak concrete must be located and removed for partial-depth repairs to be effective. These areas can be located by sounding the concrete with ball peen hammers or chains or by using sophisticated sounding equipment. Vertical saw cuts 25 to 50 mm deep then should be made beyond the boundary of the unsound area to be removed. Removal of the concrete is often done with jackhammers (up to 15 kg), cold milling machines, waterblasting machines, and diamond blade grinding machines. If the deterioration exceeds one-third of the slab thickness, a full-depth repair should be placed.

Following concrete removal, the repair area must be thoroughly cleaned by sandblasting, dry sweeping, and airblasting to provide a clean, dry, irregular surface to which the repair material can bond. If required, a bonding agent is then applied to the bottom and sides of the repair area.

Consolidation and curing considerations for partial-depth repairs are similar to those for full-depth repair operations. It is extremely important that joints be re-established in all joint or crack locations to prevent premature failure of the repair due to differential movement or compression failure. Bondbreaker and/or forming materials are used to minimize the effects of curling stresses and compressive stresses, and to prevent spalling of the repair area during curing due to interlock with adjacent materials. Joints must be formed in any repair placed across a joint or crack.

The type of repair material selected depends upon factors such as available curing time, ambient temperature, size, depth, and number of repairs, and thermal expansion compatibility with the existing pavement. PCC is the most widely used repair material (with or without set accelerators), but several other materials are also available. As an example, Caltrans has a list of approved materials for rapid repairs that addresses the needs of that agency.[25] That list, and some of the appropriate characteristics of the materials, is summarized in table 4-4.2. In any project, the cost-effective selection of repair materials should take into consideration the in-place repair costs and the desired performance life of the repair. Very effective repairs can be placed with inexpensive materials on pavements that are scheduled for major rehabilitation or reconstruction within several years.

Table 4-4.2. Selected characteristics of Caltrans rapid set repair materials.[25]

Material Name	Material Type	Max. % Pea Gravel	Extended $/m^3 1994	Apply to Damp Surface	Final Set Time @ 21°C, minutes	Flexural Strength, MPa	
						24-hr	28-day
American Highway Patch II	High Alumina	50	812	No	35	4.14	5.52
Burke 928 Fast Patch	High Alumina	80	1,059	Yes	30	3.79	5.17
Dayton Superior HD-50	High Alumina	0	---	Yes	25	3.48	8.96
Patchroc 10-60	High Alumina	60	953	Yes	30	4.14	6.21
L&M Durapatch Highway	High Alumina	50	1,165	Yes	25	3.79	5.52
Set 45 Caltrans	Magnesium Phosphate	60	1,271	No	35	3.79	4.48
Polyester Concrete	Polymer	n/a	706	No	45	13.8	13.8
5 Star Products Highway Patch	High Alumina	60	741	Yes	50	5.52	5.52
Rapid Set DOT Repair Mix	Hydraulic Cement	100	671	Yes	105	5.17	5.17

9. REFERENCES

1. Good-Mojab, C.A., A.J. Patel, and A.R. Romine. 1993, “Innovative Materials Development and Testing,” Volume 5—Partial Depth Spall Repair, SHRP-H-356, Strategic Highway Research Program, Washington, DC.

2. Yu, T., D. Peshkin, K. Smith, M. Darter, D. Whiting, and H. Delaney. 1994, “Concrete Rehabilitation Users Manual,” SHRP-C-412, Strategic Highway Research Program, Washington, DC.

3. McGhee, K.H. 1981, “Patching Jointed Concrete Pavements,” Transportation Research Record 800, Transportation Research Board, Washington, DC.

4. Webster, R., J. Fontana, and L. Kukacka. 1978, “Rapid Patching of Concrete Using Polymer Concrete,” Brookhaven National Laboratory, Upton, NY.

5. Snyder, M.B., M. J. Reiter, K.T. Hall, and M.I. Darter. 1989, “Rehabilitation of Concrete Pavements, Volume I—Repair Rehabilitation Techniques,” FHWA-RD-88-071, Federal Highway Administration, Washington, DC.

6. Mueller, P., J. Zaniewski, and S. Tritsch. 1988, “Concrete Pavement Spall Repair,” Annual Meeting of the Transportation Research Board, Transportation Research Board, Washington, DC.

7. Hartvigas, L. 1979, “Patching Flexible and Rigid Pavements,” FHWA/NY/RR-79/74, New York Department of Transportation, Albany, NY.

8. Wyant, D. 1984, “Evaluation of Concrete Patching Materials,” Virginia Highway and Transportation Research Council, Charlottesville, VA.

9. Jiang, Y., and R. R. McDaniel. 1993, “Evaluation of the Impact of Concrete Pavement Restoration Techniques on Pavement Performance,” Proceedings of the Fifth International Conference on Concrete Pavement Design and Rehabilitation, Volume I, Purdue University, West LaFayette, IN.

10. Romine, A.R., L.D. Evans, K.L. Smith, T.P. Wilson, and C. Wienrank. 1996, “Long-Term Monitoring of Pavement Maintenance Test Sites, Interim Report of Data Analysis,” Federal Highway Administration, Washington, DC.

11. Patel, A.J., C.G. Mojab, and A.R. Romine. 1993, “Concrete Pavement Repair Manuals of Practice: Materials and Procedures for Rapid Repair of Partial-Depth Spalls in Concrete Pavements,” Strategic Highway Research Program Report SHRP-H-349, National Research Council, Washington, DC.

12. Smoak, W.G., T.B. Husbands, and J.E. McDonald. 1997, “Results of Laboratory Tests on Materials for Thin Repair of Concrete Surfaces,” REMR-CS-52, U.S. Department of the Interior and U.S. Army Corps of Engineers.

13. Emmons, P.H., A.M. Vaysburd, and J.E. McDonald. 1993, “A Rational Approach to Durable Concrete Repairs,” Concrete International, American Concrete Institute, Farmington Hills, MI.

14. Lavers, G.R. 1991, “A New Standard for Shrinkage,” Highways and Transportation.

15. Stefanyk, D.W. 1992, “Evaluation of Length Change of Concrete Patching Materials to AT&U Specification B-391,” ABTR/RD/RR-92/09, Alberta Transportation and Utilities, Alberta, Canada.

16. National Cooperative Highway Research Program (NCHRP). 1977, “Rapid Setting Materials for Patching of Concrete,” NCHRP Synthesis of Highway Practice No. 45, Transportation Research Board, Washington, DC.

17. Tyson, S. 1977, “Partial Depth Repairs of Jointed PCC Pavements: Cast-in-Place and Precast Procedures,” Virginia Highway and Transportation Research Council, Charlottesville, VA.

18. Federal Highway Administration (FHWA). 1985, "FHWA Pavement Rehabilitation Manual," FHWA-ED-88-025. Federal Highway Administration, Washington, DC. (Manual supplemented April 1986, July 1987, March 1988, February 1989, October 1990).

19. Furr, H. 1984, "Highway Uses of Epoxy with Concrete," NCHRP Synthesis of Highway Practice 109, Transportation Research Board, Washington, DC.

20. Krauss, P. 1985, "New Materials and Techniques for the Rehabilitation of Portland Cement Concrete," California Department of Transportation, Office of Transportation Laboratory, Sacramento, CA.

21. Tempe, M.R. Ballou, D. Fowler, and A. Meyer. 1984, "Implementation Manual for the Use of Rapid Setting Concrete," Center for Transportation Research, University of Texas at Austin.

22. American Concrete Pavement Association (ACPA). 1989, "Guidelines for Partial-Depth Repair," Technical Bulletin TB-003P. American Concrete Pavement Association, Arlington Heights, IL.

23. Zoller, T., J. Williams, and D. Frentress. 1989, "Pavement Rehabilitation in an Urban Environment: Minnesota Repair Standards Rehabilitate Twin Cities Freeways," Proceedings of the Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West LaFayette, IN.

24. Kim, T., K.W. Lee, G. Veyera, W. Mogawer, and J. Zeng. 1989, "Utilization of a Waterjet Cutting Unit in Infrastructure Management," University of Rhode Island.

25. Jerzak, H. 1994, "Rapid Set Materials for Repairs to Portland Cement Concrete Pavement and Structures," California Department of Transportation, Sacramento, CA.

MODULE 4-5

FULL-DEPTH REPAIRS

1. INSTRUCTIONAL OBJECTIVES

This module describes procedures for cast-in-place portland cement concrete (PCC) full-depth repair (FDR) of rigid pavements. The repair techniques for both jointed (plain or reinforced) concrete pavements (JCP) and continuously reinforced concrete pavements (CRCP) are discussed. The use of bituminous full-depth patches are not recommended for permanent repair of rigid pavements.

Upon successful completion of this module, the participant will be able to accomplish the following objectives:

1. List the alternative types of joint repairs and load transfer methods available and the advantages and disadvantages of each.
2. Based on visual observations (and deflection testing), identify areas requiring full-depth repair and determine appropriate boundaries.
3. Determine when large areas should be completely removed and replaced rather than removing and replacing several adjacent smaller areas.
4. Select acceptable design and construction procedures for cast-in-place repairs at joints and cracks for typical pavement conditions.
5. List specific procedures and materials to achieve early opening of full-depth repairs to traffic.
6. Describe the design of a permanent, cast-in-place PCC repair that provides continuity of reinforcement and load transfer at transverse repair joints for CRCP.

2. INTRODUCTION

Concrete pavements exhibiting linear cracking, corner breaks, or certain joint-related distresses may be candidates for the placement of full-depth repairs. Such repairs are placed only at the areas exhibiting the distress and can be an effective means of restoring the rideability and structural integrity of existing concrete pavements in order to extend their service lives. Full-depth repairs are also effective in preparing a distressed rigid pavement to receive a structural overlay.

There are several key elements in the effective installation of full-depth repairs. This module provides information on those elements in the design and construction of full-depth PCC repairs in existing rigid pavements. Full-depth repair design and construction procedures for both jointed and continuously reinforced concrete pavements are presented. Various alternative methods for performing the different repair tasks are described where appropriate.

3. DEFINITIONS

As previously mentioned, full-depth repairs are placed at deteriorated joints and cracks in rigid pavements to restore the rideability of the pavement, to prevent further deterioration of distressed areas, or to prepare the pavement to receive an overlay. They are cast-in-place PCC repairs that extend the full-depth of the existing slab. Typically, full-depth repairs are a minimum of 1.8 m long and a full lane width wide, although sometimes it is more cost-effective to replace an entire slab than to place a series of shorter full-depth repairs.

4. PURPOSE AND PROJECT SELECTION

Full-depth repairs are an effective restoration activity for existing rigid pavements. Table 4-5.1 and table 4-5.2 list the distresses that can be successfully remediated through the use of full-depth repairs for jointed and continuously reinforced concrete pavements, respectively. The following are distress types commonly addressed with full-depth repairs:

- Transverse cracks **S** transverse cracks of medium and high severity are recommended for full-depth repairs. For both JCP and CRCP, such cracks are “working” cracks that are experiencing movement and may be exhibiting spalling, pumping, and faulting. Working cracks in CRCP are probably also an indication that the reinforcing steel has ruptured.

- Longitudinal cracks **S** longitudinal cracks of medium and high severity warrant full-depth repair. Such cracks typically have opened and may be exhibiting spalling or faulting.

- Blowup **S** blowups in JCP and CRCP of any severity warrant full-depth repair due to the localized disruption to pavement integrity and the potential safety hazard.

- Joint spalling **S** joint spalls of medium or high severity are recommended for full-depth repairs unless it can be determined that the deterioration is limited to the upper one-third of the slab. If the distress is limited to the upper portion of the slab, partial-depth PCC repairs are a feasible alternative. Some joint spalling may be the result of concrete durability distresses, such as D-cracking or reactive aggregate, and coring on either side of the joint may be necessary to determine the extent of subsurface deterioration.

- Punchouts **S** punchouts in CRCP are candidates for full-depth repair as they represent a structural failure of the pavement. Even low severity punchouts should be repaired as they will otherwise deteriorate rapidly under additional traffic loadings.

In determining the need for full-depth repair, consideration must be given to the extent of distress within a project. Concrete pavements in which deterioration is limited to the joints or cracks are good candidates for the application of full-depth repair techniques, provided that the deterioration is not widespread over the entire project length. Rigid pavements exhibiting severe structural distresses throughout the entire length of the project are more suited to appropriate structural overlay strategies (e.g., hot-mix asphalt [HMA] overlay of cracked/seated rigid pavement or unbonded PCC overlay) or total reconstruction. Where the damage is not quite as severe, retrofitted load transfer should be considered as an alternative.

Table 4-5.1. Candidate JCP distresses addressed by full-depth repairs.

DISTRESS TYPE	SEVERITY LEVEL REQUIRED FOR FULL-DEPTH REPAIR
Blowup	L, M, H
Corner Break	L, M, H
D-Cracking (at joints or cracks)	M[1], H
Deterioration Adjacent to Existing Repair	M[1], H
Deterioration of Existing Repairs	M[1], H
Longitudinal Cracking	M, H
Spalling of Joints	M[1], H
Transverse Cracking	M, H
Reactive Aggregate Spalling	M[1], H

[1] These distress types may only require partial-depth repair if they are limited to the upper one-third of the pavement slab.

NOTE: Traffic level will affect repair requirements. For example, highways with low traffic volumes may not require repair at the recommended severity level.

Table 4-5.2. Candidate CRCP distresses addressed by full-depth repairs.

DISTRESS TYPE	SEVERITY LEVEL REQUIRED FOR FULL-DEPTH REPAIR
Blowup	L, M, H
Construction Joint Distress	M, H
D-Cracking (at cracks)	H
Deterioration Adjacent to Existing Repair	M[1], H
Deterioration of Existing Repair	M[1], H
Localized Distress	M[1], H
Longitudinal Cracking	M, H
Punchout	L, M, H
Transverse Cracking	M, H *(wherever steel has ruptured)*

[1] These distress types may only require partial-depth repair if they are limited to the upper one-third of the pavement slab.

NOTE: Traffic level will affect repair requirements. For example, highways with low traffic volumes may not require repair at the recommended severity level.

Full-depth repairs typically represent a large cost item in a rigid pavement rehabilitation project. Furthermore, extended periods between the original field survey and the actual rehabilitation construction may lead to an increase in repair quantities. Due to the high cost of full-depth repairs, the lack of adequate funds, and an increase in repair quantities, some agencies may not repair distressed areas that should be addressed during the rehabilitation. This results in either continued deterioration of the distressed area, or, if an overlay is placed, premature failure of the overlay. A final survey of the project is recommended before rehabilitation construction in order to accurately identify distressed areas.

5. LIMITATIONS AND EFFECTIVENESS

While full-depth P.C. repairs can be designed and constructed to provide good long-term performance (10 or more years), the performance of full-depth PCC repairs on many inservice pavements has been inconsistent.(1,2) The major causes of premature failures of full-depth repairs have been inadequate design (particularly poor load transfer design), and poor construction quality. In addition, the effectiveness of some full-depth repair installations has been limited due to their placement 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.(3) 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.(3) Furthermore, the poor performance of the repairs was traced to "socketing" of the dowel bars placed in the repair, a condition in which oval gaps develop around the dowel bars. The dowel bars thus become very loose in the socketed hole and become largely ineffective in transferring load from one side of the joint to the next. The development of the socketing was attributed to inadequate grouting of the dowel bars and, in some cases, to the application of excess graphite on the dowel bars.(3) Illinois also experienced failure of full-depth repairs, but this was attributed to failure of the epoxy to retain the dowels.

Thus, the effectiveness of full-depth repairs appears to be strongly dependent upon 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. The overall condition of the pavement and the extent of deterioration should be carefully examined to ensure that the placement of full-depth repairs will perform as intended.

6. DESIGN CONSIDERATIONS

Selection of Repair Locations and Boundaries

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 condition survey should be performed immediately prior to construction to verify the quantity of repair work needed, since it is expected that additional pavement deterioration may have occurred since the previous pavement inspection.

<u>Jointed Concrete Pavements</u>

Jointed concrete pavements typically require far more repairs at joints than between joints. However, some pavements develop intermediate cracks that deteriorate under repeated heavy traffic loadings. Locking of the doweled joints will accelerate this crack deterioration by forcing open the intermediate cracks. 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.

The types of JCP distresses that can be successfully addressed through full-depth repairs are described in table 4-5.1. Each agency should examine these recommendations and modify them as needed to develop a table that more closely represents local conditions.

Sizing the Repair

After the areas needing repair are identified, the boundaries of each repair must be determined. This is typically performed by the project engineer at or just before the time of construction. Repair dimensions can play a major role in repair performance. It is important that the repair boundaries be selected so that all of the significant deterioration in the slab and underlying layers (including the subgrade) is removed. The information on the extent of deterioration beneath the slab surface may be obtained by conducting coring and deflection studies. The deterioration at the bottom may extend as much as 1 m or more beyond the visible boundaries of deterioration at the surface (see figure 4-5.1).

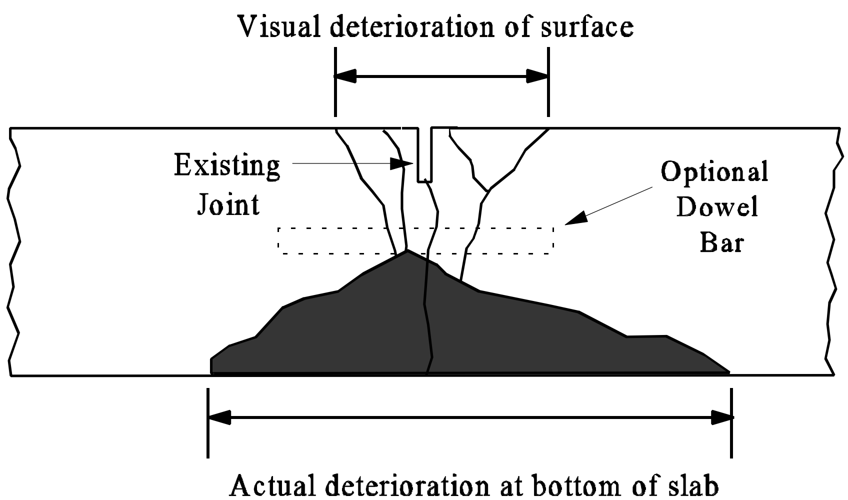


Figure 4-5.1. Illustration of potential extent of deterioration beneath a joint.

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

- <u>Doweled or Tied Repair</u>. A minimum length of 1.8 m and a full-lane-width repair are recommended to minimize rocking, pumping, and breakup of the slab.[1,2]

- <u>Nondoweled or Nontied Repair</u>. The minimum recommended repair lengths are 1.8 m for pavements with low truck traffic volumes and 2.4 to 3.0 m for pavements with medium to high traffic volumes.

Partial-lane-width repairs are generally not recommended due to their relative instability.[2]

Engineering judgment is required in selecting repair boundaries, particularly in areas exhibiting several types of distresses. Guidelines on developing repair boundaries for JCP are provided below:

1. Long repairs have a tendency to crack at midslab; therefore, repairs longer than 3 to 4 m should be constructed with either an intermediate joint to prevent cracking, or steel reinforcement to hold the cracks tight, should they occur.[1,4]

2. The repair boundary should not be too close to an existing transverse crack or joint; otherwise, adjacent slab distress will occur. A minimum distance of 1.8 m is recommended from the full-depth repair joint to the nearest transverse crack or joint.[1,5]

3. A boundary that would fall at an existing doweled transverse joint (distress at one side of the joint only) should be extended 0.3 m to include the existing joint. Attempts at salvaging the existing dowel system, even if the dowels are properly aligned and corrosion-free, frequently result in damage to dowel bars and the adjacent slab during the concrete breakup and cleanout operations.[5,6] If distress is present on only one side of an existing nondoweled joint, that joint may be used as a boundary.

4. Cracks located 3 m or farther from the joint can be repaired individually or, if severe enough, the entire slab can be replaced.

Figure 4-5.2 and figure 4-5.3 illustrate these construction recommendations.

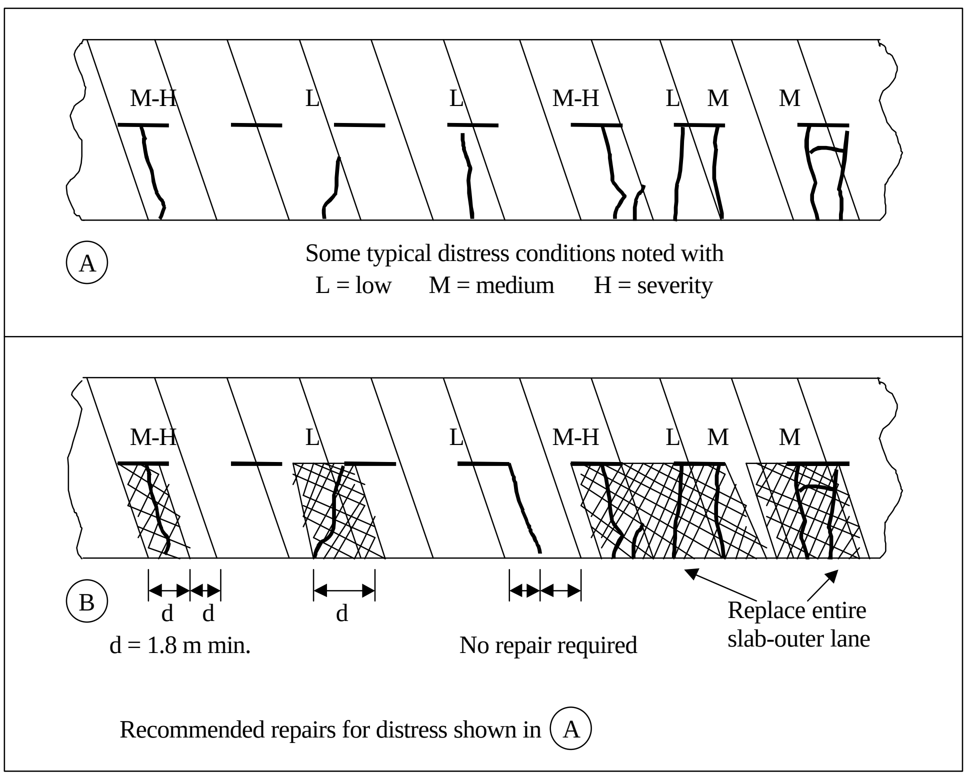


Figure 4-5.2. Repair recommendations for jointed plain concrete pavements (JPCP).

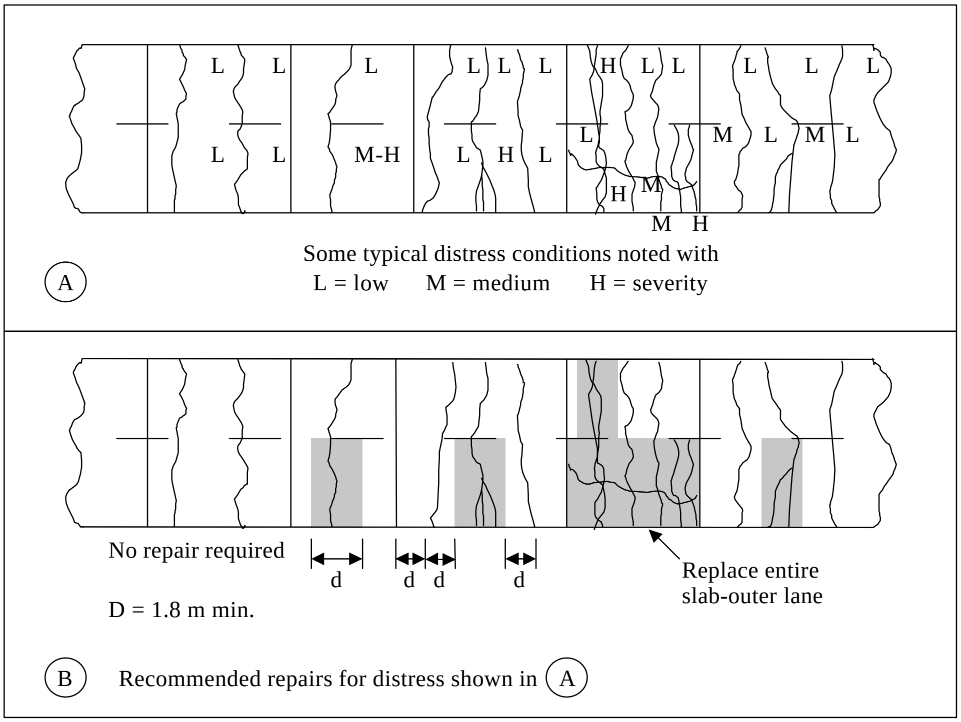


Figure 4-5.3. Repair recommendations for jointed reinforced concrete pavements (JRCP).

Sawcutting

Two types of sawed transverse joints—rough-faced and smooth-faced—have been used for full-depth repairs. Each type is described as follows:

1. Rough-Faced. Less than 30 percent of the slab depth is sawed, and a jackhammer is used to break up the deteriorated slab for removal. This produces a rough face that provides some degree of aggregate interlock (see figure 4-5.4a).

2. Smooth-Faced. The joint is sawed full-depth, resulting in a smooth face with no potential for aggregate interlock (figure 4-5.4b).

Generally, the use of smooth-faced joints is recommended, since they can be easily created and require less hand removal. In addition, rough-faced joints have the potential of spalling beneath the slab during removal, which can be eliminated by sawing full depth. Dowel bars are recommended, especially when smooth-faced joints are established.

Reinforcing steel in JRCP creates some additional concerns for full-depth repairs. Smooth-faced joints in which the slab is cut full depth are recommended to expedite removal of the concrete. There is no need to leave the reinforcing steel exposed because the repair does 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 3 m wide and can be expected to crack.

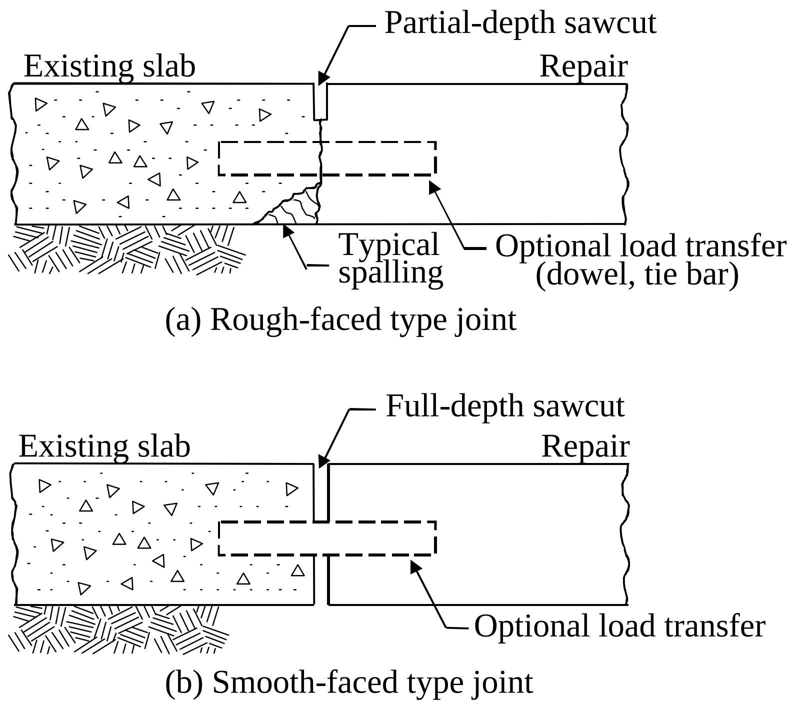


Figure 4-5.4. Types of sawed transverse joints: (a) rough-faced (b) smooth-faced.

Continuously Reinforced Concrete Pavements

Most full-depth repairs on CRCP will be placed at areas exhibiting punchouts and other localized distresses. Full-depth repairs may also be required at medium- and high-severity transverse cracks in which the steel has ruptured. The types of CRCP distresses that can be addressed through full-depth repairs were listed in table 4-5.2. Again, these recommendations should be evaluated by highway agencies and modified for use under their local conditions.

Sizing the Repair

As illustrated in figure 4-5.5, 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. An example measure of deflection near a punchout is shown in figure 4-5.6.[7] The area of deteriorated support for the CRCP is indicated by high deflections.

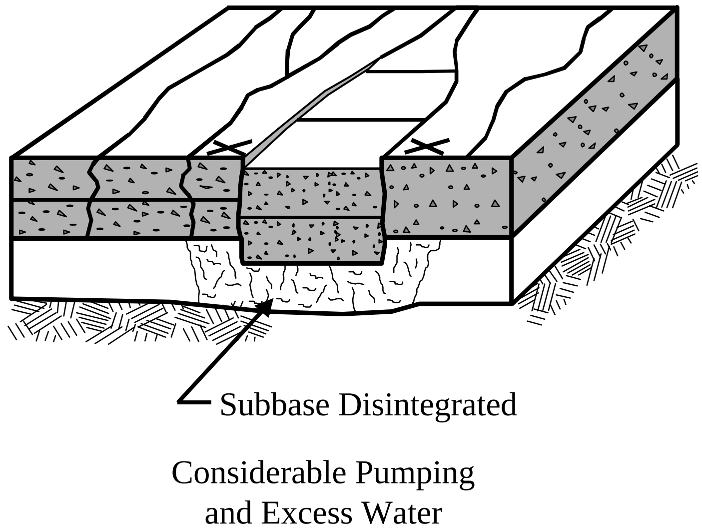


Figure 4-5.5. Potential deterioration of subbase near CRCP structural distress (punchout).

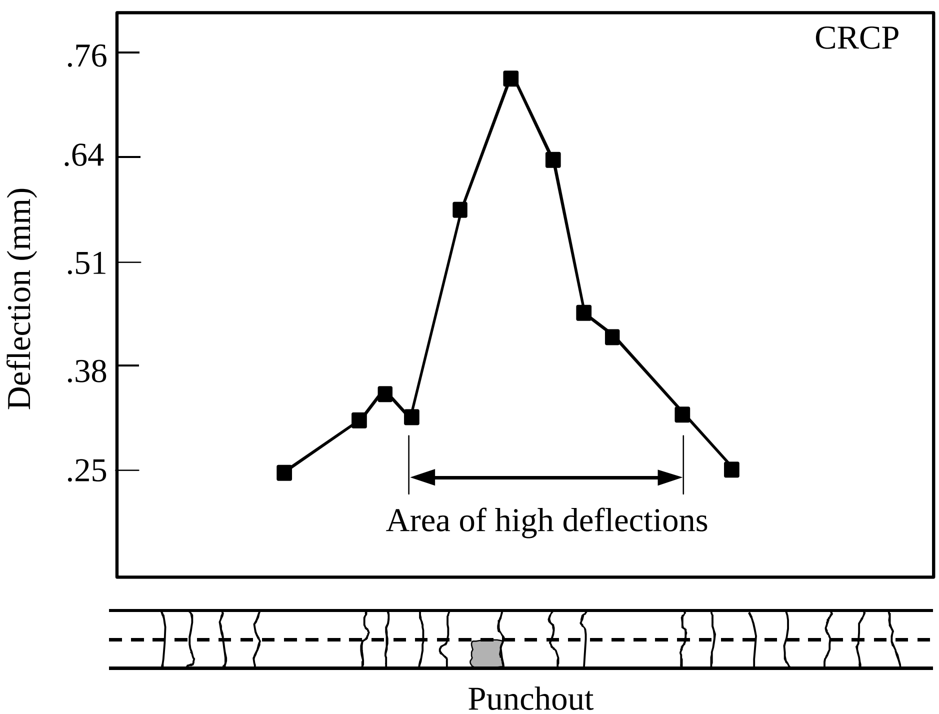


Figure 4-5.6. Deflection profile near a distressed area.[7]

Guidelines for the determination of repair boundaries for CRCP are given below:[8,9]

1. A minimum repair length of 1.8 m is recommended if the reinforcing steel is tied; 1.2 m if the steel is mechanically connected or welded.

2. The boundaries between repairs should not be closer than 460 mm, although sometimes this cannot be avoided if cracks are very close together. Where cracks are very closely spaced, it may be necessary to place the repair as close as 150 mm to an existing tight transverse crack.

3. Full-lane-width repairs are generally recommended, although a minimum width of 1.8 m may be used when all distress is contained within that width.

These criteria are given to provide adequate lap length and cleanout and to minimize repair rocking, pumping, and breakup. Figure 4-5.7 illustrates these construction recommendations.

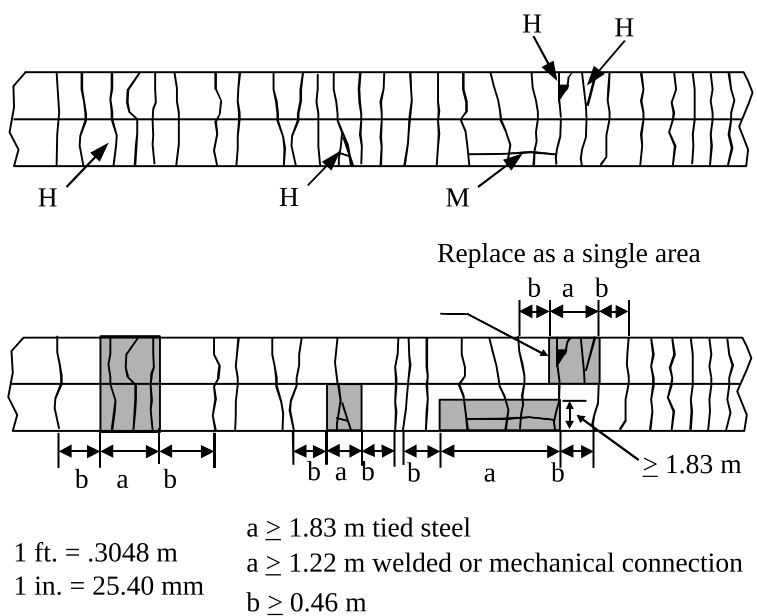


Figure 4-5.7. Repair recommendations for continuously reinforced concrete pavements.

Sawcutting

For CRCP, two sets of sawcuts are required. First, a partial-depth cut is made around the perimeter of the repair area, to a depth of about one-fourth to one-third of the slab. Then two full-depth sawcuts are made a specified distance in from the partial-depth cuts. The distance depends on the method of lapping used to connect reinforcement; the recommended distance is 610 mm for tied laps, and 200 mm for mechanical connections or welded laps.

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.

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) 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. However, this practice may result in the occurrence of blowups in the adjacent nonrepaired lane.

<u>Jointed Concrete Pavements</u>

It is generally not necessary to match joints in adjacent lanes, as long as:

- The minimum length requirements are met.
- All of the distressed area has been encompassed.
- A separation fiberboard has been placed along the longitudinal joint.

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 or to delay repair work until cooler weather prevails.

<u>Continuously Reinforced Concrete Pavements</u>

If a distress such as a wide crack with ruptured steel occurs across all lanes, special considerations are necessary because of the high potential for blowups in the adjacent lane, for crushing of the new repair during the first few hours of curing by the expanding CRCP slab, and for serious cracking of the repair during the first night as the existing CRCP contracts. The following procedures will minimize repair deterioration problems:

- The concrete should be placed in the afternoon to avoid being crushed by the expanding CRCP slab.

- The lane having the lowest truck traffic should be repaired first.

Concrete Repair Materials

The concrete mixture should be selected based on the available lane closure time. The shorter the time available before opening to traffic, the more rapid the curing of the concrete must be, and also the more expensive the concrete becomes. It may sometimes be acceptable to allow the concrete to cure for several days, allowing the use of a conventional concrete mixture. If earlier opening times (4 to 24 hours) are needed, high early strength concrete must be used.

Typical 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), and an accelerator to permit opening in 1 to 3 days, and sometimes in as little as 4 hours to 6 hours.[10,11] Type III cement, high cement factors (385 to 530 kg/m^3), and chemical accelerators are required for opening in 4 to 6 hours. The use of proprietary concrete mixes are necessary to achieve opening times in as little as 2 hours.[11] However, costs of many of these special materials are much greater than the typical paving concrete. Table 4-5.3 provides the approximate time required for different concrete mixes to achieve a compressive strength of 13.8 MPa.[5] Laboratory testing of proposed repair materials using available aggregates must be conducted to ensure that the opening requirements are met.

Table 4-5.3. Approximate time required for mixes to achieve 13.8 MPa compressive strength.[5]

Concrete Mix	**Approximate Time to Achieve 13.8 MPa Compressive Strength**
Blended Cements (Proprietary Materials)	2–4 hours
Sulfo-Aluminate Cements	2–4 hours
Type III with Non-Chloride Accelerator	4–6 hours
Type III with Calcium Chloride Accelerator	4–6 hours
Type 1 with Calcium Chloride Accelerator	6–8 hours
Type III with Type A Water Reducer	12–24 hours
Type I	24–72 hours

To ensure the durability of the repair, the concrete mix should have between 4.5 and 7.5 percent entrained air, depending on the maximum coarse aggregate size and the climate.[5] The slump should be between 50 and 100 mm for overall workability and finishibility. Other properties of the repair material, including drying shrinkage, coefficient of thermal expansion, and elastic modulus, can also affect performance and should be evaluated. A study is currently underway to evaluate the effect of various material properties on field performance.[12] The material selection should also consider the condition of the existing pavement. The estimated service lives of the pavement and the patch should be somewhat compatible. There is little sense paying for a patch that is designed to last 20 years when the rest of the pavement is only expected to last 10 years.

Curing and Opening to Traffic

Moisture retention and temperature during the curing period are critical to the ultimate strength of the concrete. Tensile stresses caused by the loss of water at the repair surface may cause surface shrinkage cracks if the concrete has not attained sufficient tensile strength. Proper curing is even more important when using accelerating admixtures.

Application of a curing compound after the repair has been finished is the most common means of curing the concrete. This curing compound serves to keep moisture within the concrete and thereby promotes additional cement hydration. White pigmented curing compound (ASTM M-148) is used by most highway agencies.

On very early opening projects (4 to 6 hours), it may sometimes be necessary to use insulation blankets to obtain the required strength within the available time. The insulation blankets contribute to rapid strength gain by keeping the internal temperature of the concrete high, thus accelerating the rate of hydration. In general, insulation blankets are usually not needed on hot summer days, and in fact may contribute to the development of cracking.

The curing time required before the repair can be opened to traffic depends on several factors, including the concrete mix type and ambient curing conditions. There are generally two criteria that may be used to specify when the repair can be opened to traffic:[(11)]

1. Minimum strength - An agency may stipulate that the repair attain a minimum strength before it may be opened to traffic. Recommended minimum strength requirements are:[(1,11)]

    - Compressive Strength: 13.8 MPa.
    - Modulus of Rupture: 2.1 MPa center-point loading, or 1.7 MPa third-point loading.

2. Minimum time to opening - An agency may specify the mix design and curing procedure, and then based on the ambient temperature at placement and slab thickness, set the minimum time to opening to traffic.

It is generally preferable to have a measure of the actual concrete strength before allowing the repair to be opened to traffic, particularly for repairs requiring to be opened very quickly. In such cases, the use of maturity meters or pulse-velocity devices for monitoring concrete strengths may be needed.[(5)]

Pressure Relief Joints

Pressure relief joints should only be used in long-jointed pavements where blowups are a pronounced problem (see module 4-3 for a more complete discussion of this topic). They are generally not needed as part of a full-depth repair project since any pressure in the slab is relieved through the installation process. If needed, they may be placed at one end of a full-depth repair (the joint should be doweled) or they may be placed at mid-panel of the slab. In either case, their width should be limited to 50 mm or less.[(13)]

Transverse Joint Design/Load Transfer

The design of the transverse joint is one of the most critical factors influencing the performance of full-depth repairs. The performance of various joint designs under similar traffic levels within the agency should be used as a guide in selecting an appropriate joint design for a particular project. This section briefly describes the types of joints and load transfer methods that can be incorporated into full-depth repairs.

Load transfer is the ability to transmit wheel loads (and associated deflections, stresses, and strains) across a joint (or crack) in the pavement. Poor load transfer allows differential movement of the slabs under load that can cause serious spalling, rocking, pumping, faulting, and even breakup of the adjacent slab or repair itself. To obtain good repair performance, good load transfer must be provided across the transverse repair joints. The following section describes ways of achieving load transfer in rigid pavements (JCP and CRCP).

Jointed Concrete Pavements

Methods of Achieving Load Transfer

Four techniques have been used with varying degrees of success to achieve load transfer across transverse repair joints:

1. Dowel Bars - These are smooth steel bars anchored into the existing slab, used where free horizontal movement of the joint is desired. Dowel bars are generally epoxy-coated for corrosion resistance. This is the most reliable method of providing load transfer.

2. Tie Bars - These are large-diameter deformed rebars (typically 25 mm) anchored into the existing slab, used where no horizontal movement of the joint is desired. It should be noted that tie bars in themselves are not designed to transfer the load, but rather to maintain the abutting joint faces in intimate contact so that effective aggregate interlock load transfer can be achieved.

3. Undercutting - Undercutting is achieved by excavating behind and beneath the existing slab and filling the excavated area with concrete. This method provides load transfer only on the approach side of the repair, and the load transfer provided is often inadequate because of poor consolidation of concrete in the undercut area.(2) In areas subjected to freeze-thaw action, the differential heaving between the repair and existing slab may cause severe roughness. For these reasons, the use of this method is not recommended.

4. Aggregate Interlock - Aggregate interlock can be achieved on short-jointed concrete pavements by providing rough-faced joints using partial-depth sawing. This method of load transfer is not always reliable, especially for longer slabs or in cold weather because the aggregate interlock is lost when the joints open more than 1 mm. This method is not generally recommended, except perhaps for short-jointed pavements under light traffic.

Determination of Required Load Transfer

Each agency must determine the load transfer required to prevent serious faulting or rocking of their full-depth repairs, given the specific climatic zone, traffic level, and foundation type. Analysis of data from many full-depth repairs in the central United States for pavements with poor drainage conditions and granular bases has shown that faulting of full-depth repair joints will, on the average, exceed 5 mm if 100 or more commercial trucks per day use the traffic lane over a 10-year period.(14) Transverse joint faulting that exceeds 5 mm is definitely noticeable to drivers.(15) Less precipitation, a warmer climate, and the presence of stabilized bases may allow for higher truck traffic loadings.

The use of mechanical load transfer devices is strongly recommended for most full-depth repairs because they provide better performance (less faulting, rocking, and other joint-related distresses) than other means of load transfer. See references 1, 5, 6, 16, 17, and 18. The following general recommendations are given for load transfer at transverse joints:

- JPCP - If the existing slab contains dowels at transverse joints, the repair should be doweled. If the existing slab does not contain dowels, then table 4-5.4 should be used to determine the need for load transfer. This table is a general guideline only, but it indicates the factors that need to be considered in evaluating the need for load transfer.

- JRCP. Since JRCP has longer joint spacings, more movement must be accommodated at the joints. Therefore, it is recommended that dowels be placed at both repair joints. The repair joint should be designed similarly to that of a new pavement joint.

Figure 4-5.8 shows one recommended layout of the dowels or rebars. This design has been used successfully by the Illinois DOT on roads with heavy truck traffic.(2) At least four to five dowels should be located in the wheel path to provide effective load transfer. The use of 38 mm diameter dowels is recommended for most interstate pavements because they provide the most effective load transfer.(5) For light traffic and for pavements less than 250 mm thick, 32-mm diameter dowels may be acceptable.(5) Past experience has shown that 25-mm diameter dowels are not adequate to withstand the bearing stresses in repair joints.(2,5)

In general, smooth dowel bars should be used at the repair joints in order to allow free horizontal movement. This is particularly critical for JRCP. If a tied joint is used, it should be placed in the approach joint because this joint tends to become very tight due to the action of truck wheels pushing the repair backwards.(2) Tied joints are not recommended for JRCP due to generally greater slab movements.

Continuously Reinforced Concrete Pavements

On CRCP, the rough joint faces and continuity of reinforcement (reestablished during repair), which keeps the joints tightly closed, provide the load transfer across the repair joints through aggregate interlock. 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. It is also important not to damage the adjacent concrete during removal.

Cost Considerations

The cost of full-depth repairs on jointed concrete pavements varies significantly, depending on the locality and on the site conditions (e.g., traffic). Typical recent costs for 1.8 m repairs on a 250-mm slab range from $60/m^2 to $120/m^2, with many falling between $78/m^2 and $84/m^2. Repair costs for CRCP are significantly higher.

Since the highest cost items for full-depth repairs are full-depth sawing and jointing (including load transfer), the unit cost of repair can be reduced significantly when a larger area is involved. For example, typical recent costs for 9-m slab replacements range from $54/m^2 to $78/m^2. The replacement of the entire slab is a more cost-effective solution than the placement of a series of smaller repairs within the same slab, and it is also more reliable.

Consequently, it may be useful to set up various sizes of full-depth repairs for pay items. One recommended breakout for different full-depth repair sizes is as follows:(1)

1. Type I — less than 4.2 m^2.
2. Type II — 4.2 to 12.5 m^2.
3. Type III — more than 12.5 m^2.

Table 4-5.4. Full-depth repair load transfer recommendations for non-doweled JPCP.

Climate*	Subbase	Average Annual Daily Truck Traffic		
		Light (< 100)	**Medium** (100 to 500)	**Heavy** (> 500)
Wet	Granular	Aggregate Interlock	Dowels**	Dowels
	High Quality Stabilized	Aggregate Interlock	Dowels**	Dowels
Dry	Granular	Aggregate Interlock	Dowels**	Dowels
	High Quality Stabilized	Aggregate Interlock	Aggregate Interlock	Dowels**

* The boundary between wet and dry climates is a Thornthwaite Moisture Index = 0.
** Recommended if the required future pavement life is greater than 5 years.

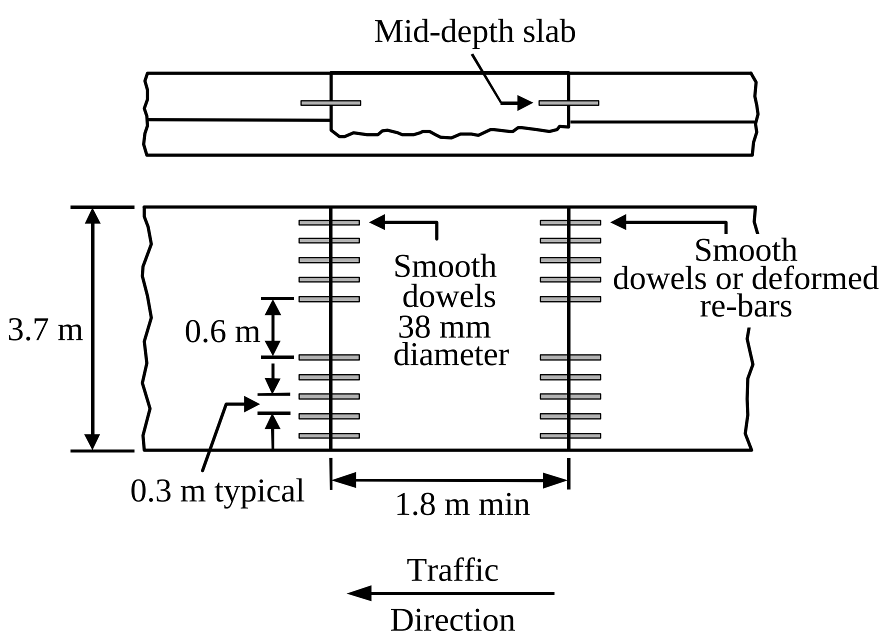


Figure 4-5.8. Recommended dowel bar design for interstate-type pavements.(2)

Thus, for a given distressed area the engineer could estimate the cost for "large removal and replacement" and for repair of each localized distress using typical unit costs and then select the lower-cost alternative. If the costs are approximately the same, the large area removal and replacement should be selected because this will be the more reliable repair.

7. CONSTRUCTION PROCEDURES

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

1. Layout of repair locations.
2. Concrete sawing.
3. Concrete removal.
4. Repair area preparation.
5. Provision of load transfer.
6. Longitudinal joint considerations.
7. Concrete placement and finishing.
8. Curing.
9. Joint sealing on JCP.

Each of these items is described in the following sections. Further guidance can be found in other publications.(5,6) The Federal Highway Administration also provides sample guide specifications that can be tailored to reflect local conditions.(6)

Layout of Repair Locations

As discussed previously, identifying the actual locations and boundaries of full-depth repairs is one of the most difficult tasks facing the construction engineer. Repair boundaries can be determined by making a field survey utilizing the data obtained from the initial project survey. This survey should be conducted as close to the contract schedule as possible. Each distressed area should be examined and the repair boundaries marked on the slab surface. Additional areas of distress that have occurred since the initial survey should be included.

Concrete Sawing

Jointed Concrete Pavements

Repair boundaries should be sawed with diamond saw blades. Full-depth cuts are recommended, although partial-depth cuts followed by jackhammering can be used. On hot days, it may not be possible to make this cut 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 outside the wheel sawcuts. To prevent damage to the subbase, the wheel saw must not be allowed to penetrate more than 13 mm into the subbase. The longitudinal joint (and concrete shoulder, if it exists) should be cut full-depth.

Figure 4-5.9 illustrates the sawing pattern for jointed concrete pavements. The slanted cut shown in the 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.(5)

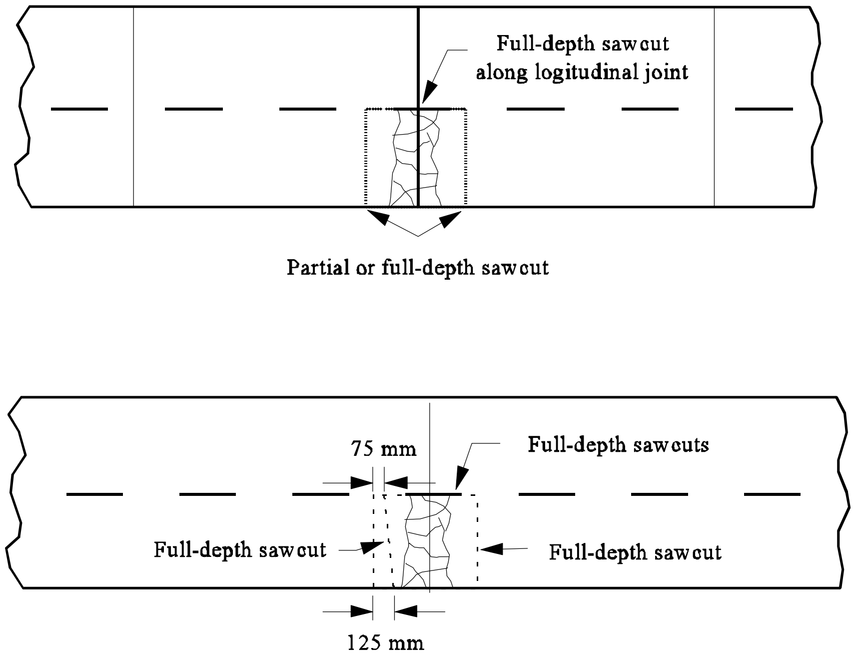


Figure 4-5.9. Sawcut locations for full-depth repair of jointed concrete pavements.

Full-depth sawing creates a smooth joint face with no load transfer capacity and thus requires the use of mechanical load transfer devices to ensure joint performance. Because of the 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. Thus, it is generally recommended that no more than 2 days of traffic be allowed over the sawed repairs before removal procedures begin.[(6)]

When an asphalt shoulder is present, it is necessary to remove the shoulder surface approximately 150 mm along the repair to provide space for the outside edge form. This will also prevent 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.

Continuously Reinforced Concrete Pavements

The outer boundaries of the repair area should be cut, partial-depth, with a diamond blade saw as shown in figure 4-5.10.[(6)] The partial-depth sawcuts should be located at least 460 mm 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.

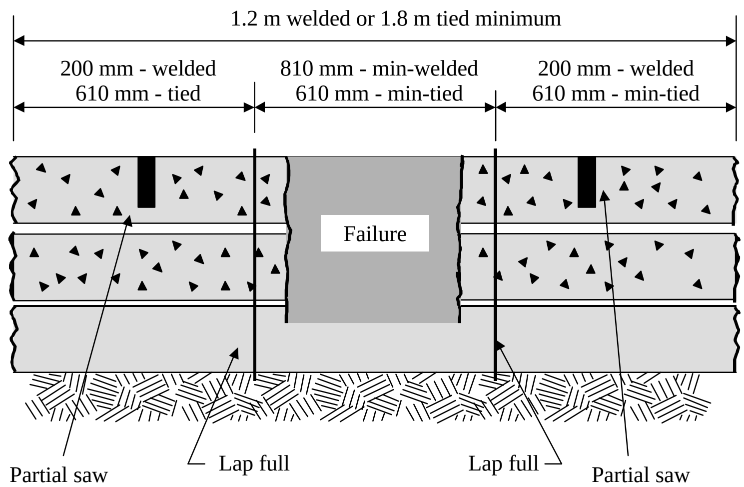


Figure 4-5.10. Required sawcuts for CRCP.[6]

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 4-5.10. This distance depends on the method of lapping used to connect reinforcement. The recommended distance is 610 mm for tied laps, and 200 mm for mechanical connections or welded laps. This distance may be reduced depending on the required lap length.

After the concrete has been removed, the reinforcement should be inspected for damage. Bent bars should be carefully straightened. 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.

Concrete Removal

Jointed Concrete Pavements

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

1. Breakup and Cleanout Method. 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.[1,5,6] Breakup should begin at the center of the repair area and not at the sawcuts.

2. Lift-Out Method. 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.[1,5,6]

Advantages and disadvantages of each method are listed in table 4-5.5.

Table 4-5.5. Advantages and disadvantages of concrete removal methods.

BREAKUP AND CLEANOUT	
Advantages	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.
Disadvantages	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	
Advantages	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.
Disadvantages	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.

The lift-out method is recommended 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.[6]

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.

<u>Continuously Reinforced Concrete Pavement</u>

The procedure for removing concrete from the center section (between the inner full-depth sawcuts) of the repair area is the same as JCP. As with JCP, the lift-out method is recommended. Concrete in the two end lap areas (between the partial-depth and full-depth cuts) must be carefully removed to avoid damaging the reinforcement in the lap area and spalling concrete at the bottom of the joint (beneath the partial-depth sawcut). This can be accomplished by using jackhammers, prying bars, picks, shovels, and other hand tools.

Breaking the concrete from around 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. The size of the jackhammer used in the lap area should be limited to approximately 6.8 kg.

The reinforcement must not be bent up to facilitate removal of concrete since the bars cannot be properly straightened afterward. Bent reinforcement in the repair area will eventually result in spalling of the repair because of the large stresses carried by the reinforcement.

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 exists in the repair area, it should be dried out before placing new material. Placement of a lateral drain may be necessary in some cases where excessive water exists. 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 the best alternative.

Provision of Load Transfer

Placing Dowel Bars and Rebars in JCP

Both smooth steel dowel bars and deformed rebar can be used for load transfer across the repair joint. The dowels should be installed by drilling holes on 305 mm centers at mid-depth of the exposed face of the existing slab. The holes can be drilled rapidly using tractor-mounted gang drills (several drills mounted in parallel on a rigid frame).(5) This equipment drills several holes simultaneously, while maintaining proper horizontal and vertical alignment. The use of a single hand-held drill is not recommended because of the likelihood of misalignment.(1) Proper hole alignment is crucial to guard against premature deterioration of the repair.

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 6 mm larger than the dowel diameter so that stiffer grout could be used.(2) 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 larger than the dowel diameter, because this type of material can often ooze out through small gaps. Since epoxy materials are often more flexible than the supporting concrete, thin layers are desirable to reduce deformation of the epoxy mortar and the accompanying dowel deflection.

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.(2) The following procedure is recommended for anchoring dowel bars:(2,5,6)

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.

2. Place quick-setting, non-shrinking cement grout or epoxy resin in the back of the dowel hole. The grout can be placed by using a flexible tube with a long nose that places the material in the back of the hole; epoxy-type materials can be placed using a cartridge with a long nozzle that dispenses the material to the rear of the hole.

3. Place a grout retention disk (a thin donut-shaped plastic disk) over the dowel and against the slab face, as illustrated in figure 4-5.11. 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).

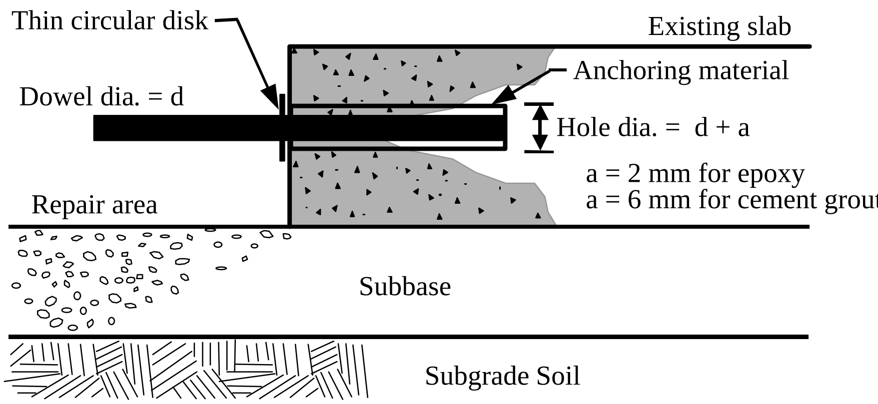


Figure 4-5.11. Illustration of dowel bar anchoring in slab face.

4. 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.

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 cover and 65-mm edge clearance.

Placing Reinforcing Steel in CRCP

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 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, and a minimum of 63-mm cover should be provided.

Depending on the type of splice used, different overlap lengths are required to allow the splice to develop the full bar strength. For both splices, a 50-mm clearance is required between the end of the lap and the existing pavement. The recommended lap lengths are given:[6,7]

1. Tied splice - Tied splice should be lapped 460 mm for 16-mm bars, and 533 mm for 19-mm bars.

2. Welded splice - A 6-mm continuous weld should be made either 100 mm long on both sides, or 200 mm 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 4-5.12. 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.

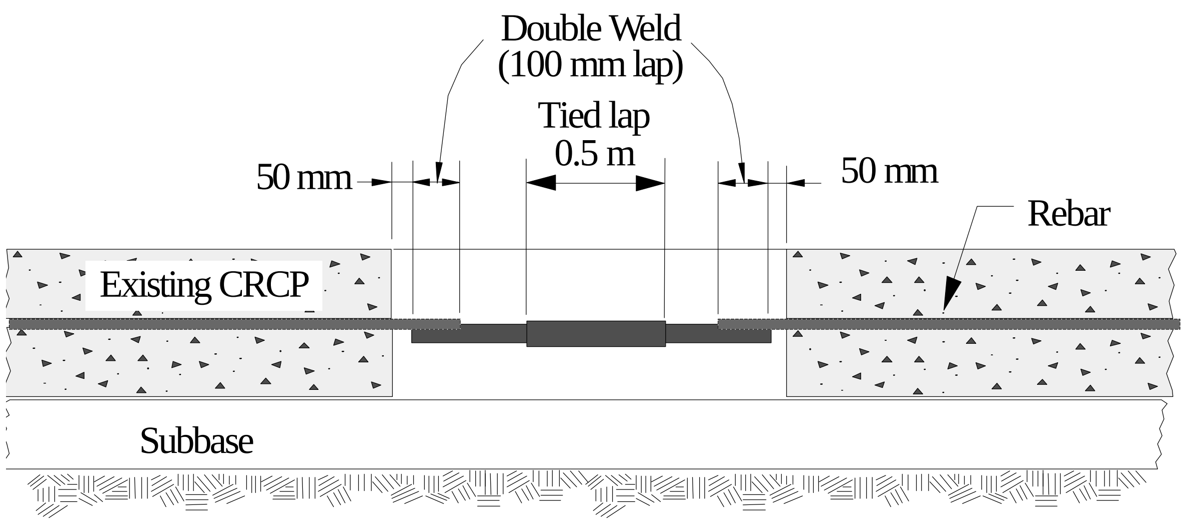


Figure 4-5.12. Details of welded or mechanical connection for CRCP repair.[6]

As previously mentioned, some agencies have experimented with making a single full-depth cut and drilling holes in the face of the exposed slab. This procedure expedites construction because it does not require any hand chipping of the concrete to expose the lap length.[5] New rebar is then anchored in the holes using an approved anchoring material. The effectiveness of this technique in terms of long-term performance is not yet known.

Longitudinal Joint Considerations

A bond-breaking material (e.g., fiberboard) should be placed along the existing longitudinal joint to ensure independent action between the lanes. This should be performed prior to concrete placement. Longitudinal joints between the repair and the adjacent PCC traffic lane are usually not tied to allow unrestricted movement of the repair at these joints.

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.[1,2] 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.

The best results have been obtained by using a vibratory screed parallel to the centerline of the pavement.[5] 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.

On CRCP repairs, it may be necessary to restrict the time of placing concrete to late afternoons, 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.

On longer repairs that require an intermediate joint, the timing of sawing is an important issue. 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. More specific guidelines based on repair length, cement type, and change in temperature throughout the year are also available.[4]

Curing

As soon as possible after texturing, the concrete should be covered with white-pigmented curing compound, wet burlap, or polyethylene sheeting, to prevent moisture loss. In general, a normal application of the pigmented curing compound gives the best results. The main function of these items is to prevent moisture loss from concrete during curing which can result in shrinkage cracks. As mentioned earlier, the need for early opening may sometimes require the use of insulation blankets to accelerate hydration and provide higher early strengths.

Joint Sealing on JCP

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. Module 4-2 should be consulted for the recommended procedures on the sealing of these joints.

8. SUMMARY

This module presents guidelines and procedures for full-depth repair of both JCP and CRCP. Only cast-in-place PCC repairs are described in this module. The use of bituminous full-depth patches is not recommended for permanent repair of rigid pavements, because bituminous patches allow excessive horizontal movements of slabs and provide no load transfer across the transverse joints.

Distressed areas requiring repair must be identified and the repair boundaries determined during a field survey using results from the initial distress survey. Structural testing (deflection testing and coring) provides valuable information on the extent of deterioration in distressed areas. Repair boundaries must be large enough to include all slab and foundation deterioration, but not so large as to unnecessarily increase repair costs. The actual construction of the full-depth repair must be carefully controlled through specifications and good inspection. Damage to the surrounding concrete and the foundation will lead to serious repair deterioration.

Areas of extensive deterioration may be more economically repaired by complete removal and replacement rather than by localized full-depth repairs. The unit cost for large area removal and replacement will generally be considerably lower than small area repairs (typically half the cost).

The opening-to-traffic requirements of the full-depth repair project will dictate the type of concrete mix to be used. Conventional concrete mixtures can be used for repairs that must be opened to traffic within 24 to 72 hours, and such mixes can be modified by increasing the cement content, reducing the water content, using an accelerator, and utilizing a layer of insulation on top for earlier opening times. In addition, proprietary cementitious repair materials are available for opening times as soon as 2 to 4 hours after placement.

Jointed Concrete Pavements

The design of the repair joint is perhaps the most critical aspect of JCP full-depth repair. Load transfer across the joint is essential to maintain the structural integrity of the existing slab and the repair under heavy truck traffic. Mechanical load transfer devices (deformed tie bars and smooth dowel bars) are recommended to achieve load transfer. Good dowel bar grouting procedures are critical to repair performance.

Adequate curing of the repair and timely joint sealing are also important to the performance of the full-depth repair. Proper curing will ensure that the repair has reached its desired strength before opening to traffic and effective joint sealing will help reduce spalling and minimize water infiltration.

Continuously Reinforced Concrete Pavements

Restoring the continuity of the reinforcement through the repair is critical to the performance of repairs in CRCP. Load transfer across the joint is essential to maintaining the structural integrity of the existing slab and the repair under heavy truck traffic. Joints must be rough-faced and vertical beneath the partial-depth sawcut. Concrete removal methods that cause spalling of the CRCP slab (e.g., the use of large drop hammers) should not be allowed.

9. REFERENCES

1. Darter, M. I., E. J. Barenberg, and W. A. Yrjanson. 1985a, "Joint Repair Methods for Portland Cement Concrete Pavements," NCHRP Report 281, Transportation Research Board, Washington, DC.

2. Snyder, M. B., M. J. Reiter, K. T. Hall, and M. I. Darter. 1989a, "Rehabilitation of Concrete Pavements, Volume I — Repair Rehabilitation Techniques," FHWA-RD-88-071, Federal Highway Administration, Washington, DC.

3. Stoffels, S. M., W. P. Kilareski, and P. D. Cady. 1993, "Evaluation of Concrete Pavement Rehabilitation in Pennsylvania," Proceedings of the Fifth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafayette, IN.

4. Carmichael III, R. F., A. H. Meyer, L. L. Caldwell, and B. F. McCullough. 1989, "Rapid Replacement of Portland Cement Concrete Pavement Segments," Research Results Digest Number 169, National Cooperative Highway Research Program, Transportation Research Board, Washington, DC.

5. American Concrete Pavement Association (ACPA). 1995, "Guidelines for Full-Depth Repair," Technical Bulletin TB002.02P, American Concrete Pavement Association, Skokie, IL.

6. Federal Highway Administration (FHWA). 1985, "Pavement Rehabilitation Manual," FHWA-ED-88-025. Federal Highway Administration, Washington, DC, (Manual supplemented April 1986, July 1987, March 1988, February 1989, October 1990).

7. Barnett, T. L., M. I. Darter, and N. R. Laybourne. 1980, "Evaluation of Maintenance/Rehabilitation Alternatives for CRCP," Report No. 901-3, Illinois Department of Transportation, Springfield, IL.

8. Transportation Research Board (TRB). 1979, “Failure and Repair of Continuously Reinforced Concrete Pavement,” NCHRP Synthesis of Highway Practice 60, Transportation Research Board, Washington, DC.

9. Darter, M. I., T. L. Barnett, and D. J. Morrill. 1982, “Repair and Preventative Maintenance Procedures for CRCP,” FHWA/IL/UI-191, Illinois Department of Transportation, Springfield, IL.

10. American Concrete Pavement Association (ACPA). 1994, “Fast Track Concrete Pavements,” Technical Bulletin TB-004.02, American Concrete Pavement Association, Skokie, IL.

11. Whiting, D. M. Nagi, P. A. Okamoto, H. T. Yu, D. G. Peshkin, K. D. Smith, M. I. Darter, J. Clifton, and L. Kaetzel. 1994, “Optimization of Highway Concrete Technology,” SHRP-C-373, Strategic Highway Research Program, Washington, DC.

12. Smoak, W. G., T. B. Husbands, and J. E. McDonald. 1997, “Results of Laboratory Tests on Materials for Thin Repair of Concrete Surfaces,” Technical Report REMR-CS-52, U.S. Army Corps of Engineers, Washington, DC.

13. Snyder, M. B., K. D. Smith, and M. I. Darter. 1989b, “An Evaluation of Pressure Relief Joint Installations,” Transportation Research Record 1215, Transportation Research Board, Washington, DC.

14. Ortiz, D., E. J. Barenberg, M. I. Darter, and J. Darling. 1986, “Effectiveness of Existing Rehabilitation Techniques for Jointed Concrete Pavements,” FHWA/IL/UI-215, Federal Highway Administration, Washington, DC.

15. Darter, M. I., J. M. Becker, M. B. Snyder, and R. E. Smith. 1985b, “Development of a System for Nationwide Evaluation of Portland Cement Concrete Pavements,” NCHRP Report 277, Transportation Research Board, Washington, DC.

16. Shober, S. and K. Johnson. 1983, “Experimental Rehabilitation of Jointed Portland Cement Concrete Pavements,” FHWA/WI-83/2, Wisconsin Department of Transportation, Madison, WI.

17. Sharma, A. K. 1988, “Experimental Rehabilitation of Jointed Portland Cement Concrete Pavement,” FHWA/WI-88/11, Wisconsin Department of Transportation, Madison, WI.

18. Federal Highway Administration (FHWA). 1996, “Pavement Notebook for FHWA Engineers,” FHWA-ED-90-016, Federal Highway Administration, Washington, DC.

MODULE 4-6

ACCELERATED RIGID PAVING TECHNIQUES

1. INSTRUCTIONAL OBJECTIVES

This module describes some of the currently available procedures and techniques that can be used to perform rapid rehabilitation of rigid pavements. After completion of this module, the participant will be able to:

1. Identify conditions under which these techniques may be considered as a part of a rehabilitation project.
2. Discuss the various materials that can be used to achieve high early strengths, and how to select the appropriate material for the time available to perform the rehabilitation.
3. Discuss the special construction procedures that accompany the use of accelerated paving techniques.
4. List the innovative field testing procedures that can be used to monitor concrete strength.
5. Determine the appropriate criteria for opening the pavement to traffic.

2. INTRODUCTION

Most of the paving community is by now familiar with the concept of accelerated rigid paving techniques (ARPT, also known as fast track paving). This technology, which is actually a combination of appropriate mix designs and paving techniques, is used to reduce the time between placing portland cement concrete (PCC) and opening a rigid pavement to traffic from 5 to 14 days down to 24 hours or less. Fast track paving has been used in applications where a strong, durable surface is desired and traffic control constraints place severe limitations on the available closure time for construction. Fast track paving technology has been developed and promoted to address what may have otherwise been considered as one of the major limitations of PCC in rehabilitation, the time required for curing.

The FHWA defines ARPT as any technique that accelerates conventional concrete paving so as to reduce any of the following:[(1)]

- Number of lane closure hours.
- Number of lanes closed.
- Overall delay time to public traffic.

It should be noted that these techniques do not concentrate on technical aspects such as mix design. Instead, they address the flow of traffic and, indirectly, user delays.

This module presents information on recent developments in the area of ARPT. These techniques are being widely used with great success in all PCC paving applications, including partial-depth repairs, full-depth repair/slab replacements, reconstruction, and concrete overlays. Many highway agencies now require same-day opening on most full-depth and partial-depth repair projects as a standard practice. The use of ARPT on larger rehabilitation projects, such as reconstruction or overlay, are somewhat more limited; however, this is now an option that is almost always considered, and frequently used, on time-sensitive projects. Most of the information presented in this module applies equally to new construction as well as to pavement rehabilitation.

3. IDENTIFICATION OF APPROPRIATE PROJECTS

Not all rigid pavement rehabilitation jobs are suitable candidates for ARPT, and even in appropriate jobs this technology need not be applied over the entire project. For example, on reconstruction projects that require lane closures of several days, only the sections placed on the last day or two are likely to require ARPT, because the sections placed earlier will have ample time to cure before opening. In general, unless the section being rehabilitated can be opened to traffic following each day's work (which is usually possible on slab repair or replacement projects), no advantage can be gained by using ARPT on that part of the project. The use of ARPT is appropriate only when the use of this technology results in some form of savings, such as reducing direct agency costs or user delay costs.

Often, using ARPT has a higher initial cost than conventional alternatives because of the associated high cost of the materials and equipment or, sometimes, the unfamiliarity of contractors with some of the techniques. Because of these increased costs, the need for ARPT on a particular project should be carefully evaluated over the entire length of the project before its use is deemed appropriate. Some guidelines are suggested below to assist in the determination of when early opening techniques might be considered appropriate:

- Urban Intersections. It is often not feasible to close major urban intersections for an extended period of time because of traffic congestion problems. Therefore, when these areas are rehabilitated, the desire to minimize the closure time frequently dictates what types of repairs are performed, rather than what needs to be done. The use of ARPT permits the reconstruction of urban intersections using PCC. Either the whole intersection can be rehabilitated by closing the entire intersection for a short time (over a weekend or overnight, for example), or the work could be completed in quadrants, under traffic.[2]

- Commercial Areas. On parking areas, loading areas, or access roads to commercial or industrial areas, the owners would expect access to be maintained almost without interruption. ARPT provides a solution to minimizing the interruption due to road work, while providing the pavement structure needed for good long-term performance.

- Single Access Roads. Public access can be a major factor determining the feasible solutions in residential or rural route rehabilitation projects. On these projects, often the road being rehabilitated provides the only access, or, in the case of rural roads, the detour route can be excessively long. ARPT makes it feasible to construct rigid pavements under these circumstances with minimal closure time (some in as little as 24 hours).

- Urban Highways. For urban highways, it can be argued that the most desirable method of rehabilitation is the one that causes the least disruption to traffic; that is, the one that can be performed the quickest. The reconstruction of an urban highway can be accomplished in a number of ways, including lane by lane (with some lanes being left open to traffic), with closures and detours between intersections, and by direction (with appropriately-placed cross-overs). These methods all have very high costs and safety problems associated with lane closures, reduced traffic flow, and disruption to traffic patterns. These can all be greatly reduced through the application of ARPT.

In some cases, ARPT may be needed to facilitate the construction process itself. This is particularly applicable on projects involving large areas of paving, such as parking lots, loading areas, and airfield pavements. Unless zero-clearance pavers are used, paving can not commence immediately adjacent to newly placed slabs until the slabs have gained sufficient strength to support the paving equipment. In such projects, it may be advantageous to use high early-strength mixes on selected lanes at the beginning of the project to either reduce or eliminate the delay time for curing until the adjacent lanes can be placed. A similar example is the use of ARPT at the end of one day's paving to facilitate the next day's startup.(2)

While suitable projects may be identified on the basis of location and the adverse effect of delays, implicit in the consideration of location should be the notion of working within the available time to perform the rehabilitation. It is often not advantageous or necessary to complete a job as quickly as possible, as there may be absolutely no benefit derived from having a job done sooner. As previously mentioned, there are usually increased costs associated with doing the job faster, and there is no reason to incur such costs if they are not needed. Conversely, many jobs or parts of jobs are best performed with the least amount of disruption possible.

4. MATERIAL SELECTION

A simple rule in selecting the material for an accelerated paving project is to use the least exotic (i.e., most conventional) material that will meet the demands of the job. The current state of the art in rigid pavement repair materials is such that virtually any opening time requirement (from less than 1 hour to 24 hours or more) can be met, using either conventional PCC or a proprietary material. However, the scope of work, job site conditions, available equipment, and cost may limit the feasibility of certain options.

In general, faster-setting mixes have higher corresponding costs and special handling requirements. Very rapid setting mixes may require a mobile mixer because of their short available working time (15 to 30 minutes), which may be even shorter in hot weather. When the ambient temperature is in excess of 32 °C, it may be very difficult to place some of the very fast materials because they harden so quickly. Although a set retarder can be used with some of these mixes to provide longer working times, it is probably a better solution to use a slower setting mix.

The local climatic conditions are an important factor to consider in selecting a material for an accelerated paving project. The steps required to achieve high early strength usually lead to high heat of hydration. This can be a problem during both hot and cold weather. During hot, sunny days, solar radiation can significantly raise the temperature at the slab surface, adding to the temperature gradient. A high temperature gradient can also result from rapid cooling, caused either by removal of insulation blankets or a rapid drop in ambient temperature after sunset. These conditions may cause cracking if the concrete is not properly cured. To minimize the possibility of cracking during hot summer days, a very hot mix should not be used and night work should be considered. If a large temperature drop is expected, a faster setting mix should be used and the concrete should be placed early enough in the day to ensure that it achieves adequate strength before the anticipated temperature drop.

In evaluating the potential for cracking on a newly placed rigid pavement, the interaction of environmental variables, pavement structural design factors, pavement mix design properties, and pavement construction factors are complex. To assist in evaluating the potential for uncontrolled cracking in new rigid pavements, the FHWA is currently developing a computer program (HIPERPAV) that takes key environmental, structural design, mix design, and construction inputs and generates a graph showing the development of pavement strength and stress over the first 72 hours after placement. If the stress exceeds the strength at any time, this indicates time periods after placement during which the potential for cracking is very high, and thereby allows for the adjustment to mixes or construction activities to reduce the potential for cracking.

Types of Cementitious Materials

PCC mixes are the most widely used materials today in the accelerated construction and rehabilitation of rigid pavements. However, other special cements and proprietary materials have also been used successfully. 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 normally considered only 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. A great variety of proprietary patching materials are available, as discussed in module 4-4. Many of these materials, along with the procedures for partial- and full-depth repairs, are being monitored in ongoing research projects.[3,4]

For PCC mixes, the high early strength needed for accelerated paving is typically achieved by reducing the water/cement ratio through the addition of more cement. High range water reducers are also typically added to reduce the amount of water required without a loss in workability. Many fast track PCC mixes utilize Type III cement, but Type I and Type II cements have also been used successfully to produce fast track mixes. It is important to note that the ASTM cement type designation does not guarantee uniform performance. Two bags of the same type of cement can have drastically different strength development characteristics if they are from different sources. Laboratory tests should be conducted to evaluate the properties of locally available cement before specifying its use.

Examples of special cements that have been used to produce fast track concrete mixes include rapid set cement (RSC) and regulated set portland cement (RSPC). RSC is similar to type K expansive cement (high alumina), but it has been modified to reduce the expansions normally associated with Type K cement. RSC concrete can develop as much strength in 4 hours as conventional PCC will achieve in 28 days. RSPC (also known as jet-set cement) is similar to ordinary portland cement, except 20 to 25 percent of the calcium aluminate phases have been replaced with calcium fluoroaluminate. The set time of this cement can be regulated in a range from 2 to 30 minutes using a set retarder. Iowa DOT showed that a concrete mix containing 363 kg/m^3 of RSPC will develop a flexural strength in excess of 2.1 MPa in 4 hours after mixing.[5]

Design of Fast Track Mixes

As previously mentioned, the approach generally adopted to achieve high early strength with a PCC mix has been to increase the cement content and minimize the water/cement ratio. Fast track mixes are often made with Type III cement, but Type I or Type II cement with accelerators are also used. These mix variables most directly affect the strength development characteristics of a PCC mix. Other variables affect the workability and durability of the mix. As with conventional concrete mixtures, air entraining agents and flyash are usually added to improve durability and workability.

In proportioning a fast track mix, greater attention must be paid to workability, because the desirable water/cement ratio for a fast track mix (0.42 ± 0.05) is much lower than that of normal mixes. Water reducers/super plasticizers can be added to the mix to maintain reasonable workability even at low water/cement ratios. Aggregate gradation can also have an effect on workability, with a uniformly graded aggregate providing better density and workability than a gap-graded aggregate.[2]

Flyash, a pozzolan material derived from the burning of coal, is often added to fast track mixtures to increase the ultimate strength and durability of the concrete. Type C flyash (from the burning of sub-bituminous and lignite coal) is often used in fast track mixes as a partial replacement for cement or as an additive. Type F flyash (from the burning of bituminous and anthracite coal) is actually better in those respects, but it can only be used as an additive (i.e., type F flyash can not be used as a partial replacement for cement). Flyash contributes to concrete durability by developing a stable air-void system; however, mixes containing flyash require a greater amount of air entraining agent (as much as 5 percent more) to produce the same percentage of entrained air. In plastic concrete, flyash has the effect of improving workability but also can slow the rate of early strength gain. The typical replacement ratio is 1.5:1, flyash to cement, by weight.

Depending on the cement type and ambient temperature, an accelerator may also be used in a fast track mix. With a Type III mix, an accelerator is normally needed only when a very fast opening time is required. A rich, Type III cement mix made with 2 percent calcium chloride (by weight of cement) can provide the strength required for opening in 4 hours.(6) The Ohio DOT uses a chloride accelerator and curing blankets to achieve their minimum strength for opening (modulus of rupture of 2.8 MPa) in 4 hours with their fast track mix, which contains 535 kg/m^3 of Type I cement.(7)

A number of field-verified fast track mixes are available for opening times from 4 hours to 24 hours. Table 4-6.1 lists some of the fast track PCC mixes along with a RSPC and a RSC mix. Figure 4-6.1 shows the typical strength development characteristics of Fast Track I and Fast Track II mixes.(8) Each agency should develop mix designs based on their local materials to ensure that sufficient strength gain is being achieved.

Table 4-6.1. Examples of Fast Track concrete mix design.(2,5)

Mix Component	**Type I (GADOT)**	**Type II**	**Type III (fast track I)**	**Type III (fast track II)**	**RSPC**	**RSC**
Cement, (kg/m^3)	447	391	381	441	363	386
Flyash, (kg/m^3)	–	59	43	48	–	–
Course Aggregate, (kg/m^3)	1067	936	828	776	1011	1070
Fine Aggregate, (kg/m^3)	612	631	808	774	832	595
w/c Ratio	0.40	0.39	0.40 to 0.48	0.40 to 0.48	0.41	0.45
Water Reducer	–	yes	yes	yes	–	–
Air Entraining Agent	As needed to obtain air content of 6 $\pm$ 2 percent.					
$CaCl_2$ % wt. cement	1.0	–	–	–	–	–
Accelerator (non-chloride)	–	yes	–	–	–	–

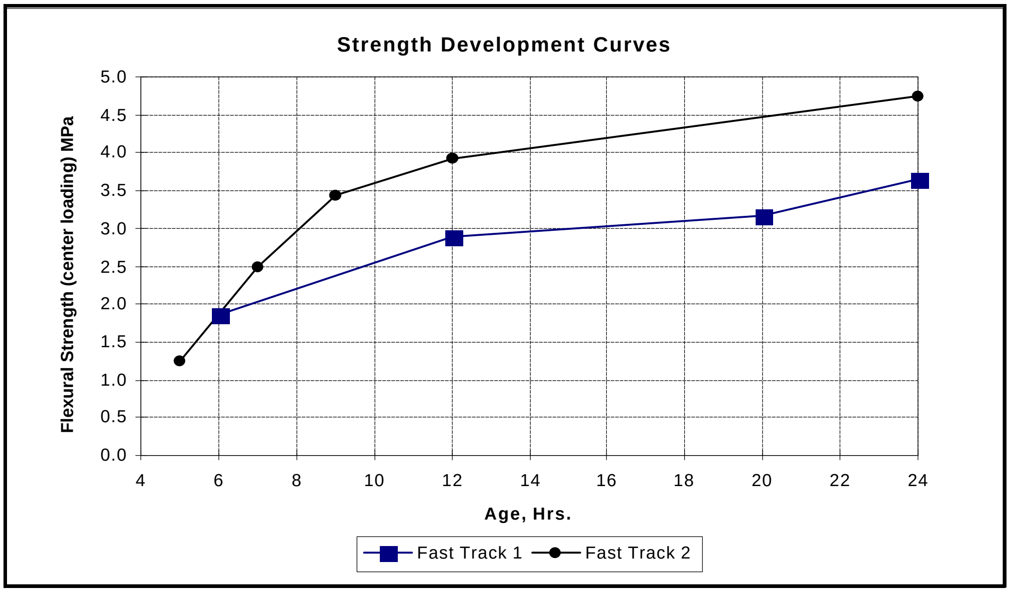


Figure 4-6.1. Typical strength development characteristics of Fast Track 1 and Fast Track 2 mixes.[8]

5. APPROPRIATE CONSTRUCTION TECHNIQUES

The compressed construction schedule and early opening of an accelerated paving project are achieved through careful planning, innovative use of available technology, and close coordination of all of the activities involved. Because the work must be completed as quickly as possible, accelerated paving requires a much closer coordination of activities than normal construction. For example, it is important to recognize that on repair/rehabilitation projects, the existing material must first be removed before the new material can be placed (except in the case of overlay projects). To allow the repair material to cure for as long as possible before opening to traffic, traffic control must be set up, old material removed, and new material placed as quickly as possible. Another complicating factor is the very short working time provided by many of the fast track mixes. Often these projects can only be accomplished at night or during non-peak traffic hours. Unless the job is carefully scheduled, either delays in opening or huge wastes of material can result. The success of an accelerated paving project depends on how well the whole repair/rehabilitation process is orchestrated.

Concrete Removal

Partial-depth repair, full-depth repair/slab replacement, and reconstruction projects require the removal of the deteriorated concrete. For partial-depth repairs, the damaged concrete can be removed either by sawing and chipping or cold milling. Cold milling is an effective means of rapidly removing deteriorated concrete from large areas. Hydroblasting can also be used for partial-depth removal of deteriorated concrete, but it is not suitable for use with many patching materials because it leaves the area wet.

If the sawing and chipping method is used, the sawing can be done in advance, leaving the sawed patches open to traffic, to facilitate the repair process. Sawing in advance allows the repair crew to start removing concrete and placing repair materials as soon as the area is closed to traffic, without having to wait for sawing. Also, this practice avoids the problem with the repair area being too wet from sawing. Only the repairs that can be filled within the available time should be removed during that shift to allow the area to be opened to traffic at the end of the period; however, the sawing can continue until all of the repair boundaries are cut.

For accelerated full-depth repair/slab replacement projects, the preferred method of concrete removal is the lift-out method (see module 4-5). In this method, the old concrete is lifted out in one or more pieces. Full-depth saw cuts are made with a diamond-blade saw at the repair boundaries and within the repair area to divide the slab into pieces small enough to be lifted out by the available lifting equipment. This method of removal helps to minimize or eliminate damage to the base and surrounding pavement/shoulder, and reduces the need for additional preparation, especially base reconstruction, which can be time-consuming.

As with the partial-depth repairs, one of the ways that concrete removal can be facilitated on full-depth repair/slab replacement projects is to identify and saw the repair boundaries in advance. The sawed boundaries do not adversely affect traffic and can be left until the material is removed and replaced. This allows one crew to continue to saw repair boundaries while another crew is removing and replacing slabs. This procedure significantly reduces the time lag until the first slab can be placed at the beginning of the work period. On several projects, the procedure of sawing in advance has been taken a step further: the old concrete was completely removed and a temporary structure was left to carry the traffic until the next day’s work. The temporary structure, which can be made with either timber or precast concrete, can be rapidly removed when it is time to place the repair material. This procedure allows the repair crew to start placing the new concrete almost immediately after the repair area has been closed off to traffic. Using this technique to replace slabs, even a large area can be reconstructed working only during the off-peak hours. The New York State DOT has used precast timber rafts to facilitate patching operations on some of their high-volume roadways.[9]

Efficient procedures for concrete removal are also available for rigid pavement reconstruction projects. The method most widely used today was developed in conjunction with rigid pavement recycling procedures. In this method, diesel pile hammers equipped with a special impact plate are used to break up the deteriorated slabs into pieces small enough for hauling. The broken slab is then windrowed for easy pickup with loading equipment and hauled away, either for disposal or recycling. Many accelerated reconstruction projects also involve recycling.

Concrete Placement Equipment

For full-depth repair/slab replacement projects, mobile concrete mixers can be used to allow the use of very fast mixes possessing short working times. Also, frame-mounted, gang drills can be used to facilitate the dowel or tie bar installation process. Gang drills consist of several drills mounted in parallel in a rigid frame to allow simultaneous drilling of several holes with accurate horizontal and vertical alignment.

For concrete reconstruction or overlay projects, several recent developments in equipment, such as zero-clearance pavers and dowel bar inserters, have been instrumental in making some of the new construction practices feasible. The zero-clearance pavers allow placement of a rigid pavement directly adjacent to traffic lanes with minimum encroachment. This allows single-lane reconstruction or resurfacing while allowing traffic on the adjacent lanes or shoulders. Dowel bar inserters allow direct placement of dowel bars in the rigid pavement joints without the need for dowel baskets. This not only reduces the amount of manual labor required for a project, but it keeps the roadbed clear of the dowel baskets, allowing lanes under construction to be used as the haul road and keeping the construction traffic off of the adjacent traffic lanes. Studies of the effectiveness of dowel bar inserters have shown that their placement accuracy is as good or better than traditional dowel baskets.[10]

When rigid pavements are being reconstructed or resurfaced one lane at a time while maintaining traffic on adjacent lanes, special precautions may be required to ensure the safety of the motorists using the open lane. The drop-off from the edge of the newly paved slab to grade can pose a serious safety hazard to users.[1] Some highway agencies limit the maximum height of the drop-off for adjacent lanes open to traffic.

Curing

There are two major considerations for curing in accelerated construction: moisture retention and heat retention. Under most conditions, adequate moisture retention can be achieved by applying pigmented curing compound at 1½ to 2 times the normal rate of application. The curing compound creates a seal that limits the evaporation of mix water and helps promote more thorough hydration. This practice has been effective in preventing shrinkage cracking and other curing-related problems.

Heat retention generally involves the use of some form of insulation. Whether or not the insulation is needed depends on climatic conditions. During hot weather, a satisfactory rate of strength gain can be achieved without any insulation. In fact, it is now recommended that insulation blankets be used only in cold weather concreting, when the anticipated time needed for strength attainment does not meet staging requirements, or when thermal shock from severe temperature drops may be experienced. If insulation is needed, curing blankets, consisting of closed cell polystyrene foam protected on one side by a plastic film, are recommended.[2]

Again, the FHWA is currently developing a computer program (HIPERPAV) that will assist the design engineer in evaluating the potential for uncontrolled cracking based on expected environmental conditions, proposed structural and mix designs, and proposed construction practices.

Joint Sawing and Sealing

There are no equipment limitations for accelerated paving, but the scheduling of sawing and sealing must be modified to be consistent with the accelerated rate of construction and project opening.[2,11] Joints should be sawed as soon as possible in order to prevent uncontrolled cracking, usually within 3 to 4 hours. However, there are many factors that influence the most appropriate time for joint sawing operations, including mix design, ambient temperatures, and coarse aggregate type (shape and hardness). Table 4-6.2 provides recommended compressive strengths needed to begin sawing for different coarse aggregate properties.[11]

After joint sawing operations, some delay may be needed before the sealants can be placed. Dry reservoir sidewalls are desirable for most currently used joint sealants. Some of the sealants that have worked well on previous projects include low-modulus polymer sealants and silicone sealants. Preformed compression sealants have not been used in accelerated paving projects to date, but they may be ideal for this application. The preformed sealants are not highly sensitive to dirt or moisture on the joint faces.

Table 4.6.2. Recommended compressive strength for initiation of concrete sawing operations.[11]

Coarse Aggregate Shape	Coarse Aggregate Hardness	Cement Content of Mix (kg/m^3)	Compressive Strength for Acceptable Cut With Some Raveling (MPa)	Compressive Strength for Excellent Cut (No Raveling) (MPa)
Crushed	Soft	300 385 475	2.5 2.2 1.9	3.9 3.7 3.4
Crushed	Hard	300 385 475	4.9 4.8 4.7	7.0 6.8 6.6
Rounded	Soft	300 385 475	1.4 1.0 1.0	2.5 2.1 1.8
Rounded	Hard	300 385 475	3.3 3.1 2.9	4.9 4.8 4.6

6. EARLY OPENING TO TRAFFIC

A number of initiatives are being followed to improve strength monitoring of in-place repair material to move away from curing time as the criteria for opening PCC repairs to traffic. Because strength development of concrete is so sensitive to local conditions, such as ambient temperature and humidity, it is desirable to base the opening of concrete repairs to traffic on the actual, in-place strength of the repair rather than curing time. Several approaches to determining the in-place strength of PCC have been explored in recent studies. A listing of various nondestructive test (NDT) methods for evaluating the strength of in-place concrete is provided in table 4-6.3.[2] Some of these techniques also provide quality control, by providing assessment of heat development and effectiveness of curing. For any given NDT approach, proper correlation curves should be developed (i.e., strength versus NDT test result) to relate the strength of the concrete to the given NDT test result.

Another means of monitoring the strength of in-place concrete is through the use of temperature-matched curing. In this methodology, cylinders are cast and kept at the same temperature as the field concrete, and can provide a more reasonable estimate of the strength of the actual concrete in the field. The strength of in-place materials can be significantly higher than that of standard cured cylinders because of the retained heat in the repairs. In one study, Sprinkel reports that compressive strengths for temperature matched cylinders are more than twice those of air-cured cylinders.[12] Although the use of standard cured cylinder strengths is more conservative, this would not result in the earliest possible opening. For early strengths, concrete cylinders cured in an insulated box also give a good estimate of the in-place strength.

Table 4-6.3. Nondestructive test methods for concrete.[2]

Test Method	Standard	Basic Description	Testing Precision to Baseline Cylinder Strength
Surface Hardness (Swiss Hammer)	ASTM C 805	Rebound of hammer correlates to surface hardness and compressive strength	+ 40%
Penetration (Windsor Probe)	ASTM C 803	Penetration depth of gun-fired probe correlates to surface hardness and compressive strength	+ 20%
Pullout	ASTM C 900	Force to remove cast-in metal probe correlates to surface compressive strength	+ 15%
Break-Off	ASTM C 1150	Force necessary to break a circular core cast or cut partially into slab correlates to flexural strength	+ 15%
Maturity	ASTM C 1074	Internal temperature of concrete relates directly to concrete strength	+ 5%
Pulse Velocity	ASTM C 597	Velocity of sound wave from transducer to receiver through concrete relates to concrete strength	+ 10%

Strength Monitoring

The use of innovative nondestructive testing techniques has been advocated by the FHWA.[1] At this time, the maturity and pulse velocity approaches are the most common and appear to provide reasonable estimates of insitu rigid pavement strengths.

Maturity

Maturity, using the Nurse-Saul approach, is the accumulated product of time and temperature. Maturity may also be determined using the Arrhenius method, which accounts for nonlinearity in the rate of cement hydration, but this approach is less commonly used. Both maturity functions are outlined in ASTM C 1074, and they give comparable results.

The Nurse-Saul method calculates the time-temperature factor using the following equation:

$$M(t) = \sum (T_a - T_0)\Delta t \qquad (4\text{-}6.1)$$

where: M(t) = temperature factor, degree-days or degree-hours
T_a = average concrete temperature during time interval, °C
T_o = datum temperature, °C (typically -10 °C)
Δt = time interval, days or hours

The degree-hours is the product of each time interval multiplied by the number of degrees by which the average temperature exceeded the datum temperature during that time interval.[1] The datum temperature is considered to be the temperature below which hydration of the cement will no longer occur, and will vary with the composition of the concrete.

In order to use the maturity approach, laboratory testing of the actual field concrete mixes is required in order to develop a relationship between the compressive strength and the temperature time factor. An example of such a laboratory-derived maturity curve is shown in figure 4-6.2.[2] A relationship such as that shown in figure 4-6.2 becomes the calibration curve for evaluating the field concrete strength. However, should the mix design change in any way, a new calibration curve will have to be developed.

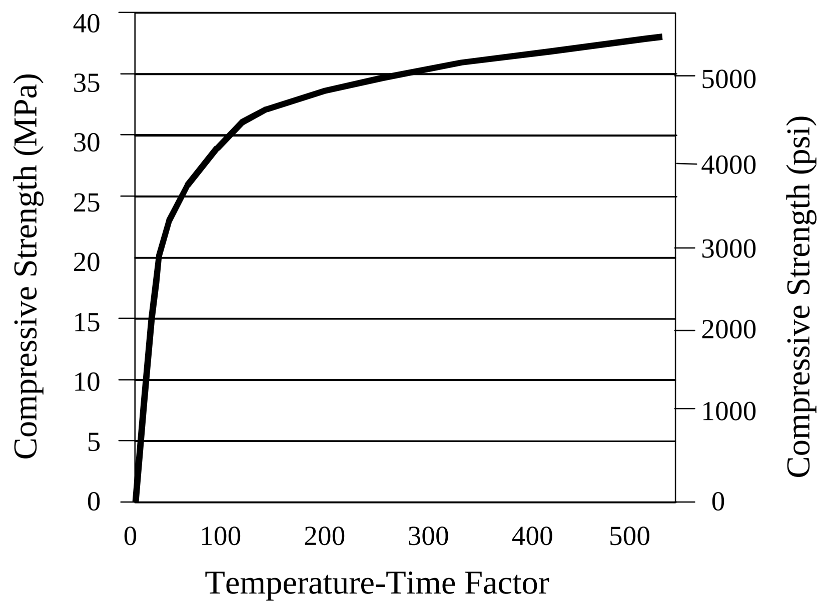


Figure 4-6.2. Example maturity curve.[2]

In the field, the temperature of the concrete is monitored using temperature probes or thermocouples embedded in the pavement and hooked to a data logger or maturity meter. Temperature readings are then taken at regular intervals and used in the maturity curve to estimate the concrete strength.

For example, assume the required compressive strength for opening a pavement to traffic is 20.7 MPa. The curve in figure 4-6.2 shows that this corresponds to a temperature-time factor of approximately 50 degree days. Thus, when the combination of time and temperature from the data logger or maturity meter indicate a maturity of 50 degree days, the pavement can be opened to traffic.

Pulse Velocity

Pulse velocity uses short pulse sound waves through plastic concrete to determine in-place strength, taking into account principles of sound wave transmission through water and aggregates. A special device is used that imparts the wave and transducers spaced at a set distance from the sender receive the signals. The velocity of the wave propagating through the concrete correlates to the strength of the concrete.

A typical plot of pulse-velocity curve is shown in figure 4-6.3. As with the maturity approach, the curve must be developed in the laboratory with the specific project materials. If any mix component changes, a new calibration curve should be developed.

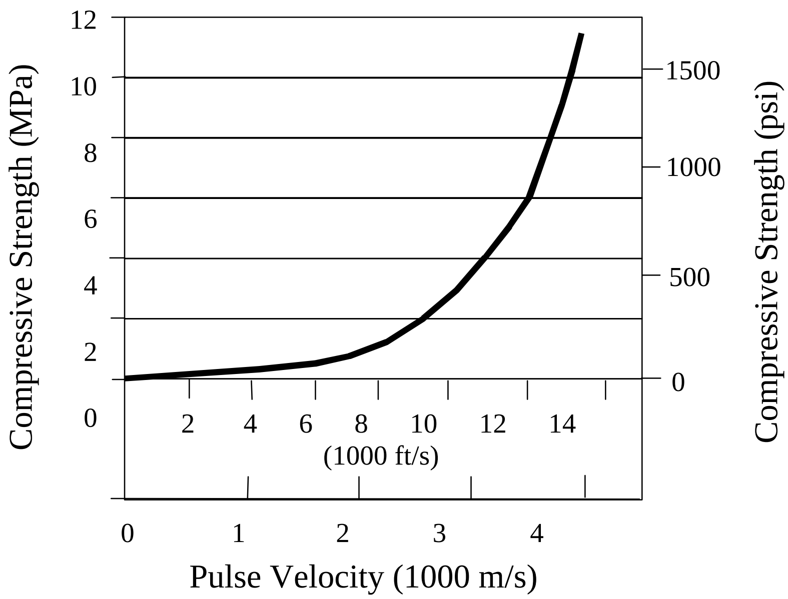


Figure 4-6.3. Example pulse velocity curve.[2]

Strength Required for Opening to Traffic

Currently, there is no clear consensus on what strength is required before opening concrete repairs to traffic to assure adequate long-term performance. Davis and Darter[13] recommend values based on slab thickness, ranging from a minimum flexural strength of 2.6 MPa for 180-mm-thick slabs to 1.4 MPa for 25- mm-thick slabs. A National Cooperative Highway Research Program (NCHRP) study states that common practice is to require a flexural strength in the range of 2.1 to 2.4 MPa.[14] A recent study on the early opening of concrete full-depth repairs suggests a minimum flexural strength of 1.7 MPa.[4] These values all seem to be in the range of what most highway agencies are currently using.

Although most agencies use flexural strength as the strength measure to open the pavement to traffic, the use of compressive strengths can be considered. For example, a study in Britain suggests that compressive strengths can be used and would fit better with their local practices.[15]

7. CONCLUSIONS

Accelerated rigid paving techniques use a combination of special PCC mixes and innovative construction procedures to provide rehabilitation solutions for rigid pavements under rapid traffic re-opening requirements. Fast track paving is only one of several techniques that have been proven successful in this area, but the concept also extends to other rigid pavement rehabilitation procedures. The following summarizes the important aspects of this module:

1. A number of techniques can be used on concrete rehabilitation projects to allow the pavement to be opened to traffic within 4 to 8 hours. These include proper construction sequencing, the use of suitable mixes and curing, and an understanding of strength gain based on actual conditions.

2. The current state of the art in concrete repair materials is such that virtually any opening time requirement (from less than 1 to 24 hours or more) can be met. In general, the faster setting the mix, the higher the cost will be and the more special handling requirements it will have. By adopting a simple rule of selecting the least exotic material that will do the job, the selection of materials that would unnecessarily complicate the construction process will be avoided.

3. By far, PCC mixes are the most widely-used materials today for accelerated paving of rigid pavements. High early strength can be achieved with most conventional materials by increasing the cement content and reducing the water/cement ratio of PCC mixes.

4. For partial-depth spall repairs, the use of proprietary patching materials may be justified to achieve very rapid opening (under 2 hours). A wide variety of proprietary patching materials are available, but these materials are generally too expensive for use in other applications.

5. Because strength development is highly sensitive to local conditions such as ambient temperature and humidity, the opening of concrete repairs to traffic should be based on the actual, in-place strength of the repair, rather than the elapsed curing time. Several methods are available for determining the in-place strength of PCC, including temperature-matched curing, maturity, and pulse-velocity, with the latter two being most widely used.

6. The strength required at opening to ensure adequate performance has not been well established, with values of the minimum flexural strength ranging from 14 MPa to 2.6 MPa.

While there is still a need for further research and refinement of many of the techniques presented in this module, there is a wealth of information available today that can be used to successfully implement some of these technologies in rigid pavement rehabilitation projects. As more and more long-term performance data become available, these technologies should receive even more widespread acceptance and usage.

8. REFERENCES

1. Federal Highway Administration (FHWA). 1994, "Accelerated Rigid Paving Techniques—State-of-the-Art Report," FHWA-SA-94-080, Federal Highway Administration, Washington, DC.

2. American Concrete Pavement Association (ACPA). 1994, "Fast-Track Concrete Pavements," Technical Bulletin TB004.02, American Concrete Pavement Association, Skokie, IL.

3. Good-Mojab, C. A., A. J. Patel, A. R. Romine. 1993, "Innovative Materials Development and Testing, Volume 5—Partial-Depth Spall Repair," SHRP H-356, Strategic Highway Research Program, Washington, DC.

4. Whiting, D. M. Nagi, P. A. Okamoto, H. T. Yu, D. G. Peshkin, K. D. Smith, M. I. Darter, J. Clifton, and L. Kaetzel. 1994, "Optimization of Highway Concrete Technology," SHRP-C-373, Strategic Highway Research Program, Washington, DC.

5. Jones, K. 1988, “Special Cements for Fast Track Concrete,” Report No. MLR-87-4, Iowa Department of Transportation, Ames, IA.

6. Hallin, J. P. 1990, “Concrete Pavement Restoration as a Rehabilitation Strategy,” Proceedings, Workshop on Evaluating Portland Cement Concrete Rehabilitation Strategies, Federal Highway Administration, Washington, D.C.

7. Green, R. 1990, “Rehabilitation of Concrete Pavements in Ohio,” Proceedings, Workshop on Evaluating Portland Cement Concrete Rehabilitation Strategies, Federal Highway Administration, Washington, DC.

8. Grove, J. D., K. Jones, K. Bharil. 1993, “Fast Track and Fast Track II: Cedar Rapids, Iowa,” Report No. HR-544, Iowa Department of Transportation, Ames, IA.

9. Klemens, T. L. 1990, “When Slab Replacement Resembles a Production Line,” Highway and Heavy Construction, October 1990.

10. Tayabji, S. D. and P. A. Okamoto. 1987, “Field Evaluation of Dowel Placement in Concrete Pavements,” Transportation Research Record 1110, Transportation Research Board, Washington, DC.

11. Okamoto, P. A., P. J. Nussbaum, K. D. Smith, M. I. Darter, T. P. Wilson, C. L. Wu, and S. D. Tayabji. 1991, “Guidelines for Timing Contraction Joint Sawing and Earliest Loading for Concrete Pavements, Volume I—Final Report,” FHWA-RD-91-079, Federal Highway Administration, Washington, DC.

12. Sprinkel, M. M. 1991, “Applications of High Performance Concretes,” Proceedings, Strategic Highway Research Program Specialty Conference, American Society of Civil Engineers/Federal Highway Administration, Denver, CO.

13. Davis, D. D. and M. I. Darter. 1989, “Early Opening of Concrete Pavements to Traffic,” Proceedings, Structural Materials at the ASCE Structures Congress, American Society of Civil Engineers, New York, NY.

14. Transportation Research Board (TRB). 1989, “Rapid Replacement of Portland Cement Concrete Pavement Segments,” NCHRP Research Results Digest Number 169, Transportation Research Board, Washington, DC.

15. Franklin, R. E., B. J. Walker, and P. M. Hollands. 1992, “Fast Track Concrete Paving: Laboratory and Full-Scale Trials in United Kingdom,” Research Report 355, Transport Research Laboratory, Crowthorne, Berkshire, Great Britain.

MODULE 4-7

SLAB STABILIZATION AND SLAB JACKING

1. INSTRUCTIONAL OBJECTIVES

This module covers both stabilization of slabs to restore support and jacking of slabs to level up settled areas. The participants will be able to accomplish the following upon successful completion of this module:

1. State the purpose for and discuss the importance of slab stabilization.
2. Describe the cement grout mixtures and asphalt cements that have been used for slab stabilization and slab jacking and the problems associated with the use of each.
3. Describe procedures for performing slab stabilization, including how to locate areas that need slab stabilization, typical hole patterns, and how to determine if slab stabilization has been effective.
4. List the steps required to estimate the amount of cement grout required for a given project.
5. List the typical construction procedures for slab stabilization and slab jacking.

2. INTRODUCTION

Loss of support from beneath rigid pavement slabs has long been recognized as a major contributor to the accelerated deterioration of the pavement. If significant pumping has occurred and slab support is lost, more significant distresses such as joint faulting and corner breaks can develop. If a hot-mix asphalt (HMA) overlay is placed on such a deteriorated rigid pavement, high deflections at joints will occur that will lead to the development of severe reflection cracks. In many cases, slab stabilization is an effective rehabilitation measure that can prolong the service life of a rigid pavement exhibiting loss of support. In a study reported in 1994,[1] 16 States were using undersealing as a rehabilitation strategy.

Slab settlements sometimes occur on rigid pavements in areas of poor 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 through slab jacking.

Both slab stabilization and slab jacking of rigid pavements or HMA overlays of rigid pavements have been performed for many years; however, within the past 10 years they have become highly specialized operations with improved materials, specially trained personnel, and special equipment. Although these techniques are commonly used, there is some question as to the effectiveness of slab stabilization, with at least one agency currently maintaining a moratorium against this operation.[2]

3. DEFINITIONS

Slab stabilization is the pressure insertion of a material beneath the slab and/or stabilized subbase. The material is used to both fill voids beneath the slab and to provide a thin layer that reduces deflections and resists pumping action. Various terms have been used to describe this process, including pressure grouting, undersealing, and subsealing. It should be emphasized that slab stabilization is used to fill existing voids beneath the pavement and not to raise the slab.

The actual lifting or raising of a portland cement concrete (PCC) slab is referred to as slab jacking. It is used to level out a localized area of depression or settlement in a concrete pavement, thereby restoring rideability. However, slab jacking should not be used to correct faulting.[3]

4. PURPOSE AND PROJECT SELECTION

Slab Stabilization

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.[4,5] It is important that slab stabilization be performed prior to the onset of pavement damage due to loss of support to be most effective.[5]

The following techniques have been used to determine whether loss of support has occurred beneath a rigid pavement surface:

- Visual distress data. Faulting of transverse joints and cracks, pumping, and corner breaks all indicate that loss of support has occurred. Figure 4-7.1 shows the progression of deterioration in non-doweled rigid pavements.[6] Ideally, slab stabilization should be conducted at the third stage, after the formation of the void but before excessive faulting and cracking have occurred.

- 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. A few deflection-based void detection methods are available, and have been found to be effective by a number of highway agencies.[6]

- Other NDT methods. Other nondestructive testing methods have been used for void detection, including ground penetrating radar (GPR) and infrared thermography. California has found GPR to be a very promising alternative for the detection of voids beneath rigid pavements.[2]

The following concurrent rehabilitation activities should be strongly considered whenever slab stabilization work is performed:

- Provide positive subdrainage to increase the drainability of the structural section.

- Seal all existing joints and cracks to minimize the amount of water infiltrating into the structural section.

- If poor load transfer exists at the transverse joints, install load transfer devices to reduce corner deflections.

- Perform diamond grinding of project to restore rideability.

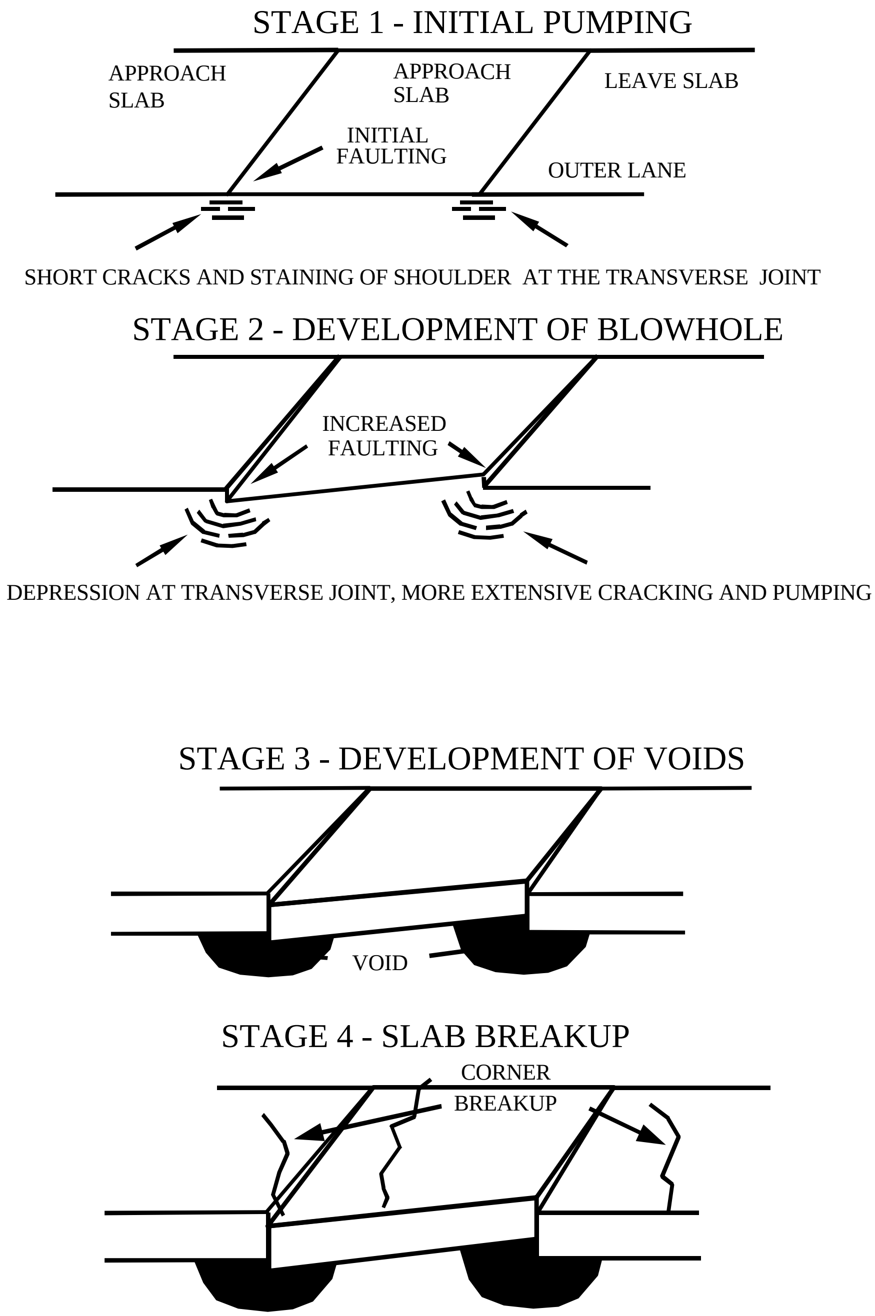


Figure 4-7.1. Typical stages in the deterioration of a concrete pavement.[(6)]

Slab Jacking

Slab jacking consists of pumping a stiff cement grout, under pressure, beneath the slab/subbase, slowly raising the slab until it reaches a smooth profile. This operation is required where the pavement has settled due to loss of support. Settlements can occur anywhere along a pavement profile, but most usually are associated with fill areas, over culverts, and at bridge approaches.

Ideal projects for slab jacking are pavements that exhibit localized areas of settlement. In such cases, the settled slab can be slowly raised back to its original elevation through the injection of grout beneath the slab. However, slab jacking is not recommended for repaired faulted joints; this is more effectively addressed through the use of diamond grinding.

5. LIMITATIONS AND EFFECTIVENESS

Slab Stabilization

The effectiveness of slab stabilization can be determined only by monitoring the subsequent performance of the pavement. The best early indication of the effectiveness is obtained by remeasuring the slab deflection after grouting and determining if the deflection has reduced to the point of full support. This procedure is illustrated in figure 4-7.2, in which an falling weight deflectometer (FWD) load versus corner deflection plot shows the change in the leave corner load-deflection curve after grouting, indicating that full support exists under the slab corner.[6,7]

One agency determines the effectiveness of slab stabilization by remeasuring the deflection after the initial stabilization.[8] If the deflection under an 80-kN single-axle load is still in excess of 0.6 mm, the slab is regrouted, with the assumption that the existing voids were not entirely filled, or additional voids were formed during the initial stabilization operation. This procedure is only repeated once. Other agencies use different deflection methodologies and may regrout up to two times, after which the slab or section is removed and replaced.

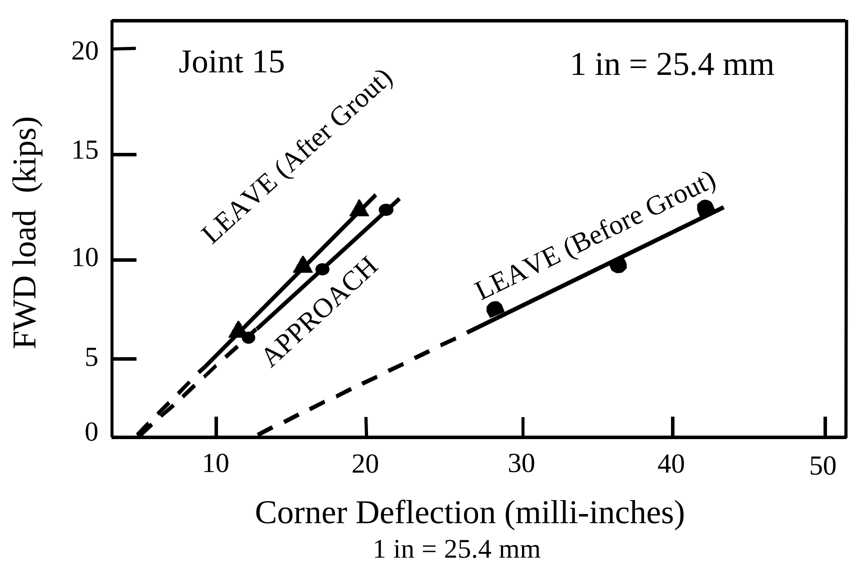


Figure 4-7.2. Example of load versus deflection plot before and after slab stabilization.[6,7]

Several research studies have shown that slab stabilization is effective at filling voids in the pavement. For example, a study in Illinois showed that undersealing of continuously reinforced concrete pavements (CRCP) with both cement grout and asphalt cement prior to the placement of an HMA overlay was effective in reducing deflections and subsequent reflection cracking.(9) An Indiana study showed that asphalt slab stabilization remained effective in restoring slab support after 3 years of service.(10) A North Carolina study indicated that slab stabilization was effective at restoring support when performed properly.(5) However, that same study showed that the improper or unwarranted use of slab stabilization could result in slab cracking and the introduction of incompressibles into the joints.(5)

The results of a recent survey of highway agencies indicated that 16 of 33 responding agencies are currently using slab stabilization as part of their concrete rehabilitation activities.(1) Those that do not use slab stabilization cite the following reasons:(1)

1. The difficulty of locating voids beneath the slab.
2. The difficulty of filling all voids.
3. The damage to the pavement that may be caused by overgrouting.
4. The dependence of undersealing on the experience and skill of the contractors.
5. The lack of cost-effectiveness of slab stabilization.

The survey indicated that those agencies using slab stabilization rely on long-term monitoring of the concrete panels to assess the effectiveness of the slab stabilization operations.(1) No information was provided on the perceived long-term effectiveness of slab stabilization.

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 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 depressed areas.(11)

6. DESIGN CONSIDERATIONS

Materials for Slab Stabilization

The most common materials that have been used in conventional slab stabilization activities are asphalt cement and cement grouts. Other materials have also been employed, including limestone dust-cement grouts and polyurethane and silicone rubber foam.(1)

The material chosen for slab stabilization must be able to penetrate into very thin voids and have the strength and durability to withstand pressures caused by traffic, moisture, and temperature. Although asphalt cements were used more extensively in the past for slab stabilization, they have been largely replaced by pozzolanic-cement grouts. However, Indiana has continued their use of asphalt stabilization through the years and reports good results.(1,12,13)

<u>Asphalt Cement</u>

The type of asphalt cement commonly used for slab stabilization must have a low penetration (e.g., 15 to 30) and a high softening point (e.g., 82 ºC to 93 ºC). It must also have a viscosity suitable for pumping when heated to temperatures from 204 ºC to 232 ºC. The use of a regular paving grade asphalt cement could lead to large amounts of asphalt extruding out of joints onto the pavement surface. The

Asphalt Institute recommends the use of asphalt cements that meet the requirements of AASHTO Designation M 238 (ASTM Designation D 3141) for undersealing.[14]

Cement Grout Mixtures

Many different grout mixtures have been used for slab stabilization, including pozzolanic-cement grout and limestone-cement grout. According to a recent survey, pozzolanic-cement grout is the most popular material currently being used by highway agencies.[1] The materials that make up the grout greatly affect the consistency, strength, and durability of the mixture, and thus have a large impact on the success of the grout stabilization operation.

A grout mixture used for slab stabilization must be fluid enough to flow into very small voids, and then develop adequate strength and durability to resist load and climatic effects. Typical flow cone times of 9 to 16 seconds are required (for comparison, water has a flow cone time of 8 seconds). The pozzolanic-cement (or flyash-cement) grout mixtures provide the best flowability for filling small voids due to the spherical shape of the flyash particles. An Illinois field comparison study concluded that flyash grouts were more “flowable” and produced a stronger material than limestone grouts due to the spherical shape of flyash and the pozzolanic reactions that occur.[15]

Materials for Slab Jacking

Cement grout has been used almost exclusively for slab jacking operations. However, the material is slightly stiffer than that used for slab stabilization procedures, generally having a flow time of 16 to 26 seconds.

7. PAVEMENT SURVEYS

Deflection testing is the most common approach currently in use for locating and estimating the size of voids. Many States use a maximum corner deflection criteria to determine if a void is present. Table 4-7.1 summarizes the maximum corner deflection value triggering the need for slab stabilization as reported by selected States. However, specifications based on a single corner deflection may not always provide reasonable estimates of the presence of a void because variation in load transfer from joint to joint can cause considerable variation in corner deflections.

Another deflection-based method of identifying the presence of voids measures and plots the profile of both the approach and leave corner deflections.[6] An example of this procedure is shown in figure 4-7.3, in which deflection measurements are recorded at a constant load at both the approach slab corner and the leave slab corner.[6] 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.[6] The procedure recommends the identification of a corner deflection value above which slab stabilization is warranted. For example, in figure 4-7.3, a reasonable value might be 0.50 mm.

When using this approach, it is important that the same equipment and loads are applied in the initial evaluation to determine void location and to rate the effectiveness of the stabilization operation. The selected “trigger” corner deflection value should be reevaluated after a few days of construction to assess its practicality and modifications should be made to reflect the additional data collected. The main weaknesses of this approach are that little information is provided related to the size of the voids and that the corner deflection is highly dependent upon the degree of joint load transfer.

Table 4-7.1. Maximum corner deflection criteria used by selected States for assessing the presence of voids.[1]

State	Maximum Corner Deflection, mm
South Dakota	0.25
Florida	0.38
Pennsylvania	0.50
Oregon	0.64
Georgia	0.76
Texas	0.50
Washington	0.89

Another void detection method is based on measuring the magnitude of the corner deflection at three different load levels.[4] Typically, load levels of 27, 40, and 63 kN are used to develop load versus deflection plots for each test location, similar to that shown in figure 4-7.2.[4] A load versus deflection plot that does not pass through the origin, such as that representing the leave slab prior to stabilization in figure 4-7.2, indicates that a void likely exists at that location. Load versus deflection plots that pass through or near the origin indicate that no loss of support exists. This method of void detection can be conducted concurrently with the stabilization operation, providing close coordination between the void detection and slab stabilization operations. However, because of variations in load transfer, it cannot be used to determine the size of the void.

A procedure has recently been developed for verifying the presence of a void as indicated by nondestructive testing (NDT) methods. This procedure, termed the epoxy/core test method, uses the following procedure to confirm the existence of a void:[16]

1. A 25- to 50-mm diameter hole is drilled in the slab at the suspected void location to a depth of about 25 mm into the subbase.

2. A low viscosity, two-part epoxy is colored with dye and poured into the hole. The epoxy percolates down, saturating the subbase and subgrade and fills any void that might be present.

3. After allowing the epoxy to harden, a core is taken such that it slices the access hole and cross sectioning the subbase/pavement interface.

4. The core is examined to note the areas filled by the epoxy as a means of confirming the existence of a void.

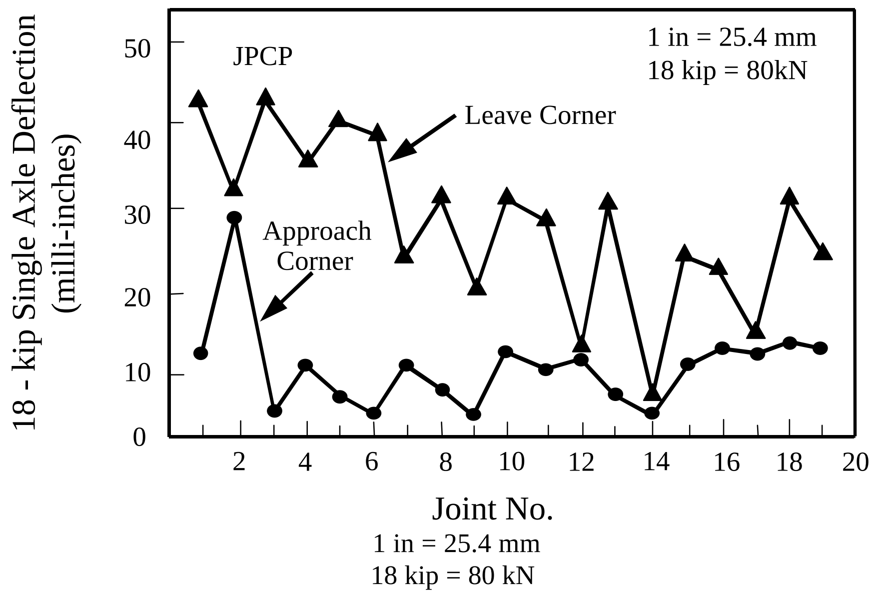


Figure 4-7.3. Example profile of corner deflections.[6]

An investigation comparing the results of deflection testing results and the epoxy/core method found that the NDT methods do not effectively make the distinction between voids and poor support.[16] The study does not suggest that current NDT methods should be abandoned, but rather that correlations be developed between deflection testing results and epoxy/core findings.[16]

When conducting a void analysis survey, it is important to assess the percentage of joints and cracks suffering loss of support. It has been found that this percentage can vary from 10 to 90 percent from project to project. Visual evidence of pumping is directly related to the percentage of joints and cracks affected, with pavements suffering extensive pumping most likely to have the highest percentage of voids.[6] Information may also be gleaned from lifting a slab or two in a project, if for no other reason than to provide visual feedback on the condition of the base.

8. COST CONSIDERATIONS

The cost of typical slab stabilization operations depends on the quantity of material pumped beneath the slab. Methods for estimating these quantities and determining the resultant costs are described in this section.

Asphalt Cement Stabilization

Typical Asphalt Quantities

The amount of asphalt pumped per hole depends on the extent of pumping and voids beneath the slab. The Asphalt Institute estimates that where only minor voids exist and no serious pavement pumping exists, as little as 38 liters of asphalt will be needed per hole.(14) For projects with extensive pumping, the quantity may range from 114 to 152 liters per hole. Results from Indiana show that 57 to 68 liters per hole is required for jointed concrete pavements (JCP) and 38 liters per hole is required for CRCP.(12)

Costs of Asphalt Slab Stabilization

The cost of asphalt slab stabilization will vary from region to region, and is highly dependent on the size of the voids, the number of voids, and the joint spacing. Typical costs in Indiana in 1986 were averaging about \$25 per hole for JCP and \$17 per hole for CRCP. This translates to a cost of approximately \$0.54/m^2 to \$0.60/m^2.(12)

Cement Grout Stabilization

Estimating Cement Grout Quantities

The amount of grout that is required to fill voids and stabilize slabs is, of course, highly dependent upon the pavement condition, particularly the amount of existing pumping. Unfortunately, it is also highly dependent upon the grouting crew, which is able to force excessive grout into almost any hole if not closely monitored.

One successful procedure for estimating the required grout take is as follows:(4,6)

1. Select a representative section of pavement 57- to 95-m within each kilometer of the project and measure corner deflections with the FWD. Apply a sequence of three load magnitudes such as 40 kN, 53 kN, and 67 kN to each corner.

2. Plot the load versus deflection data on graphs, as shown in figure 4-7.2, and determine the proportion of joints that have loss of support (e.g., 25, 56, 90 percent).

3. Compute the overall estimated dry grout quantities needed to fill the voids and stabilize the slabs using the following expression:

$$\text{GROUT} = \text{PJG} \times \text{AGT} \times \text{TNJ} \qquad (4\text{-}5.1)$$

where:

GROUT	=	Total grout quantity for project (m^3 of dry materials)
PJG	=	Proportion of joints requiring grouting
AGT	=	Average grout take per joint grouted (typically 0.031 to 0.093 m^3)
TNJ	=	Total number of joints in project

For example, if a project is tested and the proportion of joints that have loss of support is 35 percent and the joint spacing is 9.3 m, the total quantity of grout estimated for the 8 km project is estimated as follows:

$$\begin{aligned}\text{GROUT} &= 0.35 \times 0.61 \times (5 \times 1609.3)/9.3 \\ &= 17.4 \text{ m}^3 \text{ of dry grout}\end{aligned}$$

It may be wise to increase this quantity by 10 to 20 percent to allow for some over run and for regrouting of some joints.

$$17.4 \text{ m}^3 \times 1.2 = 20.9 \text{ m}^3 \text{ of dry grout}$$

If deflection data are not measured, and every joint is grouted, the total grout take could be twice as high, with the extra amount wasted on joints where there is no loss of support. Many projects have been over grouted, resulting in broken slabs and wasted resources.

Costs of Cement Grout Slab Stabilization

The cost of cement grout stabilization is related directly to the number and size of the voids present. This cost will vary from region to region, with typical average values ranging from about $1.08/m^2 to $1.20/m^2.

9. CONSTRUCTION

The success of asphalt or cement grout slab stabilization operations is highly dependent upon the skill of the contractor. Thus, there should be an experience clause requiring that the contractor has successfully completed similar work. The following discussion describes the typical construction sequence.

Asphalt Cement Slab Stabilization

Hole Patterns

After identifying the locations requiring slab stabilization, the proper hole pattern is chosen. The hole patterns used on different projects have varied. For example, a CRCP project conducted on I-65 in Indiana experimented with two rows of holes, staggered every 1.2 m across the outer lane in areas of high deflections and each side of potential failed areas or patches.(17) After several sections were undersealed, it was concluded that "just one row of holes on 2.4-m centers in the centerline of the right-hand lane was needed because the asphalt was traveling across and coming out the staggered holes."(17) This pattern resulted in a continuous layer of asphalt, 3 to 9 mm thick, beneath the slab in the outer lane.

Another CRCP undersealing project on I-57 in Illinois included holes spaced on 3-m centers, 1.2 m from the centerline in a staggered pattern. A nearly continuous layer of asphalt cement was present along the edge of the slab. With this pattern, asphalt was not often forced up in the adjacent holes, indicating good coverage with a minimum of overlap.(18)

Other projects have employed holes spaced at 2- to 4-m intervals longitudinally along the centerline of the lanes. The staggering of holes provides that the longitudinal distance between two holes in two adjacent lanes ranges between 1.2 to 1.8 m. Holes should not be placed closer than 1 m from an existing joint or working crack. For stabilizing slab corners, holes should be placed about 1 m on each side of the joint and 1 m from the edge of the slab closest to the corner.(14)

Stabilization Operations

Some projects require that compressed air at approximately 480 kPa be blown through the holes for 15 to 60 seconds before pumping asphalt. This procedures blows out water that exists beneath the slab. Excessive water beneath the slab rapidly cools the asphalt cement, resulting in inadequate filling of the voids.[14] If adequate drying cannot be obtained, either subsealing should be postponed, or higher asphalt cement temperatures and pressures should be used and the entire operation performed at a rapid rate.[14]

The following sequence is typical for an asphalt stabilization operation:

1. The asphalt is heated to the desired temperature (between 204 ºC to 232 ºC) before pumping operations begin. Because these temperatures approach and may exceed the flash point of some asphalts, extreme care must be exercised to ensure that the asphalt does not contact an open flame. The asphalt is circulated prior to pumping in order to free and warm up the lines of the circulating hose.

2. Water, lime water, or sand is sprinkled around the hole prior to pumping to prevent any asphalt that may leak out from around the nozzle from sticking to the pavement. If asphalt seeps from the joints or cracks before the undersealing is completed, the pumping is stopped until the extruded asphalt has congealed. The use of cold water sprayed on seeping asphalt will accelerate the hardening process.

3. The holes are blown out and the asphalt nozzle is firmly wedged into the hole. Asphalt is pumped at a pressure between 170 and 620 kPa until the underside of the slab is sealed and all cavities filled, as indicated by the shoulders showing signs of breaking away from the pavement edge, or when the pavement begins to rise. Indiana recommends that asphalt cement pumping is halted for JCP when the pavement rises 6.3 mm, or when 15 seconds of pumping time has elapsed (for CRCP, 3.2 mm of rise or 12 seconds of pumping time has elapsed is recommended).[12]

4. After pumping has been completed, the nozzle is removed and the hole temporarily plugged with a cylindrical wooden plug.

5. When the asphalt has hardened, the temporary plug is removed and the hole filled with a cement grout. All asphalt and any other materials spilled on the surface are removed.

6. Asphalt is not normally pumped when the atmospheric temperature is below 2 ºC, or when the foundation material is frozen.[14]

Cement Grout Slab Stabilization

<u>Hole Patterns</u>

After the specific areas of slab stabilization have been located, the actual grouting can begin. The first step is selection of an appropriate hole pattern and depth, depending on type and design of concrete pavement.

- On jointed concrete pavements, the typical hole patterns used in the outer truck lane are illustrated in figure 4-7.4. However, the hole pattern may need to be adjusted during construction depending upon the results obtained.

- On jointed reinforced concrete pavements (JRCP), intermediate cracks of medium to high severity that are not scheduled for full-depth repair should be treated in the same manner as joints.

- A typical hole spacing for CRCP is shown in figure 4-7.5.[18] It is recommended that this pattern be adjusted after initial field trails are completed to provide the best coverage.

The hole should be drilled or cored just beyond the bottom of the slab when a granular subbase is present, and to the bottom of the subbase if it is stabilized. Voids often exist beneath the stabilized subbase, and it is important that these be filled. If a pneumatic drill is being used, the downward pressure on the drill should not be excessive, or severe spalling at the bottom of the slab may result

A quick check of whether or not the hole should be grouted may be made by pouring water into the drill hole. If it does not take water there is no void and therefore no need to grout. The hole can be plugged and the operation can proceed to the next hole.

<u>Mix Design</u>

The following is a recommended mix design for a pozzolanic-cement grout for use in slab stabilization:

1. One part by volume portland cement type I or type II (type III may be specified if there is a need for early strength).
2. Three parts by volume pozzolan (natural or artificial).
3. Water to achieve required fluidity.
4. If ambient temperatures are below 10 $^{\circ}$C, an accelerator may be used, subject to approval of the contracting agency.
5. Additives, superplasticizers, water reducers, and fluidifiers as needed.

Laboratory analysis has shown that pozzolans obtained from different sources can react quite differently to the same additives.[6] Compatibility between the pozzolan and the cement must be demonstrated prior to use.

Additives are available that increase the flowability of the grout. For example, powdered ammonium lignin sulphonate will increase the fluidity of the grout without adding water. Thus, a stronger, denser grout is obtained without increasing the cement content.[19] A superplasticizer additive to a flyash grout also produces a mix achieving high flow rates.[15]

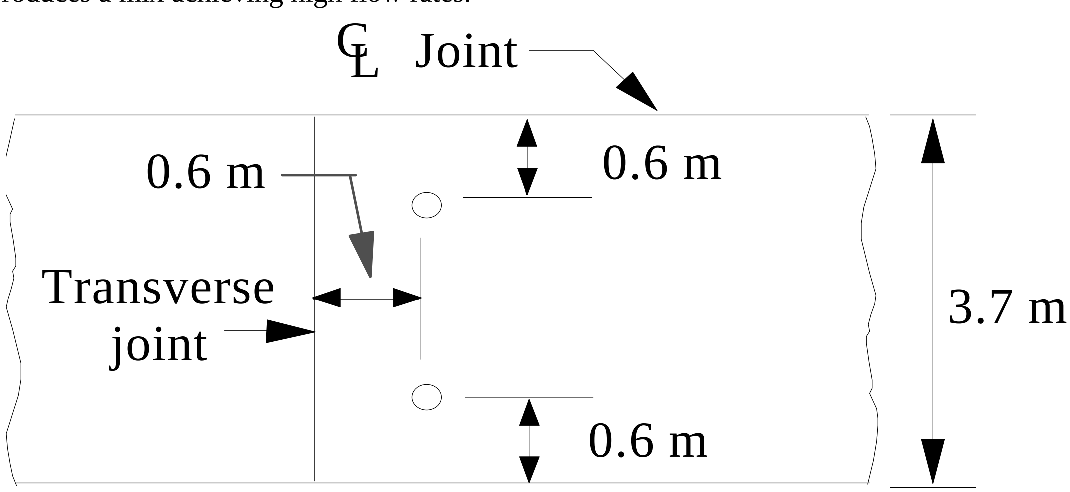


Figure 4-7.4. Typical hole pattern for jointed concrete pavements.

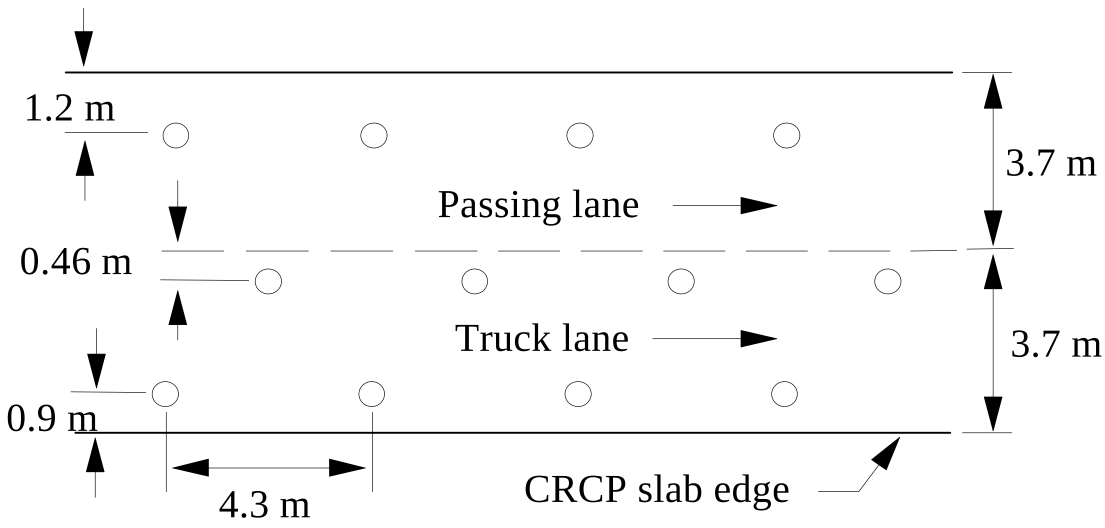


Figure 4-7.5. Hole pattern used on a continuously reinforced concrete pavement.

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. The following material recommendations are offered based on the guide specifications developed by Darter et al.[6]

- Portland cement (type I, type II, or type III) shall conform to AASHTO M 85 and shall be tested for compatibility with the pozzolan, if one is to be used.

- Pozzolans shall conform to the requirements of ASTM C 618, and limestone dust shall comply with AASHTO M 17 for mineral fillers. Limestone dust must be spherical in shape, and dust containing primarily flat platelet or rhombohedral-shaped grains will be rejected.

- Fluidity shall be measured by the Corps of Engineers flow cone method (CRD-C-611). As previously indicated, flyash-cement grout time of efflux shall be within 10 to 16 seconds and limestone-cement grout time of efflux shall be within 16 to 22 seconds.

- Compressive strength shall be measured in accordance with AASHTO T 106. The 7-day compressive strength shall not be less than 4140 kPa.

Stabilization Operations

After the holes are drilled, an expanding rubber grout packer, which is connected to the discharge hose on the pressure grout pump, is lowered into the holes. The discharge end of the packer is not extended below the lower surface of the concrete slab.

The purpose of grout slab stabilization is to fill existing voids, and not to raise the slab above grade. 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. Such an event occurred on a project in Washington State, in which excessive grout raised the joints while leaving the center of the slab unsupported; this resulted in several fatigued and settled slabs.[20]

Slab uplift should not be allowed to exceed 3.2 mm total for any given slab corner. A maximum sustained pumping pressure of 690 kPa or less is generally allowable, with surge pressures as great as 1390 kPa tolerated as pumping begins.[6] Normally, indication that the slab is beginning to rise or grout is flowing out of an adjacent hole, joint, or edge of the slab is sufficient evidence that all cavities or voids have been filled within the range of the hole being grouted, and pumping in that hole should cease.

Other factors that are used to determine when to cease pumping are the appearance of grout in adjacent holes, joints, or cracks; the displacement of water from under the slab; and the time of grout pumping, which should not exceed a reasonable amount of time (around 2 minutes). Grouting time is especially important when pumping grout near an adjacent lane, as in some instances the grout has broken through to the inside shoulder from the outside traffic lane.

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, such as a stiff grout or an approved concrete mixture.[19]

Slab stabilization should not be performed when the ambient temperature is below 4 ^{o}C. Traffic should be kept off a stabilized slab for at least 3 hours after grouting to allow for adequate curing of the grout.[6]

Slab Jacking

This section describes the procedures for performing slab jacking on settled PCC slabs. Additional information on slab jacking techniques is found in work by ASCE[21] and Bandimere.[11]

Location of Slab Jacking Holes

The best location of holes for a given site can only be determined by experienced personnel. A general guideline for establishing the location of holes is that they should be placed in about the same location as hydraulic jacks would be placed if they could be used to the lift the pavement. Holes should not be spaced less than 305 mm nor more than 460 mm from a transverse joint or slab edge. In addition, holes should be spaced 1.8 m or less center to center, so that less than 2.32 to 2.78 m^2 of the slab is raised by grouting a single hole. A greater number of holes may be required if the slabs are cracked.

The proper location of holes varies according to the defect that is to be corrected, as illustrated in figure 4-7.6. For a pumping joint where faulting has not yet occurred, a minimum of two holes can be used. For a pumping joint with one corner of the slab faulted, the hole at the low corner should be set back as shown in figure 4-7.6 to avoid raising the adjacent slab. Additional holes may be required to ensure filling all the voids under the slab. If both corners of the slab have settled, the inside hole should be relocated accordingly. Where the pavement has settled, a single hole located in the middle of the panel, about 1 m from the faulted joint, is sufficient.

Figure 4-7.7 illustrates the location of holes in a triangular pattern, correcting a lane settlement. 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 arrangement. It may be necessary to shift the holes at the end of the slab slightly to conform to this pattern and still be near the edge. A hole pattern used on one successful project is shown in figure 4-7.8.[11]

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 (along with survey instruments, if needed) is an excellent way to not only control the pumping sequence, but also to achieve the proper grade. One method is illustrated in figure 4-7.9. Small wooden blocks, 20 mm high, are set on the pavement surface along the outer and inner edges and a stringline is secured at least 3 m from each end of the depression. As grout 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.

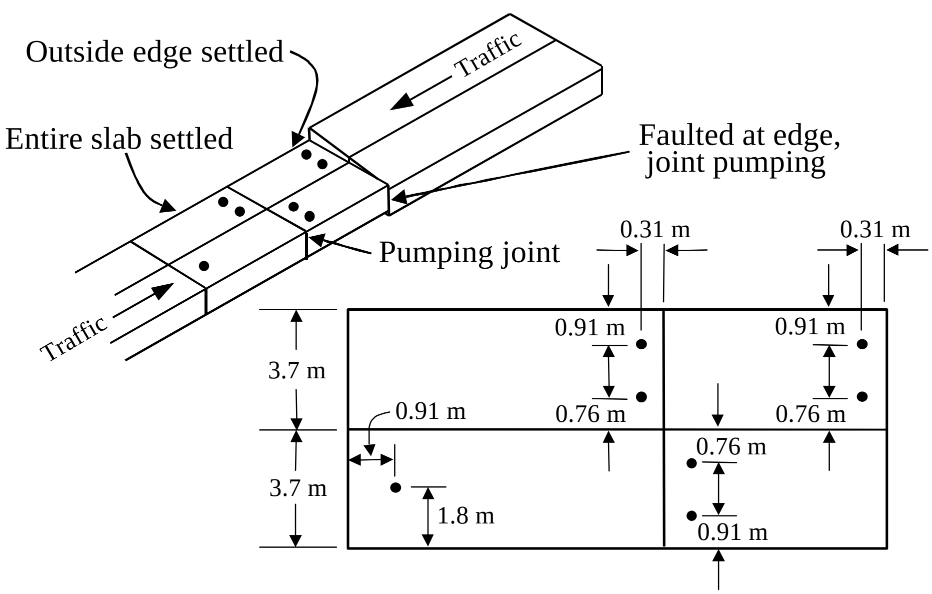


Figure 4-7.6. Location of holes depending on defect to be corrected.

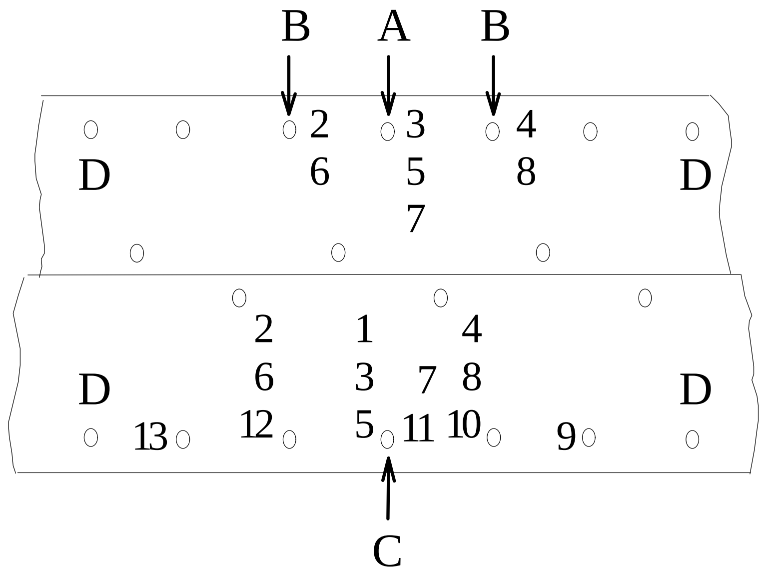
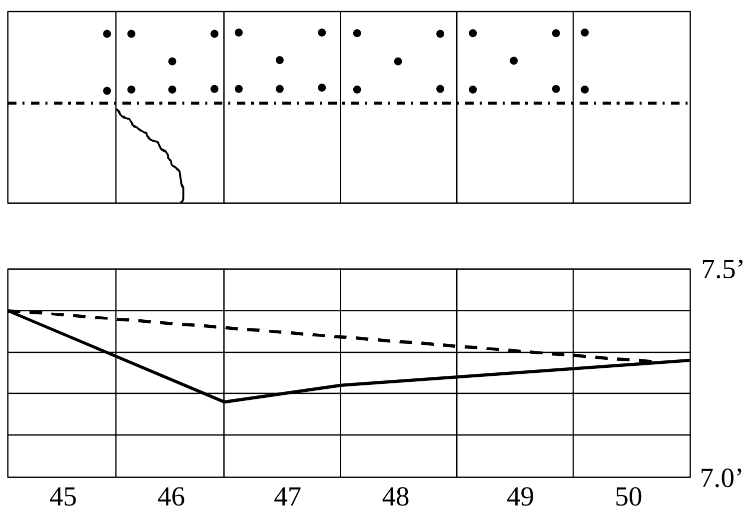


Figure 4-7.7. Location of holes and the order of grout pumping to correct settlement.



1 ft = 0.305 m

Figure 4-7.8. Plan and profile of a slab jacking segment in Louisiana.[22]

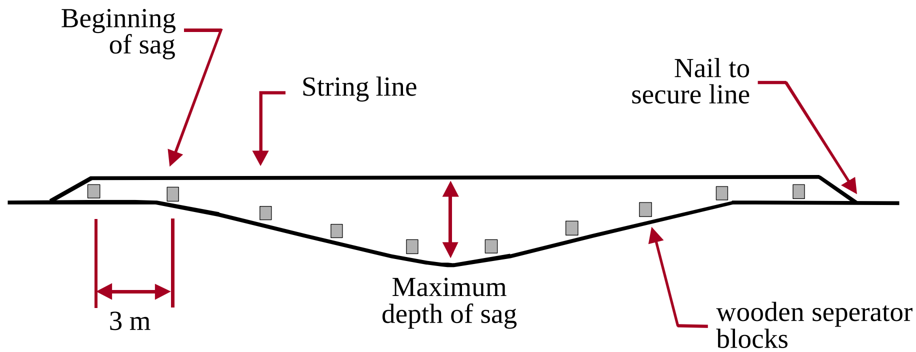


Figure 4-7.9. Stringline method of slab jacking.

Various agencies have different techniques for the raising of the slab. The following procedure is typical of that used by many agencies:

- 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 while pumping in any one hole. No portion of the slab should be more than 6 mm 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, throughout the entire operation to avoid cracking. Pumping should be done over the entire section in such a way 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 4-7.7 shows a top view of a dip. Grout pumping should begin in the middle of the dip, shown as point A. The hole where grout is initially pumped will take more grout than those at either side, due to the shape of the dip. Grout pumping should always begin at the outside holes, after which the inside row of holes will be grouted.

  b. Grout 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 4-7.7. This results in grout 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 grout 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 are 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 grouting conducted further from the center of the dip, as shown in figure 4-7.7 (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 4-7.7, should not be used until the slab is at the desired grade. A very thin grout, similar to that use for slab stabilization, should be used to ensure complete filling of the thin wedge-shaped opening that was created at this part of the dip.

10. CONSTRUCTION EQUIPMENT

Equipment For Asphalt Cement Slab Stabilization

Typical equipment used in asphalt cement subsealing includes a pressure distributor, air compressors, air hammers with drills, asphalt and air nozzles, plugs, a deflection gage, and other miscellaneous items. The insulated pressure distributor or tank truck must be capable of heating the asphalt to the required temperature and circulating it during the heating process. The equipment for pumping asphalt should be capable of developing up to 620 kPa. The distributor should be equipped with an accurate pressure gauge.

Air compressors capable of blowing air into the drilled holes and operating air hammers with drills are needed. The drills must be capable of cutting uniform holes 40 to 50 mm in diameter through the slab and reinforcing steel. Nozzles for asphalt and for air are needed that can be inserted into the holes for pumping hot asphalt and for blowing air under the pavement. A combination foot-stand and shield are commonly attached to the nozzle after it has been firmly wedged into the hole. The operator can hold it in position by standing on the wings of the stand.[14] Wooden plugs are usually used to temporarily plug the hole until the asphalt has cooled. The plugs are usually 0.9 m long and tapered on one end for ease of insertion and removal.

A sensitive slab-lift detection device must be employed to detect slab uplift as asphalt cement is pumped beneath the pavement. The Indiana Department of Transportation has developed a gauge specifically for this purpose.[12]

Proper safety clothing and face shields are required for personnel in the vicinity of the sealing operation because of the danger inherent in the pressure insertion of extremely hot asphalt cement. A shield approximately 1.2 by 1.2 m must be placed between the hole being pumped and the adjacent lane.

Equipment For Cement Grout Slab Stabilization

The following equipment is required for slab stabilization operations using cement grout:

- Air compressors to drive pneumatic hammers (if used), and blow out holes prior to grouting.

- Pneumatic hammers equipped with drills, or other drills that will cut 40- to 65-mm holes through the concrete slab and steel reinforcement. The drill must not be so heavy that excessive spalling occurs at the bottom of the hole.[1]

- A grout plant that is capable of accurately measuring, proportioning, and mixing, by volume or weight, the various materials composing the grout. The plant must also contain a positive-displacement, cement-injection pump capable of applying a 1720-kPa discharge. A colloidal mixing mill is required for flyash grout.

- Grout packers that can be inserted into the drilled or cored holes, sealing the hole while grout is being pumped. A grout return hose should be specified to recirculate the grout, preventing it from hardening in the hose.

- A flow cone, with all necessary components, so that the consistency of the mixture can be determined. The flow cone must conform to the dimensions and other requirements of the United States Army Corps of Engineers Test Method No. CRD-C611-80 or ASTM C 939-81.

- Slab-lift detection device to measure slab uplift. An example of this type of device is shown in figure 4-7.10.[23] Gauges should be capable of detecting 0.025 mm of uplift movement.

- Cylindrical wooden plugs or other approved plugs that can effectively plug holes after injection until the grout has set.

11. SUMMARY

Slab Stabilization

Loss of support from beneath rigid 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. Effective rigid pavement restoration includes the stabilization of slabs where necessary to restore support. However, because loss of support is caused by several factors (e.g., heavy loads, free moisture), undersealing alone is not sufficient to eliminate the problem; the mechanisms themselves must also be addressed.[24]

Slab stabilization is needed at joints and working cracks that are experiencing loss of support. Stabilizing slabs that have full support will only break the slabs and waste money. Deflection testing at slab corners can be used to detect loss of support, and remeasuring deflections after stabilization gives an indication of its effectiveness.

Figure 4-7.10. Device for monitoring slab lift.[23]

Typical cement grout mixtures currently used include cement-limestone dust slurry and cement-pozzolan slurry. The cement-pozzolan slurry is recommended for slab stabilization work due to its better flowability and strength gain characteristics. Fluidifiers and water reducer additives are commonly used to increase the flowability and reduce the water content of the grout, resulting in reduced shrinkage and higher strength. A hard, oxidized asphalt cement (penetration 15 to 30) is also used for slab stabilization. It is typically pumped at 204 °C to 232 °C at pressures of up to 629 kPa.

A variety of construction procedures are currently used by agencies and contractors. An experienced contractor and proper inspection are essential to a successful slab stabilization project. A good summary of slab stabilization procedures may be found in the most recent ACPA publication on this topic.[23] The Federal Highway Administration (FHWA) is also sponsoring the development of a manual of practice on undersealing for use in the SPS-4 activities of the Long-Term Pavement Performance (LTPP) program.

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 grout 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 (used in conjunction with survey equipment if necessary) can be used effectively to control slab movement. A variety of procedures are used, and the success of a slab jacking project is highly dependent on the skill of the contractor. Careful monitoring of slab lift is essential to minimize the development of slab stresses.

12. REFERENCES

1. Taha, R., A. Selim, S. Hasan, and B. Lunde. 1994, “Evaluation of Highway Undersealing Practices of Portland Cement Concrete Pavements,” Transportation Research Record 1449, Transportation Research Board, Washington, DC.

2. Wells, G. K. and S. M. Wiley. 1987, “The Effectiveness of Portland Cement Concrete Pavement Rehabilitation Techniques,” FHWA/CA/TL-87/10, California Department of Transportation, Sacramento, CA.

3. Vyce, J.M. 1985, “Fault-Removal Procedures for Rigid Pavement Joints,” Research Report 121, New York State Department of Transportation, Albany, NY.

4. Crovetti, J.A. and M.I. Darter. 1985, “Void Detection For Jointed Concrete Pavements,” Transportation Research Record 1041, Transportation Research Board, Washington, DC.

5. Wu, S.S. 1991, “Concrete Slabs Stabilized by Subsealing: A Performance Report,” Transportation Research Record 1307, Transportation Research Board, Washington, DC.

6. Darter, M.I., E.J. Barenberg, and W. A. Yrjanson. 1985, “Joint Repair Methods for Portland Cement Concrete Pavements,” NCHRP Report 281, Transportation Research Board, Washington, DC.

7. Dudley, S.W. 1983, “Evaluation of Concrete Pavement Restoration Techniques,” Demonstration Projects Report, Ohio Department of Transportation, Columbus, OH.

8. Tyner, H.L. 1981, “Concrete Pavement Rehabilitation—Georgia Methodology,” Proceedings, National Seminar on Portland Cement Concrete Pavement Recycling and Rehabilitation, FHWA-TS-82-208, Washington, DC.

9. Hall, K.T. and M.I. Darter. 1989, “Rehabilitation Performance and Cost-Effectiveness: 10-Year Case Study,” Transportation Research Record 1215, Transportation Research Board, Washington, DC.

10. Noureldin, A.S. and R.S. McDaniel. 1989, “Evaluation of Concrete Pavement Restoration Techniques on I-65,” Transportation Research Record 1215, Transportation Research Board, Washington, DC.

11. Bandimere, S.W. 1986, “Report and Review of a Major Slabjacking Case History,” Transportation Research Record 1104, Transportation Research Board, Washington, DC.

12. Sudol, J.J. and J.W. Muncaster. 1986, “Undersealing Concrete Pavements: The Indiana Method,” Public Works, Volume 117, American Public Works Association.

13. Jiang, Y. 1991, “Evaluation of Concrete Pavement Restoration Techniques on I-65,” Final Report, Indiana Department of Transportation, Indianapolis, IN.

14. Asphalt Institute (AI). 1975, "Undersealing Portland Cement Concrete Pavements with Asphalt," Construction Leaflet No. 13, The Asphalt Institute.

15. Slifer, J.C., M.M. Peter, and W.E. Burns. 1985, "Experimental Project on Grout Subsealing in Illinois: A 20-Month Evaluation," Transportation Research Record 1041, Transportation Research Board, Washington, DC.

16. Chapin, L.T. and T.D. White. 1993, "Validating Loss of Support for Concrete Pavements," Proceedings, Fifth International Conference on Concrete Pavement and Rehabilitation, Purdue University, West Lafayette, IN.

17. Florence, R.H. 1976, "Design and Construction of Several Maintenance Techniques for CRCP," Report No. JHRP-76-12, Indiana Department of Highways, Indianapolis, IN.

18. Barnett, T.L., M.I. Darter, and N.R. Laybourne. 1980, "Evaluation of Maintenance/Rehabilitation Alternatives for CRCP," Research Report No. 901-3, Illinois Department of Transportation, Springfield, IL.

19. Del Val, J. 1981, "Pressure Grouting of Concrete Pavements," Transportation Research Record 800, Transportation Research Board, Washington, DC.

20. Pierce, L.M. 1994, "Portland Cement Concrete Pavement Rehabilitation in Washington State: A Case Study," Transportation Research Record 1449, Transportation Research Board, Washington, DC.

21. American Society of Civil Engineers (ASCE). 1977, "Slabjacking: State-of-the-Art," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, New York, N.Y.

22. Higgins, C.M., R.W. Kinchen, and J.L. Melancon. 1970, "Louisiana Slabjacking Study," Research Report No. 45, Louisiana Department of Highways, Baton Rouge, LA.

23. American Concrete Pavement Association (ACPA). 1994, "Slab Stabilization Guidelines for Concrete Pavements," TB018P, ASCE, Skokie, IL.

24. Tayabji, S.D. 1986, "Evaluation of Load Transfer Restoration Techniques and Undersealing Practices," FHWA/RD-86/043, Federal Highway Administration, Washington, DC.

MODULE 4-8

GRINDING AND GROOVING

1. INSTRUCTIONAL OBJECTIVES

This module describes techniques for surface restoration on rigid pavements. The participant shall be able to accomplish the following upon successful completion of this module:

1. Differentiate between diamond-grinding and grooving, and describe the objectives of each.
2. List existing pavement conditions for which diamond-grinding of a jointed concrete pavement surface may be beneficial.
3. Describe conditions under which grooving of a pavement surface would be beneficial in reducing wet weather accidents.
4. Describe equipment and construction problems encountered in typical grinding and grooving operations.
5. Be familiar with the uses of carbide grinding equipment for rigid pavement restoration.

2. INTRODUCTION

Diamond-grinding and grooving are two different forms of surface restoration that are used to correct a variety of rigid pavement surface distresses. Each technique addresses a specific pavement shortcoming; either may be used in conjunction with other pavement restoration techniques as part of a comprehensive pavement restoration program. In some situations, it may be justified to use one of these techniques as the sole restoration technique in a pavement rehabilitation project. Carbide grinding is also used on a limited basis as a rigid pavement restoration tool, but its primary application is on flexible pavements (see module 3-4).

3. DEFINITIONS

Diamond-blade grinding is the removal of hardened portland cement concrete (PCC) through the use of closely-spaced, diamond saw blades mounted on a rotating drum. The major purpose of diamond-grinding is to correct surface irregularities and provide a smooth riding surface.

Diamond-blade grooving can be performed on both rigid and flexible pavements. In this operation, grooves are cut into hardened PCC using diamond saw blades with a center-to-center blade spacing of 19 mm or greater. The major objective of grooving is to reduce the incidence of hydroplaning, which can cause wet weather accidents.

Cold milling is generally performed only on flexible pavements and is discussed in module 3-4. However, it is being increasingly used on rigid pavements. Standard cold milling techniques are not acceptable on rigid pavements due to the poor surface finish and spalling at the joints. Modified cold milling techniques using drum-mounted carbide steel cutting bits (carbide grinding), however, have proven to be more effective on rigid pavements, but are still not without problems. The process effectively chips off the surface of the pavement. On rigid pavements, this technique may be used to remove small deteriorated areas or as a surface preparation technique before overlay placement.

4. PURPOSE AND PROJECT SELECTION

Diamond-Grinding

Diamond-grinding was first used in California in 1965 on a 19-year old section of I-10 to eliminate significant faulting.[1] 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. Since its first use in 1965, the use of diamond-grinding has grown to become a major element of concrete pavement restoration projects.

Diamond-blade grinding has been employed on rigid pavement surfaces for a variety of reasons, including the following:

- Removal of transverse joint and crack faulting.
- Removal of wheel path "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 concrete surface exhibiting inadequate macrotexture (improving skid resistance).
- Improvement of transverse slope to improve surface drainage.

Most often, diamond-grinding is used to improve rideability through the removal of faulting. The International Grooving and Grinding Association (IGGA) and the American Concrete Pavement Association (ACPA) recommend the following criteria to assess when a project should be scheduled for diamond-grinding to remove joint and crack faulting:[2]

- If the present serviceability index (PSI) drops within a range of 3.8 to 4.0, a thorough evaluation of the cause of this loss in serviceability should be conducted. After addressing any structural deficiencies, diamond-grinding should be conducted to restore the serviceability to a high level.

- Diamond-grinding should be conducted before faulting reaches critical levels. Such levels are dependent upon many factors, not the least of which is joint spacing. Less faulting is tolerated on pavements with shorter joint spacings. Georgia developed an indicator, called the faulting index, that has proven to be extremely helpful in scheduling diamond-grinding projects.[3,4] Each faulting index value of 5 represents 0.8 mm of faulting. For example, a faulting index of 15 translates into average faulting of 2.4 mm. Table 4-8.1 summarizes this concept.

In order to accurately characterize the degree of faulting on a project, a sampling approach is used to determine the number of joints on which to measure faulting. Table 4-8.2 recommends the number of faulting measurements needed on projects with different transverse joint spacings.[2]

The need for diamond-grinding should be based on the collection and analysis of pavement condition and roughness data. The most important factor in determining the cost-effectiveness of a repair strategy is a thorough evaluation of the collected pavement condition data.[5] Structural distress, such as pumping, loss of support, corner breaks, working transverse cracks, and shattered slabs, will require repair before grinding is conducted. Furthermore, if the cause of faulting is not addressed prior to grinding, many agencies have found that the faulting will shortly reappear.[6] The presence of widespread distress related to concrete durability, such as D-cracking, reactive aggregate,

Table 4-8.1. Faulting index concept.(2)

Average Fault	Faulting Index	Comments
0.8 mm	5	No roughness
1.6 mm	10	Minor faulting
2.4 mm	15	Grinding project
3.2 mm	20	Expedite project
4.0 mm	25	
4.8 mm	30	Discomfort begins
5.6 mm	35	
6.4 mm	40	Grind immediately

Table 4-8.2. Recommended number of fault measurements needed to assess level of faulting.(2)

Joint Spacing	Measure Cracks	Measurement Interval	Number of Fault Measurements Per Lane km
< 3.7 m	No	every 9th joint	> 32
3.7 - 4.6 m	No	every 7th joint	32 - 40
4.5 - 6 m	No	every 5th joint	33 - 44
6 - 9 m	Yes	every 4th joint	28 - 42*
> 9.14 m	Yes	every 4th joint	> 19*

* Include transverse cracks with joint fault measurements.

or freeze-thaw damage, may indicate that diamond-grinding is not a suitable restoration technique, and that a more comprehensive rehabilitation approach needs to be considered. The following factors should be considered in determining the feasibility of diamond-grinding for a particular project:

- If there is evidence that a severe drainage or erosion problem exists, as indicated by significant faulting (greater than 3 mm) or pumping, actions should be taken to alleviate the problem prior to grinding.

- The presence of progressive transverse slab cracking [all severity levels for jointed plain concrete pavements (JPCP) and only deteriorated cracks for jointed reinforced concrete pavements (JRCP)] and corner breaks indicates a structural deficiency in the pavement. Slab cracking and the faulting of these cracks will continue after grinding and will reduce the life of the restoration project.

- The hardness of the aggregate has a direct relationship on the cost of the grinding project. Grinding of pavements with extremely hard aggregate (such as quartzite) takes more time and effort than projects with a softer aggregate (such as limestone). If an extremely hard aggregate is present on a project, diamond-grinding may not be feasible due to the high costs.

- Concrete suffering from durability problems, such as D-cracking or alkali-aggregate reactivity, should not be rehabilitated through grinding.

- Significant slab replacement and repair may be indicative of continuing progressive structural deterioration that grinding would not remedy.

It should be emphasized that although grinding of either JPCP or JRCP provides a dramatic improvement in rideability, it should not be considered when serious structural or material problems are present. Grinding does not address these problems and the pavement's condition will continue to deteriorate after the grinding has been completed.

Grooving

Grooving on rigid pavements has been performed since the 1960s to reduce the potential for wet weather skidding accidents on highways and airfield runways. Grooving may be performed either transversely or longitudinally. The advantages of transverse grooving are that it not only provides the most direct channel for the drainage of water from the pavement, but it also introduces a surface that provides considerable braking traction. However, transverse grooving, although common on runways and bridge decks, is rarely used on highway pavements. This is due in part to construction difficulties encountered in maintaining traffic on the adjacent lane and in part due to excessive noise.

Longitudinal grooving has commonly been used on highways, especially along curved sections. 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 around horizontal curves.

Grooving may be an appropriate treatment where a wet weather accident problem exists. This normally occurs at curves and interchanges, but grooving could be done along an entire project if the number of wet weather accidents is substantial. It should only be used on pavements that otherwise are structurally and functionally adequate. Documentation of the effects of grooving have shown a dramatic reduction in wet weather accidents.[7,8]

Carbide Grinding

Carbide grinding uses carbide-tipped cutters or bits mounted on a revolving drum to chip away the pavement surface. This procedure is most commonly used on flexible pavements and has seen only limited use on rigid pavements. Carbide grinding operations can be conducted over the entire lane width and project length, as in the case of rut removal, or on small areas for use in partial-depth repairs.

Both carbide grinding and standard cold milling techniques have been used successfully on rigid pavements to provide a surface for bonding a concrete overlay and for removing an asphalt overlay that has deteriorated.[9] Conventional cold milling is not recommended for final surface texturing of rigid pavements because it produces too rough a surface and significantly spalls transverse joints.[10]

Carbide grinding and cold milling should be used cautiously on rigid pavements. Major uses for these techniques on rigid pavements include the following:

- Restoration of surface friction—cold milling machines modified with multi-head blocks (carbide grinding) may possibly be used on rigid pavements for these applications. The specification should require positive grade control, uniform texture, and an acceptable limit on the amount of joint spalling.

- Provision of a roughened, clean surface for bonding a concrete overlay—although secondary cleaning of the surface is still required. However, cold milling must be carefully performed as it can cause microcracking of the concrete. Shotblasting is preferred over cold milling, as cold milling may reduce the bond strength up to 50 percent.

- Removal of HMA and PCC material when conducting partial-depth repairs.

Cold milling and carbide grinding are most often used to remove material prior to placement of an overlay. These techniques create a clean, roughened surface for bonding and eliminate the need to raise drainage structures and other utilities to the level of the new pavement.

5. LIMITATIONS AND EFFECTIVENESS

Diamond-Grinding

The immediate effect of grinding is to provide a very smooth pavement surface. Examples of the improvement in roughness for several projects are shown in table 4-8.3.[11] These after-grinding roughness values are as good or better than what can be achieved at new construction.

Table 4-8.3. Measured roughness before and after diamond-grinding using a Mays Ridemeter.[11]

Project Location	Roughness (m/km)		
	Before	After 2 years	Average change
Alabama, Cullman, I-65, MP 308-316	1.86	0.92	0.95
Arizona, Phoenix, I-17, MP 202-213	1.99	0.58	1.40
Georgia, Atlanta, I-86, MP 56-68	1.55	0.63	0.92
New York, Newburg, I-84, MP 42-46	2.35	0.60	1.75
South Dakota, Rapid City, MP 50-66	1.45	0.58	0.87
Average of Five Projects	1.85	0.66	1.18

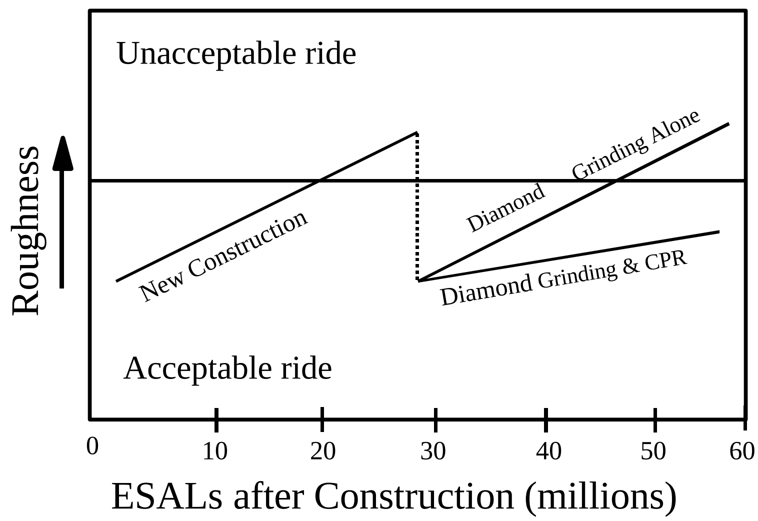
While grinding clearly has an effect on roughness, its effect on skid or friction numbers is probably more important. Roughness is affected by factors other than surface texture, such as faulting and full-depth and partial-depth repair condition, but friction numbers are greatly affected by the surface texture. A study in Indiana looked at roughness and friction numbers both before and for 5 years after CPR was performed.[12] The friction results are shown in table 4-8.4, and show both the impact of grinding on friction, as well as the decreasing impact over time (for reference, many agencies use a friction number of 30 as a point at which friction needs to be restored). A standard towed trailer with a ribbed, locked wheel torque measuring friction testing system (ASTM E-274) was used to measure the wet pavement skid resistance in terms of the friction number.

Table 4-8.4. Friction numbers (FN) before and after a grinding project.[12]

Years after Grinding	Average Friction Numbers		Average FN, Both Directions
	NB Lane	SB Lane	
0 (before grinding)	31	29	30
0 (after grinding)	40	44	42
1	49	45	47
2	40	42	41
3	34	34	34
4	34	33	34
5	33	33	33

However, if the cause of the faulting is not treated prior to grinding, faulting will begin to develop again, as illustrated in figure 4-8.1.[2] In New York, the expected service life of grinding operations is only 5 years before faulting will again be a problem.[13] A study of 76 grinding projects in 19 States found that after diamond-grinding is performed, faulting tended to develop at an even faster rate than originally.[3] This tendency to fault at an accelerated rate may be offset by concurrent CPR work to improve load transfer, establish slab support, and reduce pumping. Figure 4-8.2 illustrates this effect, showing the development of faulting on a typical project restored using diamond- grinding alone, and the development of faulting on a typical project restored with grinding in conjunction with other restoration procedures.[2] The results emphasize the need to combine grinding with other appropriate CPR techniques to minimize the recurrence of faulting.

It is not cost-effective to attempt to remove depressions through diamond-grinding. Any significant depressions should be corrected by slab jacking before grinding begins. Roughness profile measurement along the project in each lane is an excellent indicator of depressions and swells.



1 in = 25.4 mm

Figure 4-8.1. Effect of grinding on pavement roughness over time.(2,3)

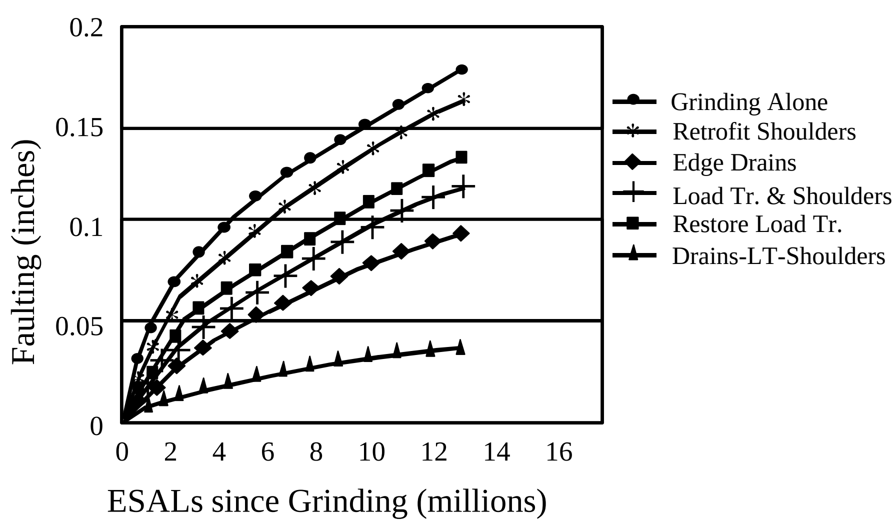


Figure 4-8.2. Effect of concurrent CPR techniques on pavement roughness over time.(2,3)

Grooving

Pavement grooving is an effective means of restoring surface friction. Grooving increases the macrotexture of the pavement and provides a channel for the water to escape, thereby decreasing the potential of hydroplaning. Transverse grooving is more effective in removing water from the surface, as it provides numerous drainage channels of short distance. The transverse grooves also provide considerable frictional resistance to braking action. Random transverse grooving is desirable to reduce tire noise on pavements and bridge decks. Longitudinal grooving has an advantage, however, in that the penetration of the tires into the grooves apparently holds the vehicle in alignment with the roadway, helping the vehicle to track around curves.(8,14) An excellent discussion of the technical aspects of grooving, hydroplaning, and friction is given by Horne.

The use of grooving varies widely from agency to agency. While many have done extensive grooving, other agencies have limited experience or have never used grooving. California is one State that has many years of experience with grooving, and had grooved over 16.7 million m^2 of pavement from 1965 to 1980.(14)

Carbide Grinding

The surface texture produced by any milling operation is a function of the carbide bit spacing, the rotational speed of the drum, the bit quality and wear, and the speed at which the milling machine is advanced over the surface. Increasing the number of carbide bits, increasing the rotational speed of the drum, and using slower advance speeds helps produce a smoother surface texture.

The technology for carbide grinding is rapidly improving, although it is still not without its problems. Currently, the recommendation is to use a test section approach to evaluate the adequacy of the technique before it is used on a wide scale basis. Where used, it is further recommended that an end result specification be developed for final surface texture rather than the current method specifications.

6. DESIGN CONSIDERATIONS

Diamond-Grinding

The surface characteristics of the pavement after grinding are highly dependent on the blade spacing, which in turn is highly dependent 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 produce a chip or "land area" between the sawed grooves. This concept is illustrated in figure 4-8.3.

For soft aggregate, the land area should have a minimum width of 2 mm, with an average width closer to 2.5 mm. The minimum land area in concrete with harder aggregates should be 1.7 mm, with an average width of 2 mm. This corresponds to blade spacings of between 174 to 187 blades per meter for hard aggregate and 164 to 177 blades per meter for soft aggregate.(2) The contractor is normally given the option in selecting the number of blades best suited for the job.

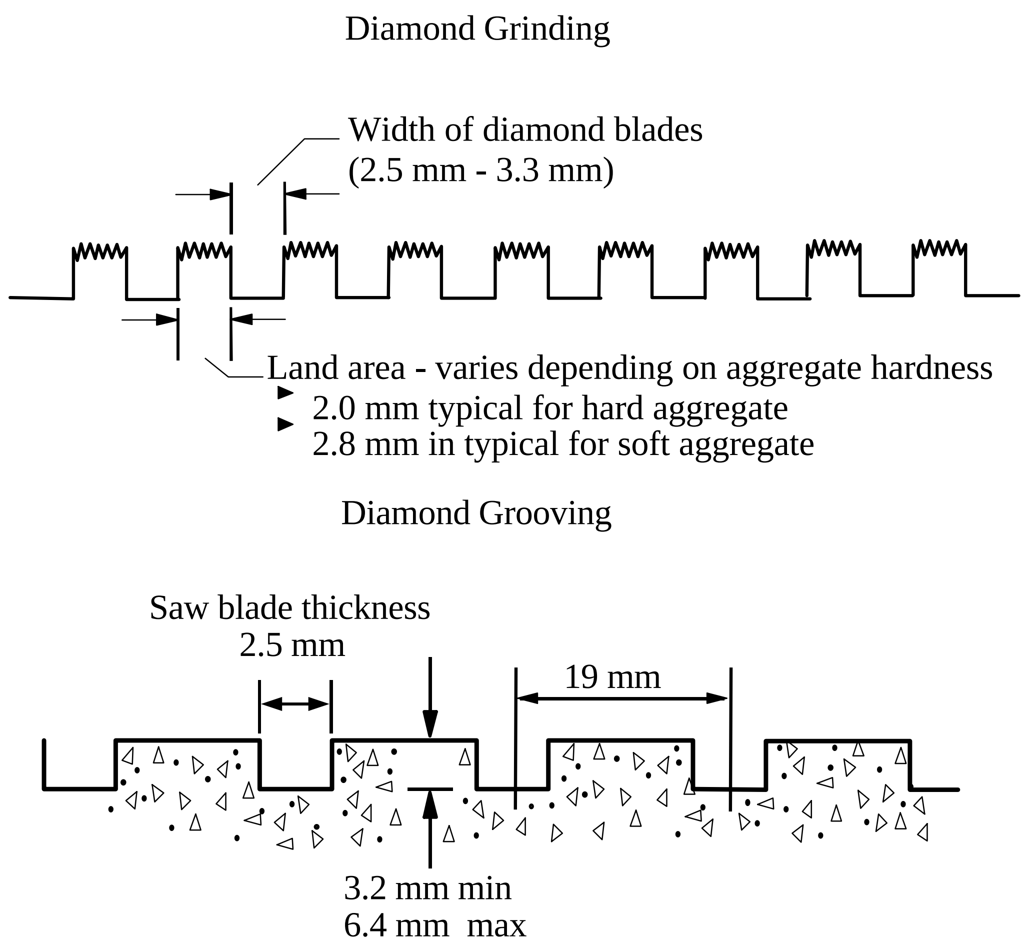


Figure 4-8.3. Typical dimensions for grinding and grooving operations.

Assessing Resulting Surface Profile

The quality of diamond-grinding can be assessed through measurement of the pavement roughness after grinding. The same ride quality standards are used for grinding projects as for new rigid pavement construction.(15) Profile traces obtained prior to grinding can be compared to profiles obtained after grinding to determine and document improvements in the ride quality.

The most commonly used profile measuring device for grinding operations is the California profilograph. Many agencies currently use this device for acceptance testing on new PCC pavements and these specifications can be applied on grinding projects as well. Typical acceptance values for the California profilograph are as follows:

- 0.16 km increments: 0.19 m/km maximum.
- 1.6 km increments: 0.11 m/km maximum.
- 1.6 km increments: 0.16 m/km average for job.

Other devices, such as the K. J. Law Profilometer, the Rainhart profilograph, Mays Ridemeter, BPR Roughometer, and the PCA Roadmeter, have also been used for acceptance testing. Equipment used in acceptance testing should be the same as that used in the initial evaluation and should be specified along with procedures to be followed in acceptance testing. The acceptance criteria for any of these devices should be the same for projects that have been ground as they would be for new construction. Whenever a roughness measuring device is used, especially the road response types, particular attention should be paid to proper calibration.

Transverse slope is also important and should be specified to obtain the desired grade. A transverse slope of 2 percent, or 7 mm over a standard 3.7-m wide lane, is recommended (2.5 percent slope commonly used in Europe). Grinding limits and transitions or stop lines at bridges and ramps should be clearly marked on plans.

Each agency should carefully evaluate its pavements and equipment in arriving at reasonable acceptance standards. It must be remembered that the level of smoothness required has a significant effect on the cost of the grinding operation. The setting of unrealistic levels that require extensive removal or additional grinding will cause a large increase in the bid cost of grinding.

Table 4-8.5 presents the change in roughness resulting from a Georgia DOT grinding project conducted in 1980. These results, obtained using a Wisconsin Roadmeter, show the dramatic impact that grinding had on the vehicle response to the road.

Table 4-8.5. Example of before and after roughness measurements obtained using a Wisconsin Roadmeter by the Georgia DOT.[16]

Location	**Roughness Index (Counts per km)**		
	Before Grinding	**After Grinding**	**Percent Difference**
I-75/I-475, NB MP 165-178	441	140	42
I-75/I-475, SB MP 165-175	459	148	42
Roughest km	584	180	43
Smoothest km	360	99	45

Skid Resistance

Pavement skid resistance is improved through grinding by the restoration of the cross slope and the enhanced surface macrotexture. Proper cross slope facilitates transverse drainage and reduces the potential for hydroplaning, especially in cases where studded tire wear has produced “ruts” in the concrete pavement. The increased macrotexture initially provides high skid numbers, but this improvement may be temporary, particularly if the pavement contains aggregate susceptible to polishing.[15] This effect may be offset by properly spacing the diamond saw blades, creating more land area between the grooves.

Skid resistance is generally measured using either a standard ribbed tire (ASTM E 501) or a standard smooth tire (ASTM E 524). It is recommended that testing be conducted with the smooth tire, since the ribbed tire is not sensitive to the macrotexture improvements created by grinding.[15] Table 4-8.6

illustrates the significant improvement in surface friction measured on several grinding projects. While these values will decrease over the first few years, an adequate macrotexture will normally be maintained for many years.(16) Pavements with harder aggregates such as granite will maintain adequate surface friction longer than pavements with softer limestone aggregates.

Table 4-8.6. Measured friction number before and after diamond-grinding using a Saab friction tester with a smooth tire (ASTM E 524).

Location	**Friction Number**	
	Before Grinding	**After Grinding**
Alabama, Culmen, I-65, MP 308-316	54	69
Arizona, Phoenix, I-17, MP 202-213	42	64
Georgia, Atlanta, I-85, MP 56-68	45	95
New York, Newburg, I-84, MP 42-46	40	85
South Dakota, Rapid City, I-90, MP 50-66	30	85
Average of 5 Projects	42	80

Grooving

Areas to be grooved should be clearly indicated on project plans. The grooves should have the dimensions shown in figure 4-8.3, 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 4-8.4 shows the increase in the number of wet weather accidents over time on a California highway before longitudinal grooving and the large decrease in the number of accidents after grooving.(14) Other States, such as Pennsylvania, have also observed a large reduction in wet weather accidents (75 percent) and in total accidents (57 percent) after longitudinal grooving.(8) Louisiana achieved a 64 percent reduction in wet weather accidents by performing longitudinal grooving.(17)

A disadvantage of longitudinal grooving is the perception by motorcyclists that longitudinal grooving impairs their ability to control their vehicle. Drivers of smaller, lighter cars with radial tires have also complained about poor handling on longitudinally grooved pavements. The tire industry now designs tires to minimize this problem.(18)

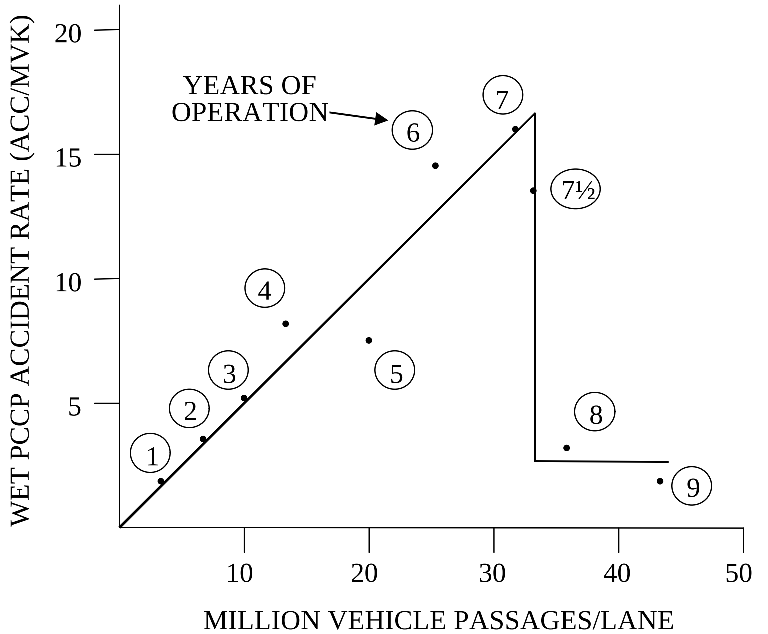


Figure 4-8.4. Wet weather accidents (accidents/million vehicle kilometers) for a selected California pavement before and after longitudinal grooving.[8]

7. PAVEMENT SURVEYS

Diamond-Grinding

Prior to considering a diamond-grinding operation, information on the degree of faulting at transverse joints (and cracks if applicable) is needed. This type of operation should be considered for restoring good rideability to those joints and cracks that have faulted to some degree, generally more than 3 mm on JPCP and more than 6 mm on JRCP (the limit on JRCP is not as strict because there are fewer joints). Information regarding past efforts to correct faulting should also be noted. Concurrent restoration techniques, such as retrofit load transfer, undersealing, and retrofit drainage, should be considered to help minimize the recurrence of joint faulting after diamond-grinding.

Grooving

Grooving operations are intended to reduce hydroplaning and accompanying accidents. Information regarding an area with a high number of accidents, as well as surface friction data for the section, should be reviewed prior to considering grooving operations.

8. COST CONSIDERATIONS

Cost of Diamond-Grinding

The cost of diamond-grinding is highly dependent on the hardness of the aggregate and the depth of cut (or amount of material to be removed). Other cost factors include the size of the project, labor rates, traffic control procedures (roadway closed or with traffic in adjacent lane) and the required smoothness of the ground pavement.

Due to increased productivity and competition, diamond-grinding costs have remained unchanged and in some cases have even been reduced. The rate for diamond-grinding typically ranges from $1.50 to $6.50 per square meter, with the major factor influencing the cost being the hardness of the aggregate.

A pavement roughness profile, either from a profilograph or profilometer, may be used by the contractor to help estimate the cost of grinding. Information on the type, source, hardness, and abrasiveness of concrete aggregate should also be provided to potential contractors.

Grinding need only be performed in the lanes that have significant faulting, wheelpath rutting, or other problems. The cost-effectiveness of grinding is maximized if it is performed only in the lane or lanes where it is needed, although the grinding of individual joints is not recommended. The rate of grinding is highly dependent on the hardness of the aggregate, ranging from 1 to 6 meters per minute.

Grooving Costs

The cost of pavement grooving has not increased greatly for several years, primarily because of increased equipment productivity. Like grinding, the cost of grooving is highly dependent on the hardness of the aggregate. The rates for longitudinal grinding are typically around $1.50 to $3.00 per square meter for rigid pavements. Production rates range from 2 to 12 meters per minute and are likewise highly dependent upon the hardness of the aggregate.

9. CONSTRUCTION CONSIDERATIONS

Diamond-Grinding and Grooving

The grinding 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 is either discharged onto the grass slopes adjacent to the shoulder or hauled away for disposal. There is some concern related to the toxicity of this slurry, and local environmental regulations must be consulted prior to conducting a diamond-grinding project.

Carbide Grinding

Cold milling has been used on both flexible and rigid pavements, but is more commonly used on flexible pavements. However, recent modifications using carbide tips and improved techniques (carbide grinding) have increased the applicability on rigid pavements as compared to conventional cold milling techniques. Problems encountered on rigid pavements include spalling at transverse joints and cracks, poor grade control, and tire noise.

Cold milled surfaces have been opened to traffic directly without using an overlay. One milled AC surface has been in service for over 8 years on I-57 in Illinois. This pavement, an AC/PCC composite, was milled to remove excessive rutting. There were some complaints by motorists due to the high tire noise initially produced, but as the milled surface wore down and the noise level dropped, complaints subsided.

Carbide grinding has been gaining wider acceptance as a rehabilitation method for rigid pavements. One problem encountered in both Oregon and Iowa was the spalling of transverse joints and cracks. This problem may be eliminated by filling the joints and cracks with a cementitious material prior to the milling operation.[19] Although a number of projects have been conducted in Iowa, Illinois, Oregon, Puerto Rico, and Washington, the long-term effectiveness of this technique for rigid pavements has not been established. The use of end result specifications are recommended to help ensure ride quality, adequate friction, reduced tire noise, positive grade control, and minimal joint damage.

Carbide grinding can also be conducted over small areas, as when used as the primary means of material removal for partial depth patching. In addition, it can be used on rigid pavements that pass beneath bridges to provide adequate clearance, as well as near curbs and drainage structures prior to overlaying to eliminate the need for expensive grade changes.

10. EQUIPMENT

Diamond-Grinding Equipment

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 track in the smooth path that results. The cutting head typically has a width ranging from 910 to 970 mm. The desired corduroy texture is produced using a spacing of 164 to 194 blades per meter of shaft. New improved grinding machines have greatly increased the capability to provide profiles that are smoother than new construction.

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 pieces of equipment are available that have are 1.5 m wide or more. Usually, a vacuum system is employed to collect the slurry produced by the sawing.

The diamond blades are spaced to increase the “land area” between grooves, as illustrated in figure 4-8.3. Typically, the blades are spaced 19 mm apart for longitudinal grooving, and the grooves have a width between 2.5 and 3.3 mm, and are cut to a depth of 3.2 to 6.4 mm. A 10-year study conducted in Germany found that groove widths in excess of 4.5 mm create unacceptable tire noise, so a groove width of 4.1 mm was recommended, with a spacing between grooves of 20 to 25 mm.[20] For transverse grooving, random grooves spaced 10 to 40 mm apart and 3 mm wide are recommended to reduce tire noise.[18]

Carbide Grinding Equipment

Carbide grinding equipment uses carbide bits mounted on a revolving drum to break up and remove the surface material. Drum widths vary from as short as 0.3 m to as long as 3.6 m. The carbide bits must be continually maintained and frequently replaced to provide a uniform texture with no ridges or low spots. This requirement is critical if the pavement is not going to receive an overlay. Positive, definitive grade control is also an essential part of a cold milling operation.[9]

Traditionally, a single carbide bit is mounted on a block, which is then bolted to the revolving drum. This results in a conventional bit spacing of approximately 15 mm. As the drum revolves and advances forward, the pavement material is impacted and is chipped away, producing a rough texture that is adequate for a riding surface.

Newer carbide cutting blocks are available that mount three to five carbide cutting bits on a single block (3- and 5-head blocks). When attached to a conventional cold milling drum, the number of cutting bits is increased by a factor of three to five, and the spacing between bits is significantly reduced (to approximately 5 mm for a 3-head block). Drums modified in this manner produce a much smoother texture, more suitable for use as a riding surface.

In Oregon, standard cold milling equipment modified with the 3-head blocks was used to remove ruts created in a rigid pavement by studded tire wear.[21] The Oregon DOT evaluated microcracking in concrete cores obtained from pavement sections that were cold milled and diamond ground. A petrographic analysis conducted according to ASTM C 856-83 revealed that the limited amount of microcracking observed was more prevalent in those cores subjected to diamond-grinding than those abraded by the 3-head carbide bits.[21] The resulting surface, although rougher than that produced by diamond-grinding, was still considered an acceptable riding surface.

Small milling machines are currently available which are specifically designed for partial depth patching. These machines have cutting head widths from 0.3 to 0.9 m and are highly maneuverable, making them ideal for use as a primary means of material removal. Any milling equipment must be inspected frequently to ensure all cutting bits are functioning properly and that worn bits are replaced. This is particularly critical if the pavement is not going to receive an overlay, as worn cutting bits will produce a surface texture characterized by ridges and low spots.

11. PROCEDURES

Diamond-Grinding

Grinding should be performed continuously along a traffic lane for best results. The direction of grinding does not have a significant impact on the resulting pavement smoothness or profile, but grinding should always be started and ended perpendicular to the pavement centerline.

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 1m by 30 m test area be textured by the grinding operation. Isolated low spots of less than 0.2 m^2 should not require texturing if lowering the cutting head would be required.[4]

Grinding has typically been conducted on multi-lane facilities using a mobile single lane closure, allowing traffic to be carried on any adjacent lanes. This generally results in higher construction costs for increased traffic control, and also results in increased risk to the construction workers.

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. Some projects use multiple grinding machines working together to expedite grinding operations.

Sample specifications for diamond-grinding operations are found in references 3, 4, 22, and 23.

Grooving Procedures

As previously indicated, grooving is most commonly performed longitudinally along the pavement. Typically, only localized areas (such as curves) are grooved, instead of an entire project length.

However, data from surface friction and wet-weather accidents can be used to determine the extent of the grooving.

Carbide Grinding Procedures

Carbide grinding is generally conducted longitudinally along the pavement profile. The forward speed of the machine, the rotational velocity of the rotating drum, the spacing of the carbide bits, and the grade control of the cutting head should be closely controlled to produce a uniform texture throughout the project. The longitudinal profile should be held to the same tolerance as new construction.

12. SUMMARY

Diamond-grinding and grooving are surface restoration techniques that have been used successfully to correct a variety of surface distresses on rigid pavements. Carbide grinding has also seen limited use; conventional cold milling is not recommended. The appropriate application of these techniques can result in a very cost-effective extension of pavement life.

Diamond-grinding uses closely-spaced diamond saw blades to remove a thin layer of material from a rigid pavement surface, resulting in a very smooth surface. It is primarily used to remove faulting and pavement rutting caused by studded tire wear. Diamond-grinding can also improve skid resistance by increasing the macrotexture of the surface and correcting deficiencies in pavement drainage, although this effect may be temporary if the aggregate is susceptible to polishing. Diamond-grinding is typically used in conjunction with other rehabilitation techniques.

Grooving is the use of diamond saw blades to cut longitudinal or transverse grooves into a pavement surface, leading to a reduction in wet weather accident potential. Longitudinal grooving is more commonly employed along curves, where the grooves provide a tracking effect, helping to hold the vehicle on the road. For areas where increased braking resistance is required, transverse grooving can be used. Grooving is usually done on pavements suffering little or no structural distress.

Carbide grinding uses carbide teeth cutting bits mounted on a revolving drum to chip away the surface material, producing a highly textured surface. On rigid pavements, modified cold milling equipment has been used for patch preparation and surface preparation for an overlay. Carbide grinding provides an excellent tool for AC surface removal prior to placement of an overlay and when conducting partial depth patching.

13. REFERENCES

1. Neal, B. F. and J. H. Woodstrom. 1976, “Rehabilitation of Faulted Pavements by Grinding,” Report No. CA-DOT-TL-5167-4-76-18, California Department of Transportation, Sacramento, CA.

2. International Grooving & Grinding Association/American Concrete Paving Association (IGGA/ACPA), “Diamond-Grinding and Concrete Pavement Restoration 2000,” TB-008.0 CPR, International Grooving and Grinding Association, Skyland, NC, American Concrete Paving Association, Arlington Heights, IL.

3. Snyder, M. B., M. J. Reiter, K. T. Hall, and M. I. Darter. 1989, “Rehabilitation of Concrete Pavements, Volume I—Repair Rehabilitation Techniques,” FHWA-RD-88-071, Federal Highway Administration, Washington, DC.

4. Darter, M. I., E. J. Barenberg, and W. A. Yrjanson. 1985, “Joint Repair Methods for Portland Cement Concrete Pavements,” NCHRP Report 281, Transportation Research Board, Washington, DC.

5. Roman, R. J., M. I. Darter, and M. B. Snyder. 1985, “Procedures to Determine the Optimum Time to Restore Jointed Concrete Pavements,” General Electric Company and American Concrete Pavement Association, Arlington Heights, IL.

6. Pierce, L. M. 1994, “Portland Cement Concrete Pavement Rehabilitation in Washington State: Case Study,” Transportation Research Record 1449: Design and Rehabilitation of Pavements, Transportation Research Board, Washington, DC.

7. Brunner, R. J. 1973, “Pavement Grooving,” Report No. 69-1, Pennsylvania Department of Transportation, Harrisburg, PA.

8. Ames, W. H. 1981, “Profile Correction and Surface Retexturing.” Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, Federal Highway Administration, Washington, DC.

9. Van Deusen, C. 1979, “Cold Planing of Asphalt Pavements,” Proceedings of the Association of Asphalt Paving Technologists, Volume 48.

10. Neal, B. F., and J. H. Woodstrom. 1978, “Evaluation of Cold Planers for Grinding PCC Pavements,” FHWA-CA-TL-78-15, Federal Highway Administration, Washington, DC, California Department of Transportation, Sacramento, CA.

11. Mosher, L. G. 1985, “Restoration of Final Surface to Concrete Pavement by Diamond-Grinding,” Proceedings of the Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West LaFayette, IN.

12. Jiang, Y., and R. R. McDaniel. 1993, “Evaluation of the Impact of Concrete Pavement Restoration Techniques on Pavement Performance,” Proceedings of the Fifth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West LaFayette, IN.

13. New York State Department of Transportation (NYSDOT), “Pavement Rehabilitation Manual, Volume II: Treatment Selection,” New York State Department of Transportation Materials Bureau, Albany, NY.

14. Horne, W. B. (No date), “Safety Grooving, Hydroplaning and Friction,” Technical Report, International Grooving and Grinding Association, Skyland, NC.

15. Henry, J. J., and K. Satio. 1983, “Skid-Resistance Measurements with Blank and Ribbed Test Tires and Their Relationship to Pavement Texture,” Transportation Research Record 946: Interaction of Vehicles and Pavements, Transportation Research Board, Washington, DC.

16. Tyner, H. L. 1981, “Concrete Pavement Rehabilitation—Georgia Methodology,” Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, Federal Highway Administration, Washington, DC.

17. Walters, W. C. 1979, “Investigation of Accident Reduction by Grooved Concrete Pavement,” FHWA/LA-79/133, Louisiana Department of Transportation and Development, Baton Rouge, LA.

18. Hibbs, B., and R. Larson. 1996, “Report of the PCC Surface Texturing Technical Working Group,” FHWA-SA-96-068, Federal Highway Administration, Washington, DC.

19. Iowa Highway Research Board (IHRB). 1987, “Pavement Texturing by Milling,” Iowa Highway Research Board Project HR-283.

20. Sulten, P. 1989, “Improving Road Friction,” Industrial Diamond Review Magazine.

21. Construction Technologies Laboratory, Inc. (CTL). 1990, “Microscopical Examination of Pavement Cores for Microcracks—Conducted for Oregon Department of Transportation,” Construction Technologies Laboratory, Inc., Skokie, IL.

22. Federal Highway Administration (FHWA). 1985, “Pavement Rehabilitation Manual,” FHWA-ED-88-025, Federal Highway Administration, Washington, DC (Supplemented April 1986, July 1987, March 1988, February 1989, October 1990).

23. AASHTO, AGC, and ARTBA. 1985, “Guide Procedures for Concrete Pavement 4R Operations,” AASHTO-AGC-ARTBA Joint Committee, Subcommittee on New Highway Materials, Task Force 23, American Association of State Highway and Transportation Officials, Washington, DC.

24. Federal Texturing of PCC Pavement Surfaces,” Technical Advisory T 5140.10, Federal Highway Administration, September 1979.

MODULE 4-9

LOAD TRANSFER RESTORATION OF JOINTED RIGID PAVEMENTS

1. INSTRUCTIONAL OBJECTIVES

This module presents information on an increasingly popular jointed rigid pavement rehabilitation technique, load transfer restoration (LTR). Upon completion of this module, the participants should be able to accomplish the following:

1. Identify the problems that can be addressed through the use of load transfer restoration devices.
2. Describe the techniques available for determining the need for load transfer restoration.
3. Describe procedures for properly installing load transfer restoration devices.
4. Identify the performance capabilities of various types of load transfer devices.

2. INTRODUCTION

For years, many jointed plain concrete pavements (JPCP) were constructed without dowels. In such a design, load transfer from one slab to the next at the contraction joint relies upon aggregate interlock. This type of design has performed well in some cases, particularly where truck volumes are low, the pavement structure is well-draining and support is good, the subsurface materials are non-erodible, and climates are mild. A very low annual rainfall can also contribute to the good performance of this design.

Unfortunately, the vast majority of these designs do not experience such favorable conditions. The result is that some rigid pavements have pumped, faulted, and experienced corner breaks and other cracking. The typical rehabilitation approach for such pavements involves the application of one or more of the suite of techniques that are presented in this block (e.g., slab stabilization, full-depth repair, retrofitted subdrainage, and diamond-grinding). These techniques can be costly and produce mixed results, especially when the pavement condition is deteriorating.

Load transfer restoration may be performed at joints and transverse cracks on existing jointed rigid pavements in order to forestall the need for these more costly techniques, especially where the timing of the project is appropriate. The most effective means of load transfer restoration is the use of dowel bars placed in slots across the joint or crack. Restoration of load transfer is expected to enhance pavement performance by reducing pumping and faulting and by reducing the number of corner breaks at joints and mid-panel transverse cracks. It should be noted that most jointed rigid pavements do not fail in fatigue, but rather due to joint-related distresses.

3. LOAD TRANSFER RESTORATION

Load transfer restoration is the installation of a device at transverse joints and cracks in order to transfer loads across slabs and reduce deflections. As implied by “restoration,” these devices are retrofitted in existing pavements that either do not have load transfer devices or in which the devices are not working. Load transfer restoration is intended to retard further pavement deterioration due to joint pumping, faulting, spalling, and subsequent cracking. It can also be used to prevent deterioration of mid-panel cracks in under reinforced JRCP.

The ability of a joint or crack to transfer load from one side of a joint or crack to the other is referred to as its load transfer efficiency (LTE). Good load transfer efficiency is a major factor in a pavement's structural performance.

Deflection load transfer and stress load transfer are two ways of expressing LTE. These are illustrated in figure 4-9.1, along with their most common equation. Deflection load transfer is usually defined as the ratio of deflection measured on the unloaded side of a joint to the deflection of the loaded side. If perfect load transfer exists, the ratio is 1.00 (or 100 percent), and if no load transfer exists the ratio is 0.00 (or 0 percent). Some agencies use the difference between the deflections of the loaded and unloaded slabs rather than the ratio, which lessens the effect of the deflection magnitude in determining load transfer efficiency.

Stress load transfer is defined as the ratio of the stress in the unloaded side of the joint to the stress in the loaded side, as shown in figure 4-9.1 (b). Stress load transfer and deflection load transfer are not linearly proportional. An approximate relationship is illustrated in figure 4-9.2, although the actual relationship depends on the concrete's properties (thickness, elastic modulus, and Poisson's ratio), subgrade support conditions, and the radius of the load plate used in the deflection testing.

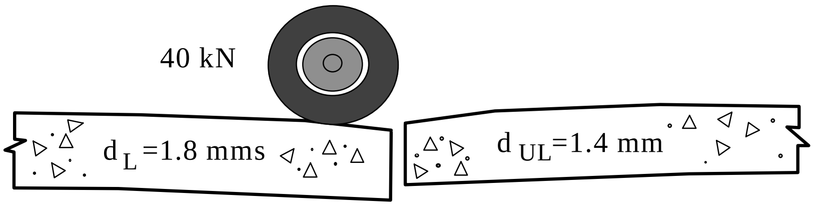
Joints that are doweled during original pavement construction normally maintain adequate deflection load transfer (70 to 100 percent) if dowels that are adequately sized and corrosion-resistant are used. However, many nondoweled JPCP have been constructed and their load transfer performance at transverse joints relies upon aggregate interlock of the abutting joint faces. Immediately after the pavement is constructed, and during warm weather when the slabs have expanded, aggregate interlock may provide adequate load transfer (unless a non-erodible stabilized base is used and joint spacing is relatively short). However, as joints open due to temperature variations, aggregate interlock is lost and load transfer is greatly reduced. Depending on the size and type of aggregate used (crushed versus rounded), openings as small as 0.25 mm can lead to a reduction of load transfer efficiency. Transverse cracks in both JPCP and jointed reinforced concrete pavements (JRCP), where steel has ruptured, also rely on aggregate interlock for good performance and may exhibit poor load transfer if the aggregate interlock is not maintained.

Types of Load Transfer Devices

Many different types of load transfer devices have been used to restore load transfer across joints and cracks in existing pavements. The most widely used and most effective devices are dowel bars that are placed in slots. Dowel bars placed in small slots (or kerfs) sawed into the pavement at joints have proven to be an effective method of load transfer restoration. See references 1, 2, 3, 4, 5, and 6. While smooth dowel bars are typically used and are the recommended load transfer device, I-beam bars have been tried on some restoration projects in New York.[7] A typical dowel bar load transfer restoration design is shown in figure 4-9.3. The dowels are installed in slots that are formed by multiple saw cuts. The concrete is removed using lightweight hammers or hand tools and then a joint-forming medium is used the full depth of the cut to maintain the joint. The dowels are coated with a debonding agent to facilitate movement, and expansion caps are provided on both ends of the dowel to allow for closure of the joint after the dowel bar is installed.

The most common alternative to dowels in slots is mechanical devices that are placed in cores. Such installations have included double-vee and plate and stud devices. Their performance has not been as good as retrofitted dowel bars.

a) Deflection Load Transfer Efficiency

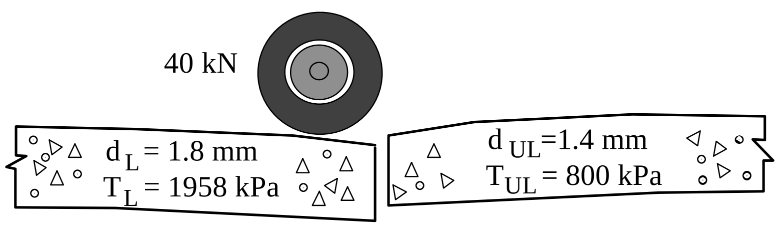


Deflection Load transfer

$$LT = \frac{d_{UL}}{d_L}$$

$$LT = \frac{1.4}{1.8} \times 100 = 80\%$$

b) Stress Load Transfer Efficiency



Deflection Load transfer

$$\text{Stress Load transfer} = \frac{T_{UL}}{T_L}$$

$$\text{Stress LT} = \frac{800}{1958} \times 100 = 41\%$$

Figure 4-9.1. Definition of deflection and stress load transfer efficiency.

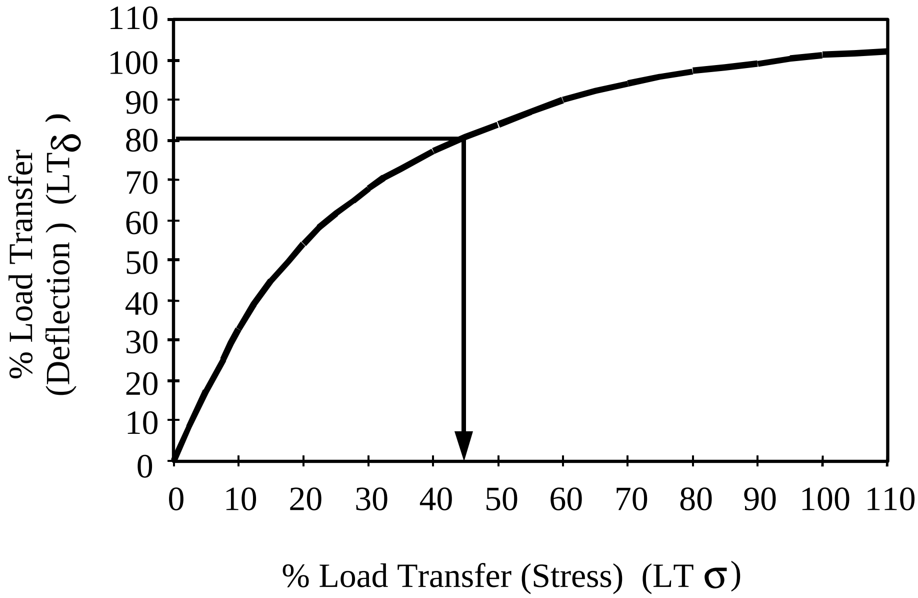


Figure 4-9.2. Approximate relationship between deflection and stress load transfer.

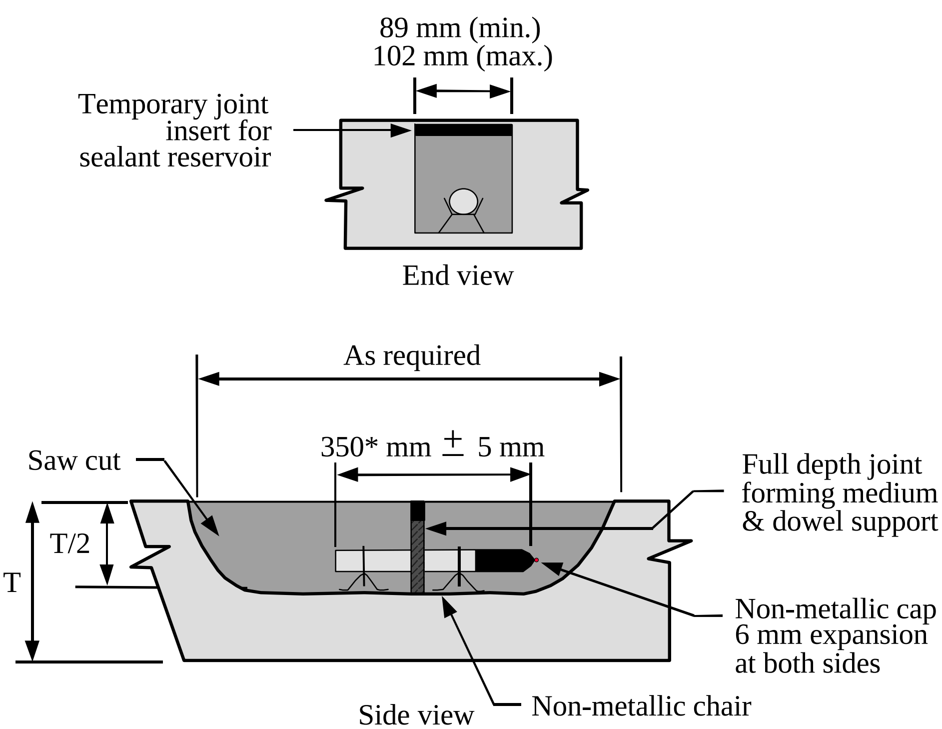


Figure 4-9.3. Dowel bar load transfer device.

Repair (Filler) Materials

Gulden and Brown[3] determined that the success or failure of a load transfer system depends upon the performance of the load transfer device, the preparation of the slot faces to ensure good bonding, and the repair materials. Desirable properties of the repair material include little or no shrinkage, thermal compatibility with the surrounding concrete, 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 slabs in a reasonable length of time. For deformed bars, the repair material must also provide sufficient bond between the device and the repair material, as well as between the repair material and the existing concrete. There are several types of repair materials that have been used with load transfer devices and can potentially meet these requirements. Generally, materials found to work well for partial-depth repairs should also work well in this application.[8] These are addressed in module 4-4. Recent guidance on material properties and testing may be found in Smoak et al.[9]

Proprietary Materials

Several proprietary materials are available for use as a filler material for LTR. The main advantage of these type of materials is that they are quick-setting, thereby allowing earlier opening times to traffic. Examples of such proprietary materials are Set 45 (magnesium phosphate-based material), Horn 240 (magnesium-phosphate based material), and Road Patch (fiberglass-reinforced, portland cement-based material). These materials, which are very sensitive to temperature, moisture, and installation procedures, were used in test sections in Georgia with both dowel bars and double-vee shear devices. They did not perform as well as the polymer concretes and plain portland cement concrete (PCC) when used with the double-vee devices on the same project.[3] Failure of the devices was primarily due to loss of bond between the repair material and the core wall. On the other hand, Set 45 has shown good performance in Puerto Rico as a filler material for retrofitted dowel bars.[11] It is strongly recommended that all materials without past satisfactory performance be tested in the laboratory for specification compliance before being used in the field.

Polymer Concretes

Polymer concretes have also been used as a filler material. Polymer concretes are commonly methacrylate-based materials and include products such as Concresive, Silikal, and Crylon. Polymer concrete consists of a liquid resin, powder filler, and fine aggregate. The mortar often attains 80 percent of its full strength in 45 minutes to 2 hours after placement under field conditions at temperatures from 4 ^{o}C to 38 ^{o}C.

Portland Cement Concrete

Portland cement concrete has been used with both shear devices and dowel bars. It is approximately one-third of the cost of the proprietary materials, and there are no thermal compatibility problems with its use. Most mixes use type III cement, calcium chloride (unless corrosion is a potential problem, in which case a non-chloride accelerator is recommended), and aluminum powder to improve setting times and reduce shrinkage. Sand and an aggregate with 9.5 mm maximum size are commonly used to extend the yield of the mix. Proprietary PCC mixes have also been used.

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.

4. PURPOSE AND APPLICATIONS

Dowel bars at transverse joints should be a part of the design of all JRCP and any JPCP subjected to heavy loads. Nonetheless, there are many jointed rigid pavements constructed without dowels. When these pavements begin to exhibit distresses associated with poor load transfer, LTR should be considered. It also may be an appropriate technique on transverse cracks of JRCP if the cracks are fairly uniform and have not widened or started faulting. When installed properly on appropriate projects, LTR improves load transfer at joints or cracks. The result is that pavement deterioration associated with poor load transfer decreases. This includes faulting, pumping, cracking, spalling, and other distresses that are caused by poor load transfer.

Load transfer restoration with retrofitted devices is not a new technique. Snyder et al.[2] reported on a number of different installations in 13 States. This approach has been used in Puerto Rico since at least 1983. Major projects include a portion of I-90 in Washington State that was undertaken experimentally in 1992 and on a much larger scale in 1993, 1994, 1995, 1996, and 1997; an experimental installation in Indiana in 1993, and projects in Kansas, North Dakota, South Dakota, Minnesota, West Virginia, and New Jersey.

5. LIMITATIONS AND EFFECTIVENESS

Joint load transfer devices are intended to restore load transfer capacity at joints and cracks where it has diminished or improve load transfer capabilities where they were not built in. Poor load transfer can be caused by many factors. These include:

- Absence of load transfer devices.
- Inoperable load transfer (due to corrosion, for example).
- Excessive joint opening and loss of aggregate interlock.
- Poor pavement drainage (and subsequent softening of underlying support).

Distresses that may develop as a result of poor load transfer include pumping, faulting, cracking, and corner breaks.

Load transfer restoration should be further explored as an appropriate rehabilitation technique for all faulted transverse joints or cracks that exhibit poor deflection load transfer (as indicated by deflection load transfer efficiencies of less than 70 percent) when measured in the early morning or in cooler weather (AASHTO 1993). Heavy-load deflection testing devices capable of simulating slab bending caused by regular traffic loads, such as the Road Rater or the Falling Weight Deflectometer (FWD), should be used for obtaining deflection and load transfer data.

Load transfer restoration is also recommended for joints and cracks with poor load transfer in rigid pavements that are to be overlaid with hot-mix asphalt (HMA) or bonded concrete overlays. This technique reduces the incidence and severity of reflection cracking, spalling, and deterioration of the overlay. Another consideration for performing load transfer restoration prior to an overlay is that overlay thicknesses do not need to be as great for pavements with adequate load transfer. The reduced cost of asphalt overlay thickness could make joint load restoration a cost-effective alternative. Where thin flexible overlays are being placed (less than 100 mm) and the underlying pavement is in good condition, saw and seal techniques may be considered to reduce reflective cracking (see module 4-14 for more information on mitigating the effects of reflective cracking).

In most instances, the pumping and faulting mechanism can be corrected by installing joint load transfer devices. For severe faulting, grinding is necessary to restore rideability after the installation of the load transfer devices or, in severe cases, full-depth joint repairs or slab replacements may be more appropriate.

The best performance has come from the use of 38-mm diameter, epoxy coated dowel bars. These bars should be 450 mm long, and placed parallel to the longitudinal joint. Tie bars may be used to restore load across non-working cracks in non-doweled JPCP. Smooth dowels are recommended for LTR on cracks in doweled JPCP or JRCP.

Performance

Retrofitted dowel bars generally have performed well.(1,2,10) Results show the dowels to have performed very well after up to 9 years of traffic, with the effectiveness of the repair material not being as critical as with the shear devices.(2) Puerto Rico has installed many miles of retrofitted dowel bars as part of their rigid pavement restoration program, and a review of over 7,000 dowel bars installed over 8 years indicated that fewer than 0.5 percent of the repairs have failed.(11) Puerto Rico used a slot 40 mm wide and Set 45 as the filler material.

Table 4-9.1 summarizes the performance of several retrofitted load transfer devices.(2) These represent average results collected from 13 projects in 9 states. It is observed that the dowel bars and the I-beams are showing the smallest percentage of failure (in terms of debonding, material failure, or device failure), although the I-beams showed excessive faulting in Georgia(3,12) and in New York.(2) The dowel bars exhibit the least amount of joint faulting. It should be pointed out that the faulting, age, and traffic loadings represent mean values averaged over all projects.

The performance of other load transfer restoration devices has varied greatly. While little information is available on the performance of the stud and plate device, they reportedly have worked well in a few airport applications.(1) The performance of the double-vee shear device has generally been poor. A significant number of failures of this device have occurred, most of which were due to a bond loss between the device and the core.(1,2) Georgia reports that the performance of the double-vee devices is influenced by the effectiveness of the patching materials.(3) While several modifications have been made to the device over the last several years, the long-term performance capabilities are not known. Results in Florida indicate that the double-vee devices exhibited higher deflections than retrofitted dowels, although each treatment was effective in reducing faulting as compared to the control sections.(13)

Table 4-9.1. Summary of field performance of LTR devices.[2]

	Dowel Bars	Double Vee	Figure Eights	I-Beams
Number of Devices	515	810	36	164
Percent Failures	2	28	25	1
Mean Age, years	3.8	2.5	9.0	2.0
Mean ESALS, millions	2.6	2.6	5.5	4.0
Mean Faulting, mm	1.02	1.78	2.03	3.30

Performance of Patch Materials

Overall patch material performance has varied. In Indiana, a high strength type I mix cracked immediately after placement, but a magnesium phosphate material performed well.[14] On the other hand, proprietary magnesium phosphate materials have not worked well in Arizona, California, Idaho, and Washington State. With advancements being made in installation equipment and project selection, the current weak link in LTR success is probably the patch material. Care must be taken to select a fill material that is appropriate for the conditions in which it is being used. Materials that are successfully used for partial-depth spall repairs are good candidates.

6. DESIGN CONSIDERATIONS

There are several key issues related to the design of an LTR project. These include project selection, selection of the LTR device, and determining the installation configuration. Secondary issues include methods of placing the device and the type of bonding agent.

Project Selection

From experience on projects in Puerto Rico, Washington, and Indiana, some guidance has emerged about when this technique is appropriate. Projects that are ideal candidates include heavily loaded jointed rigid pavements greater than 200 mm thick that are constructed without load transfer devices. The pavement should not exhibit major deterioration, such as corner breaks, faulting greater than 3 mm in JPCP and 6 mm in JRCP, and other signs of progressive structural deterioration. Additionally, LTR may be considered on pavements that already have load transfer devices that are either inoperable or under-sized, and on mid-panel cracks in JRCP that are tight and do not exhibit structural deterioration.

The condition of the existing pavement is perhaps the primary factor that affects the long-term performance of this technique. The FHWA[14] stresses that the pavement should be in overall good or fair condition for LTR to be cost effective. The optimum time to apply this strategy is when the pavement is just beginning to exhibit signs of distress, such as pumping or the onset of faulting. This point can best be identified when accurate and regular monitoring data are available, such as from a pavement management system database. Results from New York also emphasize the importance of the pavement condition, noting that LTR is appropriate when slabs are in good to excellent condition and the pavement has not lost its structural integrity.[15]

For agencies with extensive JRCP mileage, it is recommended that faulting of mid-panel cracking be monitored separately from transverse joint faulting to help to identify the appropriate time to retrofit load transfer. Early intervention would help to improve the ride quality, reduce the overall costs of rehabilitation, and extend the service life of the pavement if the slabs are otherwise in good condition.

Selection of the LTR Device

As is noted earlier, a range of devices has been proposed for LTR, with the I-10 project in Florida, representing perhaps the most comprehensive test of different devices under similar conditions.[13] If the apparent material and installation-related failures are ignored, retrofitted dowels provided greater improvements in the key measures—load transfer and faulting control—than the shear devices. The deflection LTE prior to the load transfer restoration project was less than 10 percent in most sections; for the retrofitted dowels after construction this was improved to between 50 and 80 percent, while the improvement with the shear devices was only to a value between 20 and 55 percent. Although the effect of the different devices on faulting are somewhat ambiguous, overall the dowels performed better than the shear devices and within the dowel experiment the larger diameter dowels (38 mm) performed better than the smaller (25 mm) ones. Joints with dowels also had lower corner deflections than joints with shear devices. Figure 4-9.4 compares load versus deflection results from two doweled sections and two with shear devices, and clearly shows the advantage of the doweled sections in reducing deflections.

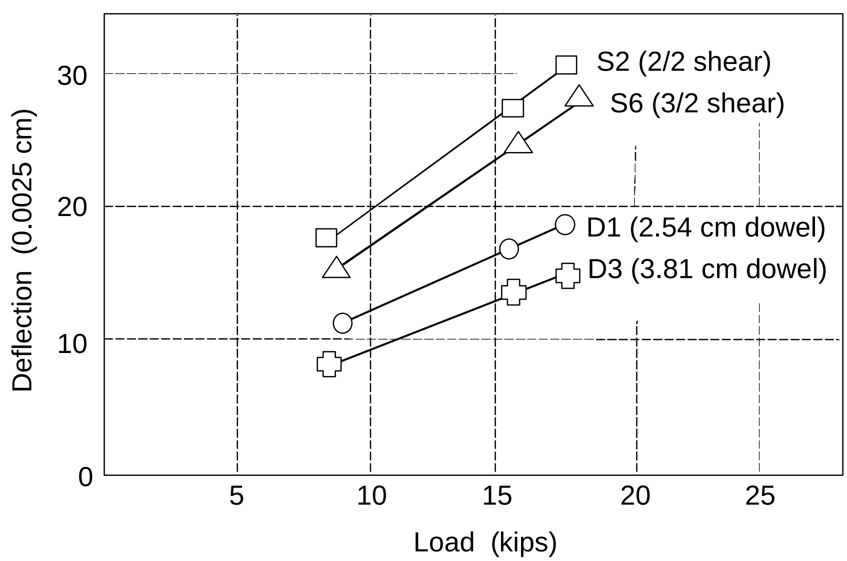


Figure 4-9.4. Example load versus deflection results for different designs on I-10 in Florida.[13]

Results from the study on I-84 in New York that considered a number of methods of restoring support suggest that a shear device performed better than retrofitted I-beams.[15] This is based on a comparison of the increase in faulting after the installations. However, it was noted that the patching material quality on the I-beams was less critical than on the shear devices.

As this brief discussion indicates, a number of different devices have been used for LTR. The trend in the United States is toward the use of smooth round dowels at joints and deformed reinforcing bars or smooth dowels at non-working cracks.[14] This may be explained in part by the good long-term performance of dowels in ongoing installations in Puerto Rico, and in projects in Washington and Indiana. Dowels have also proven to provide the best load transfer in new pavement construction.

Installation Configuration

In a number of projects, efforts were made to identify appropriate configurations for the retrofitted devices. The primary issue has been how many devices to put in each wheelpath. There is a tradeoff between putting in enough devices to obtain the desired results and putting in so many that the construction process is too slow and expensive. Many combinations have been tried in an attempt to obtain a cost-effective design. Considering only the dowel bars, the following configurations have been tried and reported on in the literature:

- Three or four dowels per wheelpath (WA).
- Five dowels per wheelpath (FL).
- Three dowels per wheelpath (FL, IN, and PR)

Four dowels per wheelpath have also been tried on the West Virginia Turnpike in milled slots.

Reported results, especially from the Florida study, suggest that the desired performance benefit may be economically obtained from three dowels per wheelpath.[13] This is also the current recommendation of the FHWA.[14] Figure 4-9.5 illustrates the three-dowel installation from Florida and the four-dowel installation from Washington

7. PAVEMENT SURVEYS

Prior to the installation of LTR devices, joints and cracks that would benefit from improved load transfer are identified. This is accomplished by measuring the faulting and the load transfer efficiency across the candidate joints and cracks. Joint and cracks with some faulting (but less than the previously noted critical values) and poor load transfer may be candidates for load transfer restoration if the slabs are otherwise in good condition. Additional information may be gleaned from lifting several slabs and examining the condition of the base, subbase, and subgrade for erodibility. Coring may also provide useful information about the condition of the bottom of the rigid slab and subsurface layers.

LTE should be measured during cooler temperatures (ambient temperatures less than 21°C) and during the early morning when the joints will not be tightly closed. As the temperature rises, the joints close tightly and load transfer efficiency cannot be adequately evaluated. In addition, LTE must be determined using a device such as the FWD or Road Rater that is capable of applying loads comparable in magnitude and duration to that of a moving truck tire. LTE should be measured in the outer wheelpath, which is subject to the most heavy 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.

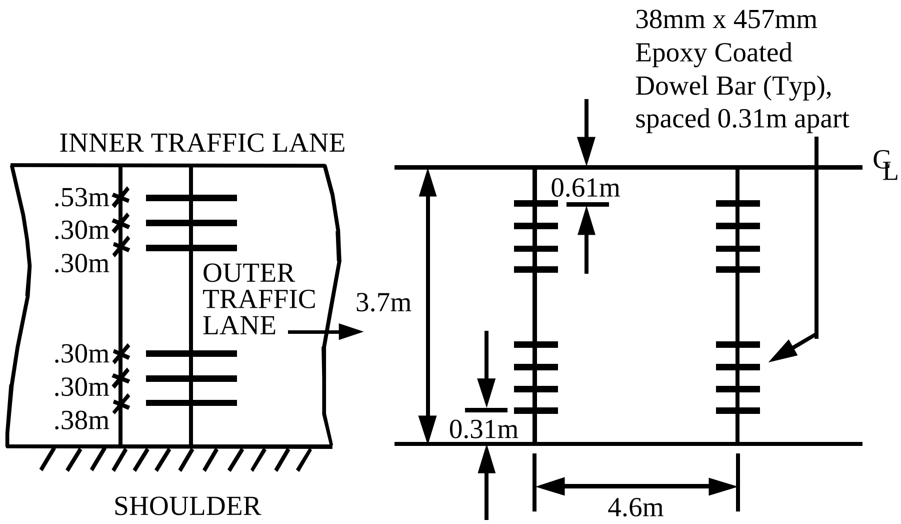


Figure 4-9.5. Typical installations of three and four dowels per wheelpath.(10,16)

In determining the LTE, it is important that slab bending effects be taken into account when the deflections are measured more than a few centimeters from the joint or crack (AASHTO 1993). Slab bending effects must be considered in determining load transfer at the joint or crack. Figure 4-9.6 illustrates how to account for slab bending effects in the determination of the LTE.

Any joint or crack having a measured deflection load transfer efficiency of less than 70 percent should be considered for load transfer restoration. A joint with lower LTE than that will exhibit very high deflections and virtually no stress transfer, both of which can lead to the development of pumping and faulting.

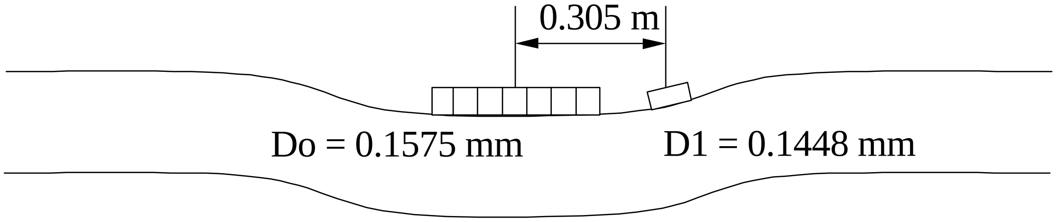
The magnitude of the corner deflections should be considered in addition to the LTE. It is possible for slab corners to exhibit very high deflections, yet still maintain a high LTE. In this case, even though the LTE is high, the large corner deflections can lead to pumping of the underlying base course material, faulting, and perhaps corner breaks. The corner deflections should be compared throughout a project to help determine if the typical corner deflection values become excessive.

8. COST CONSIDERATIONS

Installation Costs for Load Transfer Restoration

Previously reported installation costs for retrofitted dowel bars vary greatly, depending upon the size of the project. New York reported installation costs of $100 per dowel, while Florida reported installation costs of $60 per dowel. However, these were both experimental projects, and the costs should not be considered representative of more typical projects. For example, Puerto Rico, which has over 10 years of experience in using this technique, reported installation costs of $20 per slot.(14) The 1993 bid price in Washington State was $34.50 per dowel, down from the 1992 cost of $62.00. Projects in New Jersey, West Virginia, and Minnesota have generated a range of costs from $22.50 to $50.00, depending on the size of the project and the number of dowels installed.

CENTER SLAB LOADING

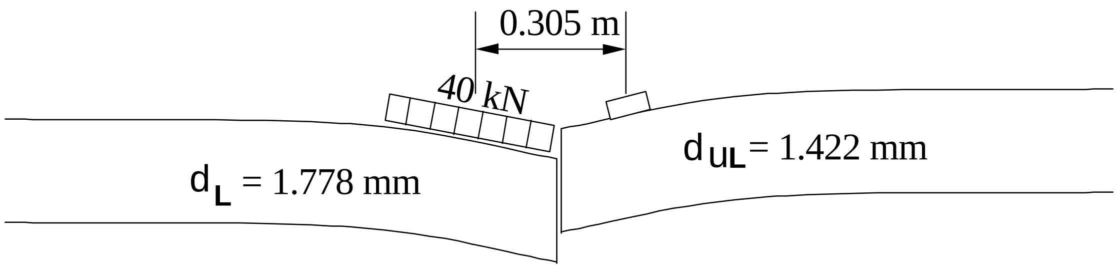


$$\text{SLAB BENDING FACTOR} = A = \frac{Do}{D1}$$

$$A = \frac{0.1575}{0.1448}$$

$$A = 1.09$$

LOADING AT JOINT OR CRACK



$$\text{LOAD TRANSFER EFFICIENCY} = A \times \frac{d_{UL}}{d_L} \times 100$$

$$\text{LTE} = 1.09 \times \frac{1.422}{1.778} \times 100$$

$$\text{LTE} = 87\%$$

Figure 4-9.6. Slab bending factor calculation and LTE adjustment.

As the technique is receiving increasingly widespread acceptance the costs are decreasing. This decrease reflects the responsiveness of contractors to developing innovative construction methods and equipment to conduct this work.

9. CONSTRUCTION

The installation of retrofitted dowel bars is a relatively simple procedure. However, it can also be rather laborious and expensive, in that it involves a time-consuming process of cutting and preparing the slots. The construction procedures that are commonly followed are illustrated in figure 4-9.7 and described below. Detailed design and construction guidelines and guide specifications are provided by Snyder et al,[2,17] and FHWA/ACPA[16].

Create Slots

The slots for dowels have been made in a variety of conventional and innovative ways. Perhaps the most common method has been to make multiple saw cuts using a diamond saws, followed by breaking up the concrete with a light jackhammer (alternate procedures are discussed below). Equipment is now available to saw cut three slots per wheelpath simultaneously. The slots should all be parallel to each other and to the longitudinal joint: this is best achieved by carefully marking the saw cut locations. A 65- to 100-mm wide slot is required for each dowel bar, although narrow slots slightly larger than the diameter of the dowel bars have been used successfully in Germany.[18] Puerto Rico has also had good success with using a narrow, 40-mm slot for their 25-mm diameter dowel bars.[11] The ACPA recommends that the slot width be about 20 mm greater than the dowel diameter.[17]

The "fins" formed by the saw cuts can carry traffic for some time without breaking and causing a hazard; traffic should be limited to 2 to 3 days. In some cases, for constructability reasons, light weight vehicular traffic on pavement with cut slots can be prohibited for up to one week.[16] A saw cut 900 mm long at the pavement surface is normally required to provide a 450-mm long slot at mid-slab depth, although this varies with saw diameter and slab thickness. The saw cut should be deep enough to position the centerline of the dowel at mid-depth of the slab, allowing a clearance of approximately 13 mm beneath the dowel bar for placement on chairs. The bottom of the slot should also be flat and uniform across the joint.

After the saw cuts have been made, lightweight jack hammers (less than 14 kg) or hand tools are used to remove the concrete in each slot. A lightweight hammer will help to keep the concrete from breaking through the full depth of the slab. After removing the concrete wedge, 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.

Prepare Slot

Prior to the placement of the dowels or patch material, the joint or crack in the slot is caulked to prevent intrusion of any patch material that might cause a compression failure. The backfill must not be allowed to infiltrate the crack beneath or on the sides of the joint. If a bonding agent is used, it is placed along the slot side walls and bottom.

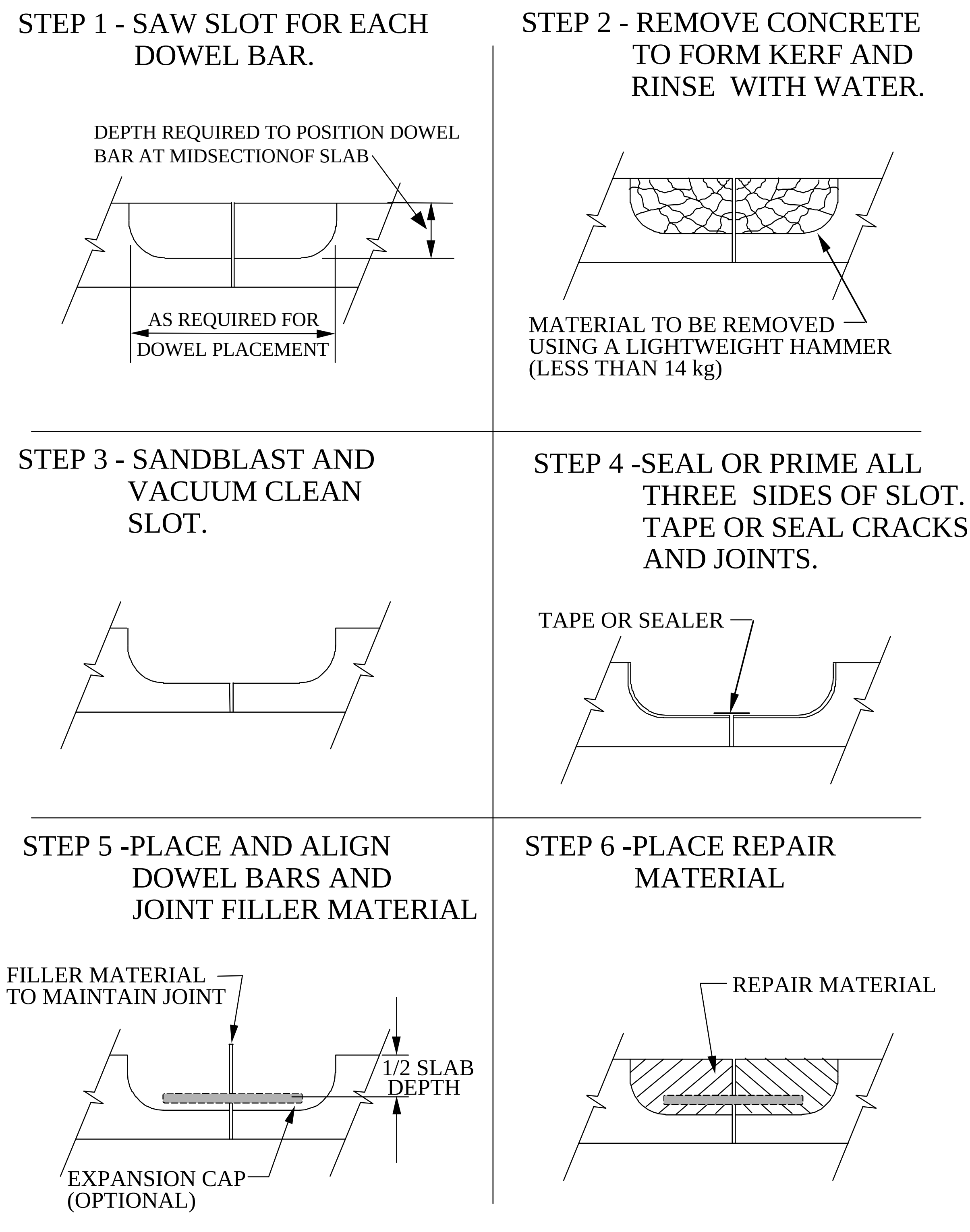


Figure 4-9.7. Construction procedures for retrofitted dowel bar installation.

Place Dowel Bars

The dowel bars should be lightly greased or oiled 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 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. Alignment is believed to be an important part of ensuring good project performance. Some agencies have used jigs to align the dowel bars.(7) The practice in Germany is to place the dowels directly in the slot on top of a thin layer of repair material.(18) Deformed reinforcing steel is used in a similar manner for transverse cracks in JRCP in Germany, but is not used for joint repair.

A filler board material placed at the mid-point of the dowel maintains the integrity of the joint or crack and prevents the repair material from infiltrating and resisting movement. The patch material is then placed in the slot according to the manufacturer's recommendations. Most repair materials require consolidation to perform well. The lane may be opened to traffic after several hours of curing, depending on the agency's experience with the repair material and slab temperature. The joint insert may be removed during resealing operations.

Final Considerations

Load transfer restoration may be a part of a project that includes other rehabilitation techniques, such as undersealing, diamond-grinding, and joint resealing. The proper sequence is to perform any undersealing first, followed by the LTR project, diamond-grinding, and joint resealing. If LTR is being performed in both lanes and traffic is diverted to the passing lane, damage can be caused to the passing lane. Repairs should first be done in the passing lane in these instances.

After the installation of the retrofitted dowel bars, deflection testing will document any change in load transfer efficiency. The installation of the dowels should increase the LTE to 90 to 100 percent when tested after a curing period of a few days. The dowel bar installations should also be monitored periodically after installation, using visual surveys and deflection testing to evaluate their long-term performance.

Successful installation of retrofitted dowel bars requires sound concrete adjacent to the joint or crack. If the concrete is deteriorated near joints or cracks, a full-depth repair should be placed rather than the installation of retrofitted dowel bars or deformed reinforcing steel at cracks.

10. EQUIPMENT

There are two recent developments in equipment used to cut the slots in the existing pavement. One device, used in the Washington State installation, is capable of cutting 6 slots in a joint simultaneously with a double saw cut; this device has a production rate of 80 to 100 joints per 12-hour shift. However, this procedure still requires the use of a jackhammer to break out the slot. An Indiana project used modified milling equipment with carbide tipped bits to mill three slots in the wheelpath at once. This technology has since been applied in a number of other projects and appears to be acceptable. If it shows good long-term performance, it could help to speed up the operation and further reduce LTR costs, as no additional break up of the concrete in the slot is required. Milling also provides a rougher surface on the sides of the slots, minimizing slot preparation work (such as sand blasting) and promoting better bond. More positive alignment controls and vacuum systems to collect the milled concrete are needed to improve the process, however. In one project, carbide grinding of concrete made with granite coarse aggregate was not successful, prompting redesign of the milling head. Another difficulty has involved the use of truck-mounted mobile mixers, which have not been successful in mixing the relatively small quantities of patch material required on at least one job.[10]

11. SUMMARY

The ability of a joint or crack to transfer load is a major factor contributing to the structural performance of the pavement. The consequences of poor load transfer are a loss in serviceability due to pumping, faulting, mid-slab cracking, and corner breaks. The measurement of load transfer should be a standard part of any rigid pavement evaluation.

JPCP constructed without dowels and subjected to heavy loadings usually have poor load transfer. Also, JPCP and JRCP with dowels or other mechanical load transfer devices can lose their load transfer capabilities after being subjected to millions of load repetitions.

Load transfer restoration is the placement of a mechanical device across a joint or crack to provide the necessary load transfer from one side of the joint or crack to the other. This is a rehabilitation technique that is receiving wider acceptance, and all indications are that it can extend pavement life, slow down the rate of faulting, and improve overall pavement performance. 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. While some improvements in the design and construction of shear devices have been made, their use is still considered experimental.

The decrease in deflection and stress in the slab as a result of load transfer restoration contributes to a substantial increase in the life of the joint or crack due to reduced pumping, faulting, and cracking. WSDOT estimates the expected performance life of retrofitted dowel bars to be between 10 and 15 years.[9] However, because the occurrence of voids beneath the slab near the joint results in very high bearing stresses, it is recommended that known voids be filled prior to load transfer restoration.

12. REFERENCES

1. Darter, M. I., E. J. Barenberg, and W. A. Yrjanson. 1985, "Joint Repair Methods for Portland Cement Concrete Pavements," National Cooperative Highway Research Program Report 281, Transportation Research Board, Washington, DC.

2. Snyder, M. B., M. J. Reiter, K. T. Hall, and M. I. Darter. 1989, "Rehabilitation of Concrete Pavements, Volume I—Repair Rehabilitation Techniques," FHWA-RD-88-071, Federal Highway Administration, Washington, DC.

3. Gulden, W., and D. Brown. 1985, "Establishing Load Transfer In Existing Jointed Concrete Pavements," Transportation Research Record 1043, Transportation Research Board, Washington, DC.

4. Tayabji, S. D. 1986, "Evaluation of Load Transfer Restoration Techniques and Undersealing Practices," FHWA/RD-86/043, Federal Highway Administration, Washington, DC.

5. Roman, R. J., M. Y. Shahin, and J. A. Crovetti. 1987, "Subsealing and Load Transfer Restoration," Transportation Research Record 1117, Transportation Research Board, Washington, DC.

6. Pierce, L. M. 1994, Portland Cement Concrete Pavement Rehabilitation in Washington State: Case Study, Transportation Research Record 1449, Transportation Research Board, Washington, DC.

7. Bernard, D. W. 1984, "A Construction Report on Reestablishing Load Transfer In Concrete Pavement Transverse Joints," Technical Report 84-6, New York State Department of Transportation, Albany, NY.

8. Jerzak, H. 1994, "Rapid Set Materials for Repairs to Portland Cement Concrete Pavement and Structures," Caltrans Division of New Technology, Materials and Research. Sacramento, CA.

9. Smoak, W. G., T. B. Husbands, and J. E. McDonald. 1997, "Results of Laboratory Tests on Materials for Thin Repair of Concrete Surfaces," TR REMR-CS-52. U.S. Army Corps of Engineers, Washington, DC.

10. Pierce, L. M. 1997, "Design, Construction, Performance and Future Direction of Dowel Bar Retrofit in Washington State," Proceedings, Sixth International Purdue Conference on Concrete Pavement, Volume 2.

11. Federal Highway Administration. 1991, "Design Review Report on Retrofit Dowels," Federal Highway Administration, Puerto Rico Division, Design Review Report.

12. Gulden, W. and D. Brown. 1987, "Improving Load Transfer in Existing Jointed Concrete Pavements," FHWA/RD-82/154, Federal Highway Administration, Washington, DC.

13. Hall, K. T., M. I. Darter, and J. M. Armaghani. 1993, "Performance Monitoring of Joint Load Transfer Restoration," Transportation Research Record 1388, Transportation Research Board, Washington, DC.

14. Federal Highway Administration. 1994, INFORMATION: SP204—Retrofit Load Transfer, Washington, DC.

15. Bendaña, L. J., and W-S. Yang. 1993, "Rehabilitation Procedures for Faulted Rigid Pavements," Transportation Research Record 1388, Transportation Research Board, Washington, DC.

16. Federal Highway Administration/American Concrete Pavement Association (ACPA). 1998, "Concrete Pavement Rehabilitation: Guide for Load Transfer Restoration," Co-Published as FHWA-SA-97-103 ACPA JP001P.Washington, DC/ Skokie, IL.

17. American Concrete Pavement Association (ACPA). 1996, "Construction of Portland Cement Concrete Pavements," FHWA HI-96-027, Federal Highway Administration, Washington, DC.

18. Research Society for Road and Transportation Concrete Road Working Group. 1985, "Memorandum for Preservation of Concrete Roads," (translated from German by the FHWA).

MODULE 4-10

SHOULDER REHABILITATION CONSIDERATIONS FOR PCC PAVEMENTS

1. INSTRUCTIONAL OBJECTIVES

This module presents rehabilitation considerations for shoulders adjacent to rigid pavements. Upon successful completion of this module, the participants shall be able to accomplish the following:

1. List the common distress types for shoulders constructed with both bituminous and portland cement concrete (PCC) materials.

2. Discuss procedures applicable to shoulder rehabilitation, and describe how the extent of deterioration of a shoulder can influence not only shoulder rehabilitation strategy selection, but also the rehabilitation of the mainline rigid pavement.

3. Describe the contributions that a tied PCC shoulder can have on the performance of the mainline rigid pavement.

2. INTRODUCTION

Shoulders are an integral and vital part of most highway pavements. They provide a safety zone for errant vehicles, an auxiliary area for emergency stops, and also can contribute to the performance of the mainline pavement. However, shoulders are generally designed to a lower structural capacity than the mainline pavement. While a few States estimate shoulder traffic as a percentage of mainline traffic and design the shoulder thickness based on that estimated traffic level, many States use standards or local experience to determine shoulder thicknesses. Thus, it is not surprising that widely different shoulder structural sections exist from one State to another.

Because of their reduced structural capacity, shoulders can often deteriorate rapidly due to encroaching and parked traffic. Therefore, due to their contribution to the overall safety of the roadway, the condition of the shoulders should be carefully evaluated as part of any pavement rehabilitation project. Most of the rehabilitation methods described for mainline pavements can be applied to the roadway shoulders as well. As with mainline pavements, an assessment of the shoulder condition should be made to identify feasible rehabilitation alternatives; these alternatives must then be considered in determining the most effective rehabilitation strategy for the entire project.

Shoulders adjacent to mainline rigid pavements may be constructed with either bituminous (consisting of an asphalt concrete surface or an asphalt surface treatment) or PCC materials. Although not always possible, it is generally recommended that shoulders be constructed of the same materials as the mainline pavement in order to facilitate construction and for overall compatibility in material responses to traffic and environmental forces. Furthermore, PCC shoulders adjacent to mainline rigid pavements provide lateral support to the mainline pavement, effectively reducing critical edge stresses and the associated development of transverse fatigue cracking in the mainline pavement. Finally, it may be desirable to construct a PCC shoulder in anticipation of it carrying traffic during some future rehabilitation of the mainline pavement; this will likely reduce user delays significantly during the rehabilitation of the facility.

3. DEFINITIONS

Shoulders may be classified as either improved or unimproved. Unimproved shoulders are aggregate-surfaced and are not discussed here. Hudson et al.[1] provide detailed information on the maintenance and rehabilitation of unimproved shoulders.

Improved or paved shoulders are those constructed with either a bituminous surface or a PCC surface. Paved shoulders are required on all interstate routes, and are justified by improved and smoother traffic operations, expectation of improved pavement performance, increased pavement life, enhanced highway safety, and reduced maintenance.[2] A recent survey of State highway agencies indicated that the majority of respondents have full paved shoulders for non-interstate freeways (94 percent) and for arterials (84 percent).[3]

4. PURPOSE AND PROJECT SELECTION

The purposes of highway shoulders are defined in a NCHRP synthesis on the design and use of highway shoulders:[4]

> *Shoulders are an important element of a highway system. Well designed and maintained shoulders are essential for safe traffic operations and serve as lateral structural support for the traveled way. Shoulders provide space for emergency stops, recovery space for errant vehicles, clearance to signs and guard rails, space for maintenance operations, and other advantages.*

The methods used for determining the condition and needs of highway shoulders are the same as those used for mainline pavement. The major items of interest, as described in block 2, are:

- Distress survey.
- Structural evaluation.
- Drainage survey.
- Subgrade and materials evaluation.
- Traffic survey.

As with the mainline pavement, this information can be used to develop rehabilitation alternatives to be applied to the shoulders. Shoulder rehabilitation techniques are essentially the same as those for mainline pavements.

The majority of shoulder improvements are made at the same time as improvements are made to the mainline pavement. In these cases, it is appropriate to collect distress data for shoulders at the same time data are being collected for the mainline pavement. Alternatives for shoulder restoration should be formulated at the same time as options are developed for the mainline pavement, and some effort should be made to ensure that the operations on the shoulder can be accommodated without interrupting the mainline operations.

Distresses to be noted when performing the pavement survey are the same as those which would be noted for a mainline pavement of the same surface type (see modules 2-2 and 2-3). In addition to those distresses, the condition of the joint between the shoulder and the pavement edge should be noted. Of particular concern is the drop-off from the pavement to the shoulder and the condition of the joint. Excessive drop-offs cause a dangerous condition for vehicles that need to move from the shoulder to the roadway at high speed. Water infiltration at the joint can contribute to weakened subgrade support, faulting, corner breaks, and excessive drop-offs to the shoulder. The cause and extent of the distress and the structure of the shoulder must be known before a rehabilitation procedure can be selected.

In the following sections, the two different shoulder types adjacent to mainline PCC pavements are discussed, along with the typical distresses that they may exhibit.

Rigid Pavement with Bituminous Shoulder

During the interstate construction of the 1960s and early 1970s, many mainline rigid pavements were constructed with bituminous shoulders whose surface consisted of either a surface treatment or a layer of hot-mix asphalt (HMA). This was often done as a means of reducing construction costs. While current construction practices are moving away from that design, there are still many existing pavements that incorporate that combination.

When a rigid pavement is constructed with a bituminous shoulder, the resulting lane-shoulder joint is one of the most difficult joints to seal.(5,6) This is because the differences in the thermal properties of the two materials can create large horizontal movements at that joint. In addition, large vertical movements can be created at the joint due to differential frost heave that occurs because of the differences in the cross-section of the two materials. The longitudinal lane-shoulder joint must be effectively sealed in order to reduce the amount of water that infiltrates the pavement structure at that joint. One study estimated that about 70 percent of the water gains entry into the pavement structure through the lane-shoulder joint.(5) Excessive moisture in the pavement can lead to cracking, corner breaks, pumping, faulting, settlements, and shoulder separation.

In addition to the difficulty in sealing and maintaining the lane-shoulder joint, the bituminous shoulder in this configuration does not provide any lateral support to the mainline pavement. Thus, traffic loading at the slab edge can produce very high stresses that can quickly consume the fatigue life of the pavement. Also, fatigue damage due to traffic encroachments can occur in the bituminous shoulder, which creates entry points for water to infiltrate the pavement structure. Critical distresses displayed by this pavement/shoulder combination are listed below:

1. Pumping. Pumping is the ejection of water and fines up through the lane-shoulder joint, resulting in a loss of support under the pavement and faulting. This erosion may occur under both the shoulder and the mainline pavement, and the erosion will accelerate faulting in a jointed concrete pavement.

2. Fatigue Cracking. Insufficient shoulder thickness or heavy traffic loading will create excessive stresses in the shoulder, especially when the foundation materials have softened under the lane-shoulder area. This increased deformation will produce accelerated fatigue cracking in both the mainline pavement and in the shoulder.

3. Lane-Shoulder Drop-off. Consolidation or settlement of the underlying granular base or subgrade under loads and voids created by pumping will combine to produce a settlement of the shoulder.

4. Block Cracking. Block cracking is caused by shrinkage of the asphalt and daily temperature cycling. It is accelerated by hardening and oxidation of the asphalt and a lack of traffic use. The development of block cracking creates entry points for water to infiltrate the shoulder and pavement structure.

5. Shoving. Shoving of the bituminous shoulder can occur if the mix is unstable or if there are extremely large shearing forces (e.g., braking or turning tires) present.

6. Localized Failures Due to Differential Shoulder Support. Shoulders constructed with a different, or lesser, base and or subbase than the mainline pavement can be subjected to frost heave different from that occurring on mainline pavements in some areas. This can accelerate breakdown of the shoulder adjacent to the mainline pavement.

7. Weathering and Raveling. Weathering refers to the oxidation of the asphalt cement in a bituminous surface. This oxidation causes the asphalt to harden, after which the aggregates can easily become dislodged and lead to raveling of the surface.

If an effective seal can be maintained at the lane-shoulder joint, many of these distresses can be substantially reduced.

Rigid Pavement with PCC Shoulder

While PCC shoulders have been constructed adjacent to rigid pavements for many years, it has only been in the last decade that its use has been more prevalent on new PCC construction. These shoulders are either paved after the construction of the mainline pavement or are paved integrally with the mainline pavement. In either case, tiebars are placed across the lane-shoulder joint to prevent separation and to provide lateral edge support to the mainline pavement.

This type of design configuration serves to address many of the problems associated with a bituminous shoulder adjacent to a rigid pavement. Not only is an easily sealed and maintained lane-shoulder joint established, the PCC shoulder is expected to provide lateral support to the mainline pavement. This can reduce critical slab edge stresses and can increase the life of the mainline pavement.

PCC shoulders are evaluated using the same procedures as used on a mainline rigid pavement. Distresses must be recorded, the structural history reviewed, and deflection data collected, if applicable, most likely in conjunction with the evaluation work done on the mainline pavement. It is important that the load transfer efficiency of the existing tiebars be evaluated in a manner similar to the evaluation of the load transfer efficiency of transverse joints in the mainline pavement to quantify the structural adequacy of the slabs.

Distresses that occur in a PCC shoulder are virtually the same as those that can occur in a mainline rigid pavement. Some of the primary distresses include:

1. Cracking. Cracking in the PCC shoulder can occur due to fatigue, poor support conditions at corners, and sympathetic cracking from the mainline pavement. Therefore, joints constructed in the shoulder should match those in the mainline. Sawing intermediate joints in the shoulder has been shown to encourage cracking in the mainline.

2. Pumping/Faulting. Pumping and faulting can occur at the corners of both the PCC shoulder and the mainline rigid pavement. If good load transfer efficiency exists across the lane-shoulder joint, then the corner and edge deflections will be greatly reduced. However, tied shoulders by themselves are not a substitute for positive load transfer devices in the transverse joints of the mainline pavement.[7]

3. Spalling. Spalling of the lane-shoulder joint may occur for a variety of reasons, including inadequate cover of the tiebars, inadequate shoulder thickness resulting in differential vertical movements, failure of a keyway (if present) at the lane-shoulder joint, and poor consolidation of the concrete surrounding the tiebars, particularly on retrofitted PCC shoulders.

Many of the distresses that may occur in a tied PCC shoulder are believed to be caused by an insufficient tiebar design or by poor construction practices.[8,9] Thus, tiebars should be of sufficient diameter (generally a minimum of 16 mm) and spaced sufficiently close (maximum spacing of 760 mm) to resist shear forces under heavy loading.[10]

5. LIMITATIONS AND EFFECTIVENESS

It is generally recommended that mainline rigid pavements include a shoulder of like material for reduced maintenance and improved performance. Bituminous shoulders provide no lateral support to the mainline rigid pavement, and thus do not contribute to the structural performance of the mainline pavement. However, it is recognized that many mainline rigid pavements contain a bituminous shoulder and, due to their age and overall condition, are not prime candidates to receive a retrofitted PCC shoulder. In these instances, the bituminous shoulder should be maintained until removal or reconstruction of the mainline pavement.

Widened truck lanes and/or tied PCC shoulders are recommended as part of most new rigid pavement designs. Table 4-10.1 provides general recommendations for shoulder types for new rigid pavements. Again, however, the inclusion of PCC shoulders on an existing rigid pavement must be carefully considered in light of its overall pavement condition and expected remaining service life.

Table 4-10.1. General recommendations on use of shoulder type for new rigid pavements.[10]

Roadway Functional Class	**Recommended Shoulder Type/Edge Support**	
	Rural	**Urban**
Interstates/Freeways	1. Widened Lane with HMA Shoulder 2. PCC Shoulder (paved monolithically)	1. PCC Shoulder (paved monolithically) 2. Widened Lane with PCC Shoulder
Arterials	1. Widened Lane with HMA Shoulder 2. PCC Shoulder 3. None (HMA Shoulder)	1. PCC Shoulder (paved monolithically) 2. Widened Lane with PCC Shoulder
Collectors	1. None (HMA, Gravel, or Turf Shoulder)	1. PCC Shoulder 2. Curb and Gutter
Locals	1. None (Gravel or Turf Shoulder)	1. PCC Shoulder and Curb and Gutter 2. Curb and Gutter 3. None (HMA, Gravel, or Turf Shoulder)

Regarding the effectiveness of PCC shoulders, one study analyzed the stress in a PCC shoulder and the influence on the fatigue life of the mainline pavement.[11] The results of that study are shown in figure 4-10.1. A load transfer of zero indicates a free edge condition, or a condition where a bituminous shoulder would exist contiguous to the mainline pavement. When the load transfer is low, the stresses produced in the slab are high. If a good tie between the slab and shoulder is achieved, the load transfer will be high and the stresses in the slab will be decreased.

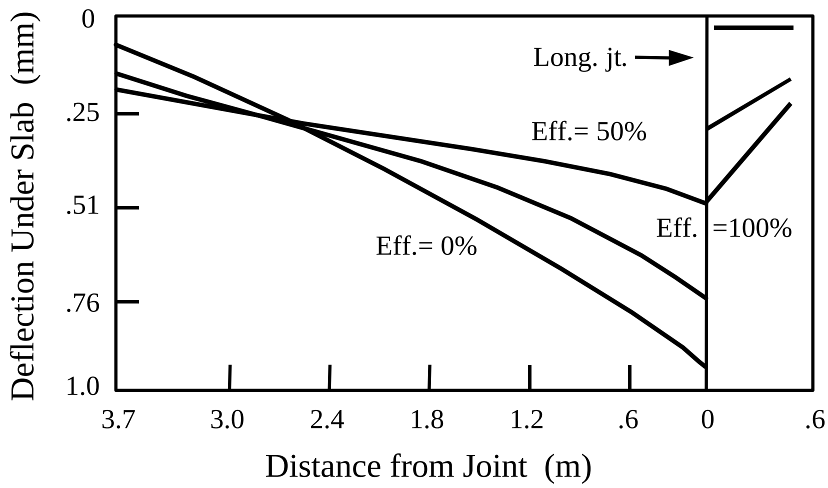


Figure 4-10.1. Effect of tied PCC shoulder on slab deflections.[11]

However, the inclusion of PCC shoulders on a mainline rigid pavements does not necessarily guarantee an improvement in overall performance. For example, the use of tied PCC shoulders has grown considerably in the last few years, but evaluations of their effectiveness have shown mixed results. In one study, low load transfer across the lane-shoulder joint was observed on several sections, suggesting an ineffective tie bar system.[9] Furthermore, the tied shoulders did not appear to have any effect on reducing joint faulting. Similarly, in a study of continuously reinforced concrete pavements (CRCP) performance, it was found that the tied PCC shoulders did not appear to have a significant impact on reducing edge deflections, again perhaps due to a poor tie bar design.[12] Finally, the effectiveness of tied PCC shoulders in improving pavement performance must be compared to the effectiveness of other rehabilitation activities. For example, a study in Washington State indicated that although the addition of tied PCC shoulders increased transverse joint load transfer and decreased corner deflections, their placement by themselves was not as effective as the addition of retrofitted load transfer devices.[7]

6.   DESIGN CONSIDERATIONS

The design considerations for most rehabilitation activities for mainline pavements also apply to the application of those strategies on the shoulder. The pertinent module in this reference book should be consulted for information on the design considerations for a specific rehabilitation measure.

For retrofitted PCC shoulders, the method of construction and the competency of the tie bar system are key elements. Generally, retrofitted PCC shoulders are tied to the existing mainline rigid pavement by drilling holes in the mainline pavement and anchoring tie bars before placing the PCC shoulder. A few agencies have experimented with placement of a PCC shoulder not tied to the mainline pavement. Rarely used, this design provides little (if any) lateral support to the mainline pavement and is not effective.[13] Consequently, this design is not recommended.

The tie bar system used to secure the PCC shoulder to the mainline pavement is a critical component of their design; of particular importance is the tie bar diameter and the spacing between adjacent tie bars. A minimum tiebar diameter of 16 mm is recommended for design; larger diameter bars may be required on heavy truck routes, particularly on ramps, truck lanes, or other merging areas where trucks frequently cross the longitudinal joints.[10] A maximum spacing limit of 760 mm is recommended in order to provide adequate support to the mainline pavement.

Tie bars should be deformed in order to effectively tie the shoulder to the mainline pavements. Tie bar lengths of 760 mm are recommended, as is epoxy-coating of the bars to help prevent corrosion. The tiebars should be strongly anchored to the existing pavement using an approved anchoring material. The use of pull-out tests of tie bars is suggested to ensure sufficient embedment lengths and to ensure adequate bonding.

AASHTO[14] , Majidzadeh and Ilves[15], and Sawan et al.[16] provide procedures for designing PCC shoulders for mainline rigid pavements. However, it is recommended that the thickness of the PCC shoulder be the same as that of the mainline pavement. This helps reduce the possibility of differential frost heave in cold regions[17] and also provides a sufficient structure that can be used as a travel or emergency lane during maintenance or rehabilitation activities.

The transverse joints in the PCC shoulder should match those of the mainline pavement in order to prevent “sympathetic” cracking in the mainline pavement from joints in the shoulder. If the PCC shoulders are envisioned to carry significant truck traffic at some point in time (during rehabilitation of the mainline pavement, for example), consideration should be given to doweling the joints.

7. COST CONSIDERATIONS

Shoulder restoration procedures will generally use techniques discussed in earlier modules of this reference manual. Costs of techniques to be used for shoulder restoration will be similar to those costs reported for similar techniques performed on mainline pavements and reported in earlier modules.

Retrofitting of PCC shoulders to existing mainline pavements will vary considerably depending upon local conditions and material availability. Typical unit costs may range from about \$20/m^2 to \$50/m^2.

8. CONSTRUCTION

Rehabilitation of pavement shoulders will generally call for techniques used on mainline pavements of the same type. The appropriate module should be referred to for more information on a specific rehabilitation technique. Installation of a retrofitted PCC shoulder will involve the following steps:

1. Removal of the existing shoulder.
2. Drilling of appropriately sized holes along the exposed longitudinal face of the mainline pavement.
3. Grouting of the tie bars into the holes with an approved anchoring material.
4. Paving of the new shoulder using conventional rigid paving equipment.

A detailed guide specification on the construction of retrofitted tied PCC shoulders is found in work by Darter et al.[18]

10. SUMMARY

General considerations for the rehabilitation of shoulders adjacent to mainline rigid pavements are presented in this chapter. Each shoulder type exhibits certain distresses that should be considered in a pavement rehabilitation. Generally, it is recommended that the shoulder be evaluated as part of the mainline pavement evaluation and rehabilitation alternatives be identified. However, there may be circumstances in which the type of rehabilitation to be received by the mainline pavement would override the rehabilitation selection for the shoulder (e.g., structural overlay of mainline pavement).

It is important that, where practical, the shoulder materials be matched to those used in the mainline pavement. Of particular importance is the edge support provided by PCC shoulders to mainline rigid pavements. Not only is an easily-maintained lane-shoulder joint established, but also the PCC shoulder helps reduce the magnitude of the edge and corner stresses, thereby extending the structural life of the mainline pavement. Tied PCC shoulders normally will not significantly reduce joint pumping or faulting problems.

The same rehabilitation procedures that can be used on a mainline pavement are also applicable to the shoulder. It is important that, in selecting the rehabilitation alternatives, consideration be given to the extent of the distress and to the condition of the mainline pavement.

11. REFERENCES

1. Hudson, S. W., B. F. McCullough, and R. Frank Carmichael III. 1987, "Surface Design and Rehabilitation Guidelines for Low Volume Roads," FHWA/TS-87/225, Federal Highway Administration, Washington, DC.

2. Federal Highway Administration (FHWA). 1990, "Paved Shoulders," FHWA Technical Advisory T 5040.29, Federal Highway Administration, Washington, DC.

3. Federal Highway Administration (FHWA). 1992, "Shoulder Treatment Study," FHWA Technical Memorandum, Federal Highway Administration, Washington, DC.

4. Transportation Research Board (TRB). 1979, "Design and Use of Highway Shoulders," NCHRP Synthesis of Highway Practice 63, Transportation Research Board, Washington, DC.

5. Barksdale, R. D., and R. G. Hicks. 1979, "Improved Pavement-Shoulder Joint Design," NCHRP Report No. 202, Transportation Research Board, Washington, DC.

6. Carpenter, S. H., M. R. Tirado, E. H. Rmeili, and G. L. Perry. 1987, "Methods for Shoulder Joint Sealing, Volume I—Serviceability Requirements," FHWA/RD-88/002, Federal Highway Administration, Washington, DC.

7. Pierce, L. M. 1994, "Portland Cement Concrete Pavement Rehabilitation in Washington State: Case Study," Transportation Research Record No. 1449, Transportation Research Board, Washington, DC.

8. Smith, K. D., D. G. Peshkin, M. I. Darter, A. L. Mueller, and S. H. Carpenter. 1990, "Performance of Jointed Concrete Pavements, Volume I—Evaluation of Concrete Pavement Performance and Design Features," FHWA-RD-89-136, Federal Highway Administration, Washington, DC.

9. Smith, K. D., M. J. Wade, D. G. Peshkin, L. Khazanovich, H. T. Yu, and M. I. Darter. 1995, "Performance of Concrete Pavements, Volume II—Evaluation of In-Service Concrete Pavements," FHWA-RD-95-110, Federal Highway Administration, Washington, DC.

10. Yu, H. T., M. I. Darter, K. D. Smith, J. Jiang, and L. Khazanovich. 1996, "Performance of Concrete Pavements, Volume III—Improving Concrete Pavement Performance," FHWA-RD-95-111, Federal Highway Administration, Washington, DC.

11. Sawan, J. S. and M. I. Darter. 1979. "Structural Design of PCC Shoulders," Transportation Research Record No. 725, Transportation Research Board, Washington, DC.

12. Tayabji, S. D., P. J. Stephanos, J. S. Gagnon, and D. G. Zollinger. 1994, "Performance of CRC Pavements, Volume 2: Field Investigations of CRC Pavements," Final Report, Federal Highway Administration, Washington, DC.

13. Wells, G. K. and W. A. Nokes. 1990, "Field Review—PCC Shoulder Performance Near Geyserville," Minor Research Report 65328-637378-30088, California Department of Transportation, Sacramento, CA.

14. American Association of State Highway and Transportation Officials (AASHTO). 1993, "Guide For Design Of Pavement Structures," American Association of State Highway and Transportation Officials, Washington, DC.

15. Majidzadeh, K. and G. J. Ilves. 1986, "Structural Design of Roadway Shoulders—Final Report," FHWA/RD-86/089, Federal Highway Administration, Washington, DC.

16. Sawan, J. S., M. I. Darter, and B. J. Dempsey. 1982, "Structural Analysis and Design of Portland Cement Concrete Highway Shoulders," FHWA/RD-81/122, Federal Highway Administration, Washington, DC.

17. Vyce, J. M. 1988, "A Summary of Experimental Concrete Pavements in New York," FHWA/NY-RR-88/141, New York State Department of Transportation, Albany, N.Y., 1988.

18. Darter, M. I., E. J. Barenberg, and W. A. Yrjanson. 1985, "Joint Repair Methods for Portland Cement Concrete Pavements," NCHRP Report No. 281, Transportation Research Board, Washington, DC.

MODULE 4-11

RETROFITTED EDGE DRAINS

"Now in reference to the question of drainage, if we want to drain a road, we must get the water out of the road; we must get it off of the road; and away from the road! When we have done that, we have a well drained road. If there is not drainage, there will be no road, no matter what the material may be." (James MacDonald, Connecticut State Highway Commissioner (1909) quoted by Gendell).[1]

1. INSTRUCTIONAL OBJECTIVES

This module presents the fundamental principles of pavement subdrainage design and focuses on the application of those principles to a rigid pavement rehabilitation project. After completion of this module, the participant should be able to accomplish the following:

1. Identify the sources of water in pavement systems.
2. Name and identify the function of the major components of subdrainage systems.
3. Discuss the criteria for selection of a filter system (fabric or granular).
4. List the basic subdrainage design steps.
5. Discuss how and why the basic design steps are modified for rigid pavement rehabilitation projects.

2. INTRODUCTION

It has long been recognized that water is a major cause of pavement distress. The best time to address this problem is during construction, when effective and durable subdrainage systems can be built into a pavement for an increase of 15 to 25 percent over conventional designs. However, in some cases, older pavements without effective drainage can be modified to improve overall drainage conditions. Because of the significant contribution of drainage to pavement performance and the prevalence of moisture-related distress, every pavement rehabilitation project design should consider the need for improved pavement subdrainage.

In recognition of the importance of subdrainage, the Federal Highway Administration (FHWA) developed Demonstration Project 87 (Demo 87), *Drainable Pavement Systems—Participant Notebook*.[2] This document summarizes the pavement community's current understanding of the role of subdrainage in pavement performance, and provides guidelines on the application of subdrainage in both new design and rehabilitation. Factors that are emphasized include:

- The primary source of water in pavements is through nonsealed joints and cracks. While sealing will not keep out all moisture, it will reduce the amount entering a pavement.

- Pavement subdrainage systems should be used to remove any water that does get into a pavement system.

- Adequate pavement and shoulder cross-slopes should be provided to ensure that water does not stand on the pavement surface or at the pavement-shoulder interface. If cross-slopes are inadequate, steps should be taken to improve them through the application of a thin overlay or through variable milling.

- Tight, tied concrete shoulders on concrete pavements provide a joint that is easily sealed and maintained, and eliminates a major source of water infiltration.

The FHWA's emphasis on drainage is demonstrated in other ways as well. Experimental Project 12, *Concrete Pavement Drainage Rehabilitation*, evaluated the performance of edge drains in 10 States, and concluded that in many cases retrofitted edge drains are added beyond the point in the pavement's life when they can improve performance.(3) The FHWA is also sponsoring the development of a training course, *Pavement Subsurface Drainage Design*.(4) This 3-day training program is designed to communicate to pavement engineers the information necessary to evaluate the need for subdrainage, to design these systems, and to monitor and maintain them.

Assessing the need for subdrainage in a rigid pavement rehabilitation project begins with an evaluation of the existing distresses and with the overall project evaluation (module 2-6). If warranted and feasible, a retrofitted subdrainage system may be designed for the existing pavement. The successful completion of these activities requires a knowledge of the potential sources of water, how subdrainage materials interact with water, and the basic principles of subdrainage design.

3. DEFINITIONS

There are several different types of drainage systems that may be associated with a pavement structure, with longitudinal edge drains perhaps being the most common. These are either circular slotted pipes or rectangular geocomposite membranes that are placed longitudinally along a project at the pavement edge. They are intended to collect water from the pavement structure and carry it to regularly spaced outlets for discharge into a ditch or other drainage structure. Longitudinal drains can be either placed during construction or can be retrofitted on existing pavements.

Transverse drains are circular, slotted pipes that are placed transversely across the pavement, perpendicular to the centerline of the pavement to remove excess moisture from beneath the pavement. Some agencies have used transverse drains beneath transverse joints, a primary location of moisture. Transverse drains may be a part of new construction, but are not normally used in rehabilitation (except in reconstruction). However, they are sometimes installed at full-depth repair locations.

Permeable bases are another means of providing positive drainage to a pavement structure during new construction. These are bases placed below the PCC slab and are designed and constructed so that excess moisture can rapidly drain through the base and to nearby edge drains. To accomplish this, the granular materials in permeable bases typically are open-graded with very few fines (material passing the 0.074 mm sieve). Permeable bases are only an option for rehabilitation when the entire pavement structure is to be reconstructed.

An important factor influencing the effectiveness of a permeable base is its permeability. Permeability is the capacity of a material to conduct or discharge water under a given hydraulic gradient. It is expressed in terms of a coefficient of permeability, k, which is a measure of the rate at which water passes through a material in a given amount of time under a unit hydraulic gradient. Permeability should not be confused with porosity, which is the amount of void space contained within a material.

4. PURPOSE AND PROJECT SELECTION

The purpose of a pavement drainage system is to remove excess water that infiltrates the pavement structure. A field survey to assess the condition of the existing pavement is the first step in determining whether a drainage system may be needed. The existence of moisture-related distresses may be an indication that improved drainage is necessary. Table 4-11.1 shows distresses in rigid pavements that may be caused by moisture problems and whether they may be corrected by retrofitting subdrainage.

Table 4-11.1. Ability of various moisture-related rigid pavement conditions to be addressed by retrofitted drainage.[5]

		Retrofitted Drainage	
Observed Condition	**Moisture Problem**	**Corrects**	**Prevents Further**
Spalling	Possibly	No	Possibly
Scaling	Possibly	No	No
D-Cracking	Yes	Possibly	No
Pumping	Yes	Possibly	Possibly
Faulting	Yes	Possibly	Possibly
Curling/Warping	Yes	No	No
Corner Cracking	Yes	Possibly	Possibly
Diagonal/Transverse/ Longitudinal Cracking	Possibly	Possibly	Possibly
Punchout	Yes	Possibly	Possibly

The presence of one or more of these distresses alone is not sufficient to justify a retrofitted subdrainage project, however. The FHWA[6] suggests that a suitable pavement for subdrainage rehabilitation be in fairly good overall condition (i.e., no more than 5 percent of the outer lane requiring full-depth repair). Studies have shown that if the pavement is severely cracked or has broken slabs, retrofitted edge drains may not be an appropriate rehabilitation technique.[7,8,9] It is also recommended that if the base has greater than 15 to 20 percent fines (material passing the 0.074 mm sieve), the base material may be too impermeable for an effective retrofitted subdrainage installation.

For rehabilitation projects, it is acknowledged that most existing pavements have impermeable layers. The best that can be done for such pavements is to provide a means for the rapid, effective removal of water that may accumulate at the slab-base interface through the installation of retrofitted longitudinal edge drains.

5. LIMITATIONS AND EFFECTIVENESS

A major concern in retrofitting edge drains is identifying projects where they are appropriate and cost effective. Although there are conditions in which retrofit edge drains can be effective in slowing or arresting moisture, in some projects they have actually accelerated damage.(10,11) For example, Georgia found that in some cases, edge drains appear to have trapped moisture along the pavement edge and caused increased deterioration.(10)

For retrofitted subdrainage to be cost-effective, it must reduce deterioration and extend the pavement's life. A study by the Permanent International Association of Road Congresses (PIARC) investigated the effectiveness of edge drains in reducing pumping when combined with nonerodible materials.(12) That study indicated that care must be taken to ensure that the drains are needed, adequately designed, and properly installed in a pavement if the pavement's performance is to be improved. The report emphasizes that drains should not be installed indiscriminately. Many failures of pavements with subsurface drainage can be attributed to poor design and construction practices.(13)

The FHWA's Experimental Project 12 looked at a number of retrofitted edge drain systems.(14) Most of the drains included in this FHWA study are continuous pipe collector systems, but some still use aggregate filled trenches (also called french drains). At the time of the initial report, the use of geocomposite edge drains was experimental in eight of ten projects. It was reported that most States add subdrainage in rigid pavement rehabilitation to extend pavement life, and that the costs of adding subdrainage ranged from approximately $4.10 to $39.00 per linear meter, although they were primarily in the range of $6.50 to $9.80 per linear meter. At the time of the report, the greatest controversy was in the use of geotextiles as trench filters and where to place them. The type of material to use as backfill in the trench was another issue of concern.

For retrofitted projects, the trend has been to use geocomposite edge drains (also known as fin drains). The main reason is their ease of installation and associated reduction in costs. These edge drains use a narrower trench than conventional pipe drains and can be placed in a continuous operation, thereby reducing excavation costs and time. These advantages translate into a lower installed cost. However, geocomposite drains can clog and be easily damaged during construction. In addition, they can not be cleaned out as part of maintenance operations. For these reasons, some States have discontinued the use of geocomposite drains.

Kentucky, which has been installing edge drains for over 20 years, has in the past 6 years almost exclusively used geocomposite edge drains.(15) In side-by-side comparisons with pipe drains, they report a number of interesting findings. The geocomposite edge drains were found to start draining much more rapidly than pipe drains after a rainfall event—a few minutes compared to 24 to 48 hours. However, in studies done by both excavation and borescope, it was found that some damage to the geocomposite drains had occurred as a result of the compactive effort used on the backfill.

On one project where the existing pavement was cracked and seated, widespread failures were reported for geocomposite edge drains.(15) These were the result of a number of causes, including damage to the drains during placement, and silting from the fines created during the cracking and seating process. Based on their study, Kentucky then made a number of changes to their specifications for retrofitted subdrainage. They restricted the distance between pipe outlets to either 60 or 140 m, depending on the slope of the drainage trench (they found that the longer distances would result in the capacity of the drain being exceeded), and they also learned that they needed to take extra care with edge drains on crack and seat projects. More recent modifications to their installation method, discussed later, include placing the geocomposite on the shoulder side of the trench and backfilling with a sand slurry.(16)

Koerner et al.[17] report similar findings regarding the use of geocomposite edge drains. Out of 41 installations that were evaluated, the performance of 10 was unacceptable. This contrasts with the performance of other types of edge drains, which was 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.

These and other problems have led some States to discontinue the use of geocomposite edge drains. Illinois conducted an extensive evaluation of the drainage design policies and found numerous examples of improper design, construction, and maintenance.[18] Indiana, Michigan, Wisconsin, and Illinois are among the States that have reportedly discontinued the use of geocomposite edge drains due to their decreased service life and high initial costs.

6. DESIGN CONSIDERATIONS

Components of a Subdrainage System

Pavement subdrainage systems may be grouped into three general categories based on their geometry: longitudinal drains, permeable bases (also referred to as open-graded bases), and transverse drains. Most positive drainage is provided by longitudinal drains (pipe or geocomposite drains) and permeable bases, as shown in figure 4-11.1. A longitudinal drain is usually located near the pavement traffic lane edge and runs parallel to the roadway centerline. When installed along the pavement edge, this type of drain is most effective at removing water that infiltrates from the surface to the base and subbase layers. Longitudinal drains can also be used to lower or intercept groundwater, although this application is not recommended. Longitudinal drains can be constructed with geocomposite materials or perforated pipes. Aggregate-filled trenches, even those lined with a geotextile fabric, are not recommended due to their reduced capacity and inability to be maintained.

A permeable base consists of a layer of highly permeable material, normally extending under the entire pavement width. Their use is only applicable in new pavement construction or in reconstruction. Permeable bases are used for the rapid removal of both groundwater and surface infiltration from the pavement system. Although daylighted sections (where the permeable base extends under the pavement to the edge slope or ditchline) are a common feature in many pavements built in the past, it is generally acknowledged that such designs are susceptible to clogging and subsequently do not function well. When clogged, the permeable base acts as a reservoir rather than a drain, and can accelerate moisture-related damage. Consequently, permeable bases should only be used in conjunction with longitudinal edge drains.

Transverse drains are placed laterally under the pavement, usually at right angles to the centerline. These are most commonly used in conjunction with a drainage blanket to intercept and remove large quantities of water where the direction of flow is generally parallel to the pavement centerline, such as on steep highway grades. They have also been used in the past at individual joints as a means of removing water that caused pumping. Because of a number of failures involving the use of this type of drain, transverse drains are not commonly used in pavement rehabilitation.

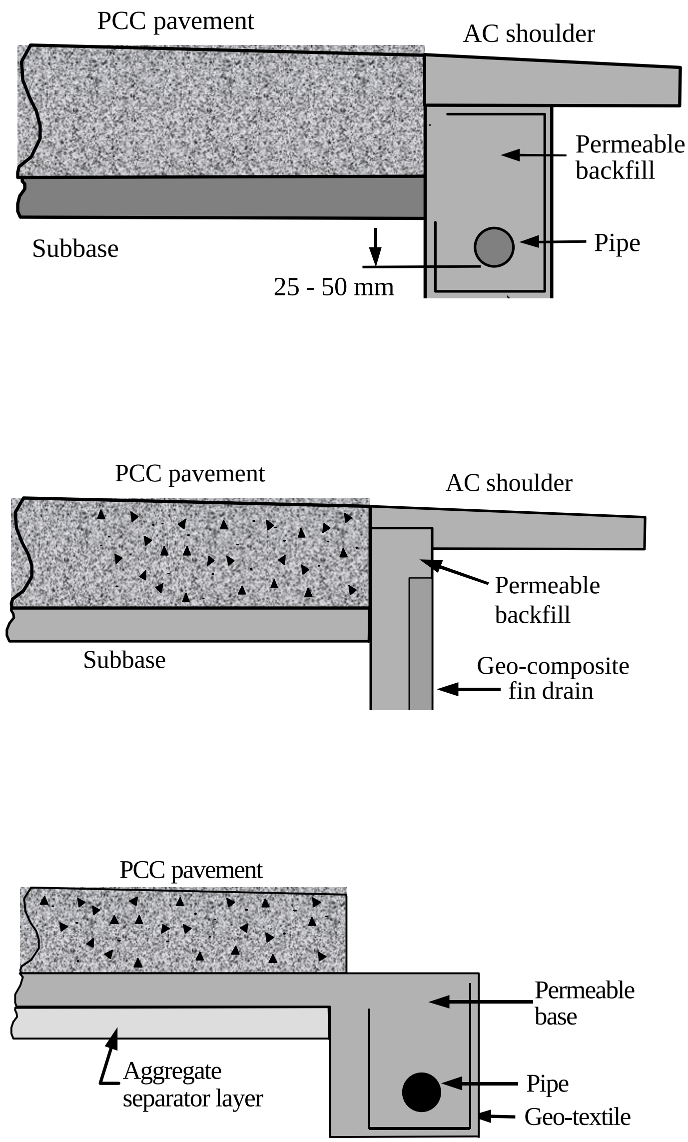


Figure 4-11.1. Typical views of pavement subdrainage features, including longitudinal edge drains and permeable base layers.

Basic Principles of Subdrainage Design

<u>Sources of Water</u>

An understanding of the potential sources of water is fundamental to the decision to retrofit subdrainage and to design a drainage system. Five sources of water in pavements are illustrated in figure 4-11.2 and are described below:

1. Seepage from higher ground. This source may be significant in cut sections where ditches are shallow and in areas where poorly drained ditches hold water.

2. Groundwater table rising into the pavement. Seasonal fluctuations of the water table (most commonly in the spring and winter) can be a significant source of water.

3. Surface infiltration of water (primarily entering through joints and cracks). This can be a very significant portion of the water in a pavement. Cedergren found that during a normal rainfall, 33 percent to 67 percent of the precipitation could enter the pavement system through surface infiltration.[19] The actual amount entering the pavement is limited by the ability of the pavement to store and remove the water.

4. Capillary movement of water from the water table. Like water absorbed by a paper towel, surface tension and capillary action can transport water well above the water table, saturating the subgrade and adding water to pavement layers. Typical values for capillary rise are 1.2 to 2.4 m for sandy soils, 3 to 6 m for silty soils, and in excess of 6 m for clayey soils. This method of water transport is responsible for frost heave damage.

5. Vapor movement of water. Temperature changes can cause water vapor present in air voids to migrate and condense. Typically, vapor water is negligible and not significant in subdrainage design.

In general, shallow geocomposite and pipe edge drains are used to address surface infiltration water only. Likewise, this module only addresses the use of retrofitted drainage to remove water that infiltrates through joints and cracks. Other sources of water are considered geotechnical problems at specific locations and are generally better addressed with deep underdrains. However, Iowa has used a 1200-mm deep underdrain system to address multiple sources of water. The system, which is installed about one year prior to recycling the rigid pavement into a base, has been shown to increase subgrade strengths and improve pavement performance.[20]

Methods for Reducing Moisture Effects

There are three general approaches available to the designer for accommodating water and reducing its potentially damaging effect on the pavement:

1. Keep the water out. This approach involves sealing the pavement surface, installing interceptor drains to cut off subsurface water, and using impervious membranes to prevent water intrusion.

2. Desensitize the pavement. In this approach, stabilized materials that are relatively insensitive to moisture are used in the pavement structure. It also may include measures taken to reduce the magnitude of pavement deflections.

3. Drain the pavement. Subdrains and drainable materials may be used to remove the water from the pavement before significant moisture damage can be initiated.

Sources of Moisture in Pavements

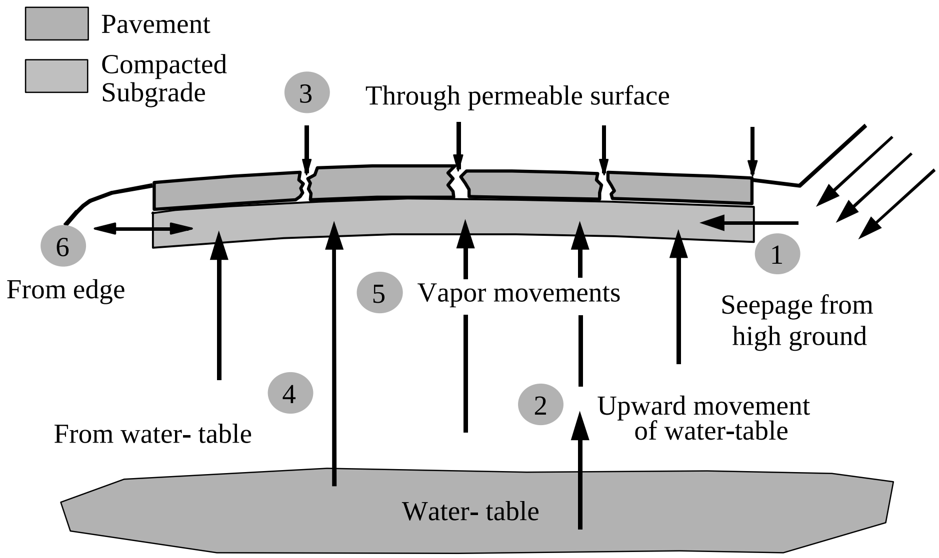


Figure 4-11.2. Sources of moisture in pavements.

Each approach should be considered in PCC pavement rehabilitation projects, as it is unlikely that any one approach can be totally successful in controlling all water-induced distresses. The optimum design may well involve a combination of more than one approach (e.g., crack and joint resealing plus retrofitting edge drains).

Subdrainage System Design

When a pavement is totally reconstructed, the designer has the option of selecting all components of the pavement system. Therefore, subdrainage design for reconstruction projects is the same as the design for new construction. Methods for pavement subsurface drainage analysis and design are available from many sources. See references 2, 4, 19, 21, and 22.

In pavement rehabilitation projects, pavement layers are already in place and little can be done to improve their ability to drain. As a result, the only reasonable way to improve subsurface drainage is to shorten the drainage path. Consequently, improving a pavement's drainage capabilities most frequently involves the design and construction of longitudinal drains retrofitted to the existing cross-section (see figure 4-11.3). The design steps for the typical drainage retrofit project are presented below.

Drainage Survey and Evaluation

A drainage survey and evaluation should be conducted (module 2-6) to determine the need for improved drainage and to evaluate the potential for improving the drainage. The existing cross-section is also examined to determine the best design and location for installing a drainage system.

Geocomposite Edge Drains

One recent development in pavement subdrainage is the use of geocomposites, which are combinations of geotextile fabrics and structural plastic. The most widely used product is the vertical geocomposite edge drain. This product consists of a filter fabric wrapped around a supporting plastic core, shown installed in figure 4-11.1 (b). The fabric serves as the filter and the voids created by the plastic core serves as the drainage medium.

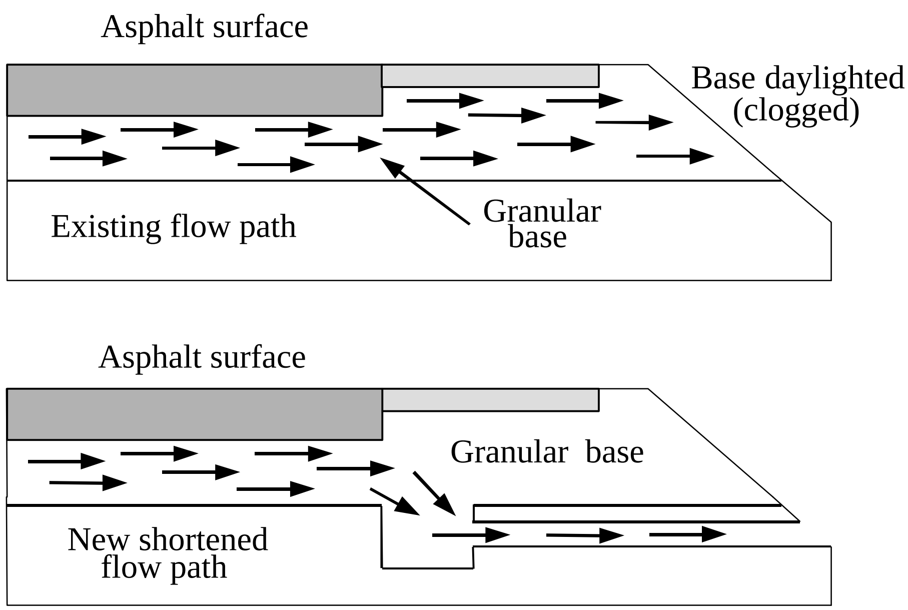


Figure 4-11.3. Longitudinal drain added to shorten flow path.

Geocomposite edge drains are approximately 25 mm 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 installations). In many previous projects, the trenched soil was used rather than a granular backfill/filter material. As noted elsewhere in this module, however, there have been problems with this design. An evaluation of the field performance of geocomposite edge drains found excessive soil loss to be the most common problem.(17) These problems are believed to be adequately addressed through modifications to the backfill material and placement of the geocomposite edge drain.

Many different types of proprietary geocomposite edge drains have been installed in numerous experimental installations and appear to be functioning well. However, long-term field performance is still under evaluation. A survey of State highway agencies found that only about one-half of the States consider retrofitted drainage as part of rehabilitation.(24) Recently, a number of States (including Indiana, Michigan, Wisconsin, and Illinois) have discontinued the use of geocomposite edge drains, as they have not been effective in extending the service life of the pavement.

Geocomposite edge drains should be used with caution when the base has an excess of 15 percent fines (material smaller than the 0.074 mm sieve). The core tends to become plugged under these conditions, and the design of the geocomposite drains is such that they cannot be cleaned.(6) The flow capacity of geocomposite edge drains is less than pipe drains and should be checked to ensure acceptability.(25) Means of reducing the infiltration of moisture, such as good joint sealing practices, may need to be employed to offset such limitations.(26)

Longitudinal Pipe Sizing, Grade, and Outlet Spacing

Pipe diameters that have been used in typical installations range from 40 to 200 mm, with 100 mm most common. The larger sizes are commonly preferred because of their ability to be easily cleaned and maintained. However, California uses a 75-mm pipe and reports no difficulty cleaning it out. The size selection, of course, must be included in the design analysis, taking into consideration the expected rate of flow, grade, and outlet spacing.

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.(21) However, this solution can be impractical. For instance, using a 1 percent grade over a flat section of 200 m, the edge drain will have to be 2 m deep on the low side. A more practical solution is to use smooth pipe and decrease the outlet spacing where flat grades exists.

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 75 to 90 m in order to permit cleaning. Figure A-1.1 can also be used to select an appropriate combination of pipe size and outlet spacing.

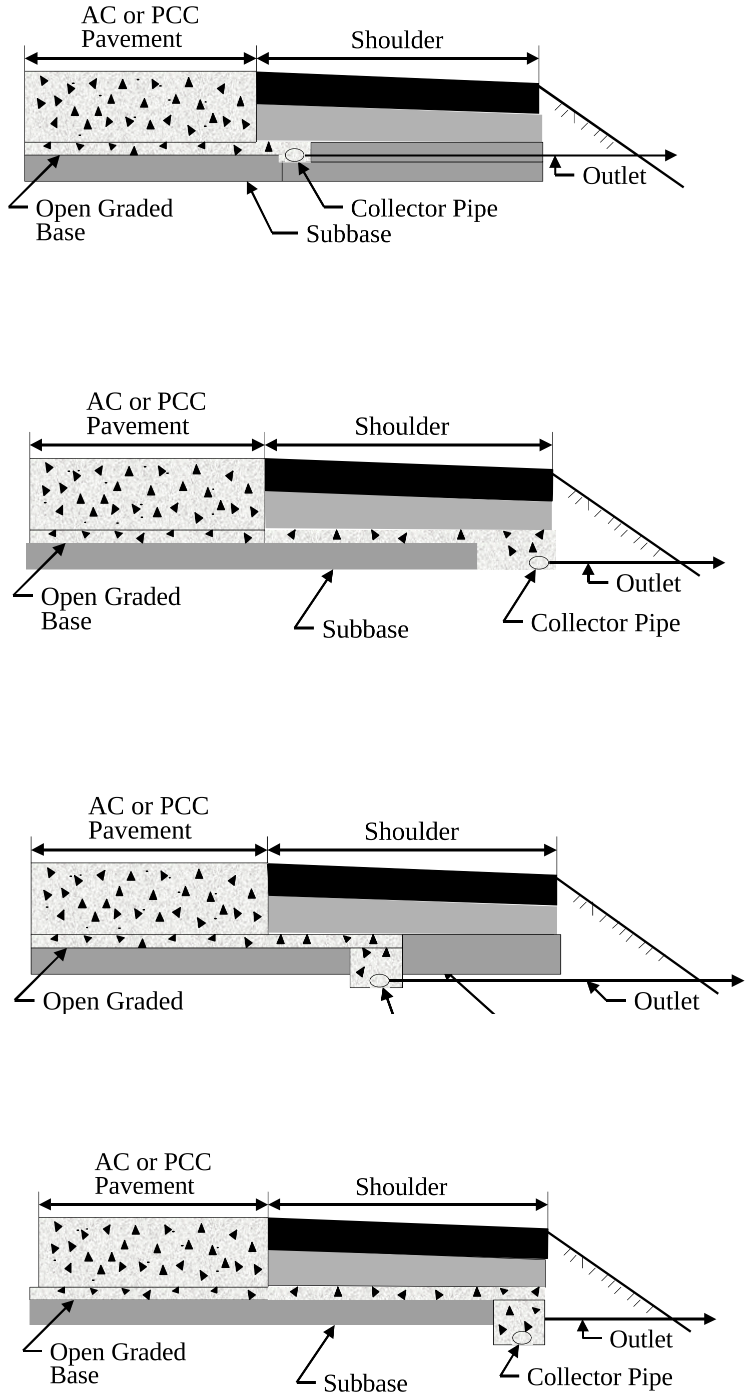
Depth of Collectors

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 below the subbase/subgrade interface. If significant frost penetration is expected, deeper trenches are necessary, as shown in figure 4-11.4.

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 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 above the 10-year expected water level in the storm drain system.

Trench Width

The required width of trench is a function of construction requirements, drainage requirements, and the permeability of the trench material. The trench width and material permeability must be such that



b) With Ground Water and/or Frost Penetration

Figure 4-11.4. Selection of longitudinal collector drains based on presence of ground water and possibility of frost penetration.

their product is equal to or greater than the design drainage rate. Depending on pipe size, many agencies use a trench width of 200 to 250 mm to allow proper placement of the pipe and compaction of the backfill material around the pipe.

Orientation of Hole/Slot

Collector pipes are normally placed with the inlet hole or slots facing down in order to reduce the possibility of sedimentation in the pipe and to reduce the potential for a static level of water in the trench.[19] However, in extremely wet or muddy conditions, where maintaining the trench and bedding materials in free-draining condition is difficult, it may be desirable to place the pipes with the holes or slots up or oriented laterally, depending on the direction of flow toward the collector pipe.[21]

Pipe Outlets

Free-draining pipe outlets are required to ensure that the system drains properly. Experience with daylighted aggregate bases has shown that the long-term drainage capacity of such systems is severely limited by plugging of the daylighted base at the outlet end due to silting and vegetation growth. When this occurs, the system originally designed for drainage becomes a water storage system.

Metal or rigid solid-walled pipe must be used for the lateral outlet pipe to ensure the proper grade and to reduce the potential for damage.[2] The outlet end should be placed at least 150 mm above the 10-year ditch flow line and protected with a headwall and splash block that is blended into the slope. Figure 4-11.5 illustrates the recommended outlet pipe design.[2]

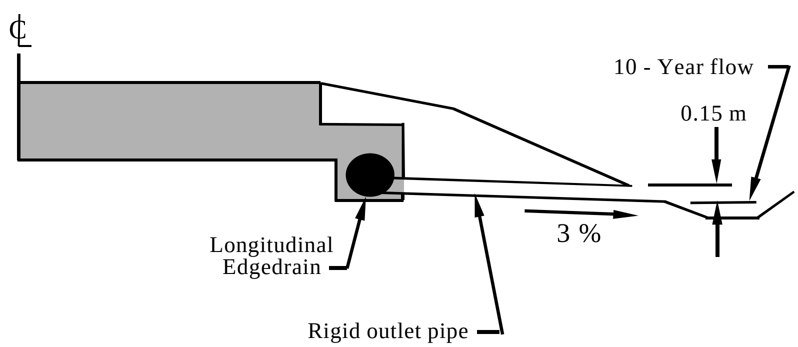


Figure 4-11.5. Outlet pipe design.[2]

Headwalls are recommended because they protect the outlet pipe from damage, prevent slope erosion, and facilitate the location of outlet pipes.[2] 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 4-11.6.

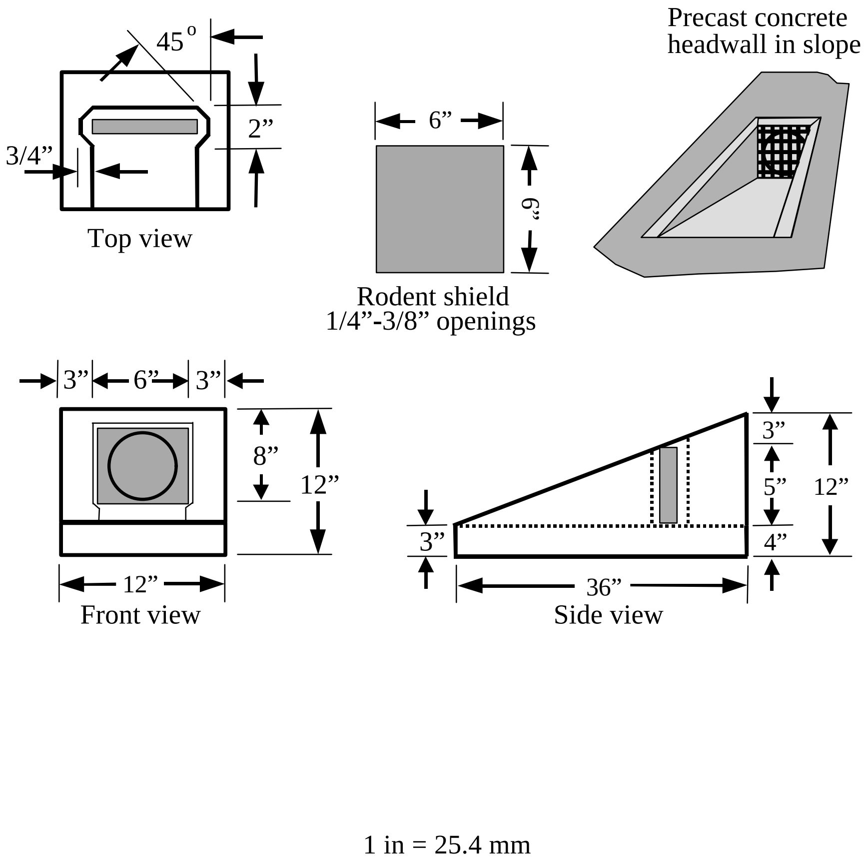


Figure 4-11.6. Precast headwall with rodent screen.[2]

To provide access for cleaning and flushing the pipe, the outlet end should be connected with the collector pipe through an elbow with a minimum radius of 300 to 450 mm. A dual outlet system is recommended to allow video inspection and maintenance from either end. The recommended design for an outlet system is shown in figure 4-11.7.

Backfill Material

The backfill/filler material placed in the trench around the pipe or alongside the geocomposite serves several functions, as noted:

- 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.

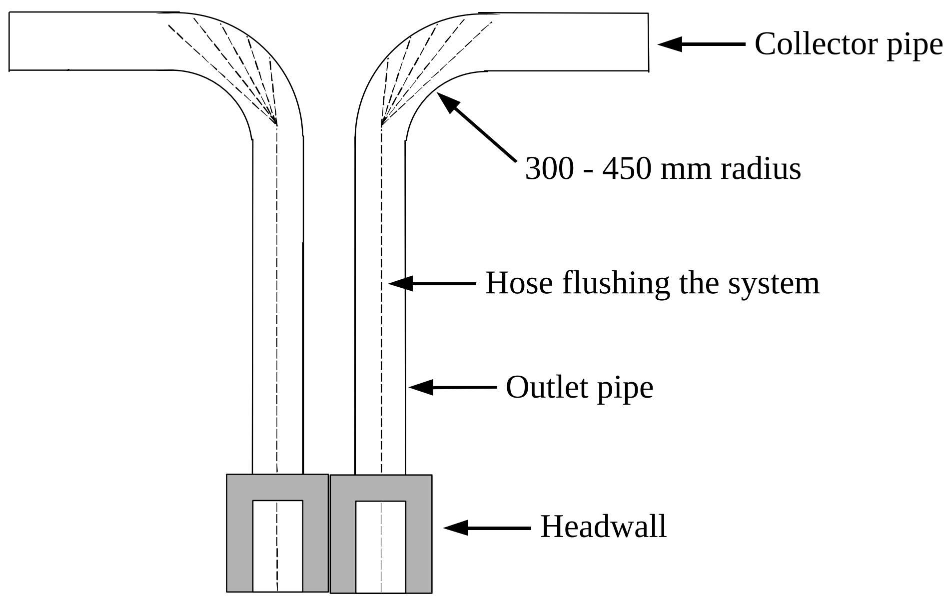


Figure 4-11.7. Outlet detail for system cleaning and video camera inspection.

There are specific procedures available to design the backfill/filler material to ensure that the drainage feature, be it a pipe or geocomposite, does not become clogged with fines. These are presented in greater detail in appendix A-1. With geocomposite edge drain installations in particular, excavated trench soil has been used as a backfill material. While this keeps the cost of the installation low, there have been problems associated with this approach. These include difficulties compacting the backfill, damage caused to the drain during the compaction process, and clogging of the filter material around the drain.[16,27]

Fleckenstein and Allen[16] recommend two steps to improve the performance of geocomposite edge drains. One modification is to move the geocomposite edge drain to the shoulder side of the trench. This helps to reduce the chance that fines from the base can clog the geocomposite's filter. The other change is to flush a sand slurry into the trench (about 12.5 liters of water per linear meter of fill) between the pavement and the geocomposite edge drain. The sand slurry acts as a filter material between the base layer and the edge drain and compacts much better than excavated soil. The recommended method for placing retrofitted geocomposite edge drains is illustrated in figure 4-11.8.

Other Considerations

As with any design or rehabilitation project, a critical evaluation of the design with respect to expected long-term performance, maintainability, and cost will help in the decision of which type of subdrainage should be constructed.

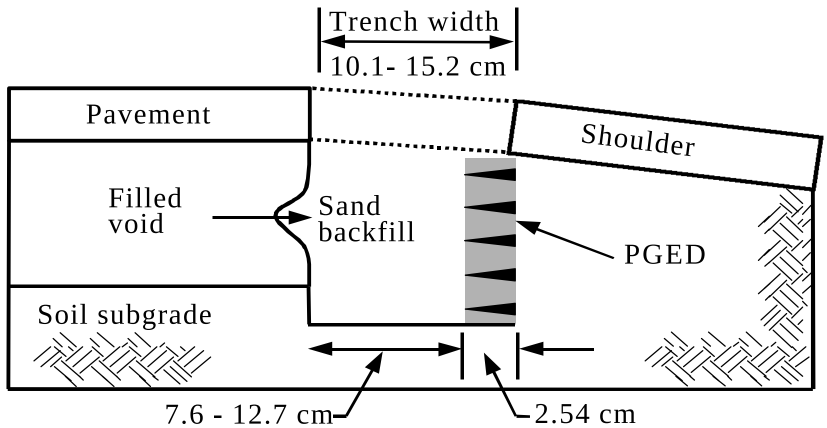


Figure 4-11.8. Recommended method of placing retrofitted geocomposite edge drains.[17]

If a rehabilitation project includes pavement widening, the design steps should also include material for the base layer. To ensure that this layer contributes to the drainage and does not entrap water or retard its removal, the widened base course must have a permeability that exceeds that of the existing base. Also, the depths of the various widening layers must be selected so that no entrapment is built into the pavement (this entrapment often occurs in a cross-section known as a bathtub section, in which the base and subbase layers are surrounded by impermeable layers).

In addition to considering the effectiveness of the retrofitted longitudinal edge drains, another problem that has to be considered is the possible disturbance of the pavement edge support when the edge drains are installed along the edge of the existing pavement. California and Florida have used stabilized backfill material to provide edge support.[6]

Another concern in the design of the drainage system is the placement of the geotextile. As voids develop at the slab/base interface, free water under pressure from heavy moving loads will erode fines in the base material. Regardless of the geotextile placement, fines will be eroded from the base. The geotextile only controls what happens to the fines after erosion. For a retrofitted edge drain against an impermeable base, it is recommended that the trench only be partially wrapped with the geotextile. By eliminating the geotextile at the slab/base interface, free water entering at the pavement shoulder joint and water flowing at the slab/base interface will be drained. This will drastically reduce the time water is available to saturate the base.

8. PAVEMENT SURVEYS

A drainage survey and evaluation are conducted to determine the need for improved drainage and to evaluate the potential for improving drainage. Included in the survey are assembling the available data about the existing highway and subsurface geometry, soil and material properties, side ditches, precipitation and frost penetration, and other factors that might contribute to the quantity and effect of water in the pavement.

The existing cross-section is also examined to determine the best design and location for installing a drainage system. This examination must recognize that the design objective is to remove the water as rapidly as possible and that this is accomplished by shortening the drainage path in a rehabilitation project. Features to examine in the field include the pavement cross-slopes and the presence of pavement distresses that are due to moisture.

9. COST CONSIDERATIONS

The costs associated with the installation of retrofitted subdrainage on existing pavements depends on the type of drain being installed. The costs for the installation of a longitudinal pipe edge drain are typically between $9.80 and $13.10 per linear meter, while the costs for the installation of geocomposite fin drains typically ranges from $6.60 to $8.20 per linear meter. Koerner[17] reports that the installed cost of geocomposite edge drains ranges from $3.30 to $6.60 per linear meter less than any other drainage system.

Cost comparisons for drained and non-drained pavements constructed in California are shown in table 4-11.2.[28] Subdrainage retrofitted in pavement rehabilitation projects can be expected to have a similar impact on pavement life only when the project evaluation clearly indicates that a moisture problem exists, and retrofitted subdrainage is engineered and constructed properly.

Table 4-11.2. Cost comparisons for drained and undrained pavements.[28]

	Thickness, m	Cost in place, $/m^3
UNDRAINED SECTION		
PCC Slab	0.26	50
Lean PCC Base	0.15	39
Aggregate Subbase	0.21	8.5
DRAINED SECTION		
PCC Slab	0.26	50
Asphalt-treated permeable base	0.15	32
Aggregate Subbase	0.21	8.5
Edge drains	$2.30/lin m (pipe only)	
Edge drain outlet pipe	$8.20/lin m	

10. CONSTRUCTION CONSIDERATIONS

Longitudinal edge drains are commonly installed after undersealing and grinding operations because of the potential for these operations to contaminate the filter fabric and backfill materials. This section provides details on the main construction phases involved in longitudinal edge drain installation.

The first step is to construct a trench of the proper width, depth, and grade. As stated earlier, 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. This allows better bonding between the slab and the cap that will cover the trench. The inspector should check the trench width, depth, and grade frequently.

The next step involves placing the drainage medium (pipe, fabric, geocomposite, and so on) and backfill. There are equipment trains that place the filter cloth, pipe, and backfill in one continuous operation. If a geocomposite fin drain or fabric-wrapped polyvinyl chloride (PVC) pipe is being used, the step involving placement of the filter cloth is not necessary. Where used, the filter fabric should be placed smooth and free of obstructions. Many agencies require that the filter fabric be lapped and secured by using a sewn or heat-bonded splice.

After the filter cloth is placed, a thin lift of aggregate backfill is placed to support the pipe, the pipe is carefully placed, and another lift of backfill material is placed. More than one lift may be needed to satisfactorily compact the backfill. Care must be exercised so that the pipe line and grade are undisturbed during the placement and compaction of the backfill. A minimum density of 90 percent of the standard laboratory density (AASHTO T 99) for the backfill material is recommended. When placing an aggregate backfill, a concrete vibrator that can be positioned at the bottom of the aggregate hopper appears to be more effective than surface or fan vibrators in helping to obtain the desired density.

Modern equipment used to install geocomposite edge drains often performs the trenching, backfilling, and backfill compaction in one step. This trench is usually narrow, 100 to 150 mm wide, and may be backfilled with the expelled shoulder material (see, however, the earlier discussion about the problems associated with its use).

The next step involves capping the trench, using either hot-mix asphalt (HMA) or PCC. A capping material similar to the shoulder material will provide a tighter joint that is easier to maintain. When PCC is used, plastic sheeting should be placed ahead of the hopper to separate the concrete and backfill material. A capping material equivalent in density or strength to the shoulder material should be specified.

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 very important to avoid high or low spots in the outlet trench, and make sure that the exposed end is not turned upward or otherwise elevated. Normal concrete placement and finishing techniques are used for cast-in-place headwalls. 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.

11. MAINTENANCE OF SUBDRAINAGE FEATURES

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. Vegetation growth, roadside debris, and discharged fines can clog outlet pipes and render the subdrainage system ineffective. A video inspection program of subsurface drainage systems found many systems to be clogged due inadequate maintenance.[13]

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 915 mm 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. Mowing around drainage outlets and inspection of the drainage outlets should be performed at least twice a year. This is best performed as a task independent of maintenance mowing.

Even when all design parameters are properly evaluated and included in the design, the performance of retrofitted subdrainage 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.

12. SUMMARY

Moisture is a major cause of rigid pavement distress and deterioration, and pavement subdrainage systems can be an effective means of removing water that infiltrates a pavement. However, depending upon the condition of the pavement and the sources of the excess moisture, subdrainage may not be an appropriate rehabilitation measure. It is extremely important that subdrainage be installed only on suitable pavements and that the system be carefully designed. The subdrainage system design must include:

- Recognition of moisture sources.
- Selection of the subdrainage design best suited to the source of moisture.
- Design of the drainage system components, including collector pipes, outlet pipes, outlet spacing, and outlet system. Provision of suitable filter materials must also be considered.

Two factors that are often cited as affecting the effectiveness of retrofitted subdrainage include the timing of the project (in relation to the progression of moisture-related distresses) and the amount of fine materials in the base. Also, after installation, the subdrainage system must be periodically maintained to ensure that it is effectively removing excess water from the pavement structure.

13. REFERENCES

1. Gendell, D. S. 1990, "Remarks on Pavement Drainage," Virginia Pavement Drainage Workshop, Williamsburg, VA.

2. Federal Highway Administration (FHWA). 1992, "Drainable Pavement Systems—Participant Notebook," Demonstration Project 87, FHWA-SA-92-008, Federal Highway Administration, Washington, DC.

3. Jeffcoat, H. H., F. A. Kilpatrick, J. B. Atkins, and J. L. Pearman. 1992, "Effectiveness of Highway Edgedrains," 92-4147, Federal Highway Administration, Washington, DC.

4. ERES Consultants, Inc. (ERES). 1998, "Pavement Subsurface Drainage Design," Training Course Under Development for the National Highway Institute, Washington, DC.

5. Carpenter, S. H., M. I. Darter, and B. J. Dempsey. 1979, “Evaluation of Pavement Systems for Moisture-Accelerated Distress,” Transportation Research Record 775, Transportation Research Board, Washington, DC.

6. Federal Highway Administration (FHWA). 1990, “Technical Guide Paper on Subsurface Pavement Drainage,” Technical Paper 90-01, Federal Highway Administration, Washington, DC.

7. Wells, G. K. and S. M. Wiley. 1987, “The Effectiveness of Portland Cement Concrete Pavement Rehabilitation Techniques,” FHWA/CA/TL-87/10, California Department of Transportation, Sacramento, CA.

8. Young, B. 1990, “Evaluation of the Performance of Fin Drains in Georgia,” FHWA-GA-90-8709, Georgia Department of Transportation, Atlanta, GA.

9. Virginia Department of Transportation (VADOT). 1990, “Guidelines for Providing Improved Drainage Systems for VDOT Pavement Structures,” Virginia Department of Transportation, Charlottesville, VA.

10. Gulden, W. 1983, “Experience in Georgia with Drainage of Jointed Concrete Pavements,” International Seminar on Drainage and Erodability at the Concrete Slab-Subbase-Shoulder Interface, Permanent International Association of Road Congresses, Paris, France.

11. Wells, G. K. and W. A. Nokes. 1993, “Performance Evaluation of Retrofit Edge Drain Projects,” Transportation Research Record 1425, Transportation Research Board, Washington, DC.

12. Permanent International Association of Road Congresses (PIARC). 1986, “Combating Concrete Pavement Slab Pumping by Interface Drainage and Use of Low-Erodability Materials: State of the Art and Recommendations,” Permanent International Association of Road Congresses, Paris, France.

13. Brent Rauhut Engineering, Inc. (BRE). 1997, “Video Inspection of Highway Edge Drain Systems,” Federal Highway Administration, Washington, DC.

14. Baumgardner, R. H. and D. M. Mathis. 1989, “Concrete Pavement Drainage Rehabilitation, State of the Practice Report,” Experimental Project No. 12, Federal Highway Administration, Demonstration Projects Division, Washington, DC.

15. Fleckenstein, L. J., and D. L. Allen. 1991, “Evaluation and Performance of Geocomposite Fin Drains in Kentucky,” Paper Presented at the 70th Annual Meeting of the Transportation Research Board, Transportation Research Board, Washington, DC.

16. Fleckenstein, L. J., and D. L. Allen. 1993, “Field and Laboratory Comparison of Pavement Edge Drains in Kentucky,” Transportation Research Record 1425, Transportation Research Board, Washington, DC.

17. Koerner, R. M., G. R. Koerner, A. K. Fahim, and R. F. Wilson-Fahmy. 1994, “Long-Term Performance of Geosynthetics in Drainage Applications,” NCHRP Report 367, Transportation Research Board, Washington, DC.

18. DuBose, J. B. 1995, “An Evaluation of IDOT’s Current Underdrain Systems,” IL PRR-120, Illinois Department of Transportation, Springfield, IL.

19. Cedergren, H. R. 1974, “Drainage of Highway and Airfield Pavements,” John Wiley and Sons, New York, NY.

20. Steffes, R. F., V. J. Marks, and K. L. Dirks. 1991, “Video Evaluation of Highway Drainage Systems,” Transportation Research Record 1329, Transportation Research Board, Washington, DC.

21. Moulton, L. K. 1980, “Highway Subdrainage Design,” FHWA-TS-80-224, Federal Highway Administration, Washington, DC.

22. Carpenter, S. H., M. I. Darter, and B. J. Dempsey. 1981, “A Pavement Moisture-Accelerated Distress (MAD) Identification System, Volume I,” FHWA/RD-81/079, Federal Highway Administration, Washington, DC.

23. Cedergren, H. R., K. H. O’Brien, and J. A. Arman. 1973, “Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Pavement Structural Systems,” FHWA-RD-73-14, Federal Highway Administration, Washington, DC.

24. Wade, M. J., K. D. Smith, and H. T. Yu. 1997, “Summary of State Drainage Practices,” 76th Annual Meeting of the Transportation Research Board, Transportation Research Board, Washington, DC.

25. Holtz, R. D., B. R. Christopher, and R. R. Berg. 1995, “Geosynthetic Design and Construction Guidelines, Participant Notebook,” FHWA-HI-95-038, Federal Highway Administration, Washington, DC.

26. Christopher, B. R., and V. C. McGuffey. 1997, “Pavement Subsurface Drainage Systems,” NCHRP Synthesis of Highway Practice 239, Transportation Research Board, Washington, DC.

27. Ford, G. R., and B. E. Eliason, “Comparison of Compaction Methods in Narrow Subsurface Drainage Trenches,” Transportation Research Record 1425, Transportation Research Board. Washington, DC.

28. Forsyth, R. A., G. K. Wells, and J. H. Woodstrom. 1987, “The Road to Drained Pavements,” Civil Engineering, American Society of Civil Engineers, Reston, VA.

MODULE 4-12

RIGID PAVEMENT RECYCLING

1. INSTRUCTIONAL OBJECTIVES

This module describes recycling as a rehabilitation option for existing rigid pavements. In many cases where reconstruction of an existing rigid pavement is warranted, substantial cost savings can be achieved by recycling the portland cement concrete (PCC) for use as an aggregate source. Upon completion of this module, the participant shall be able to accomplish the following:

1. Identify conditions when rigid pavement recycling may be considered as part of a reconstruction project.
2. Identify potential benefits of concrete recycling and the potential uses of recycled concrete aggregate (RCA) in pavement reconstruction.
3. Describe the steps in the rigid pavement recycling process.
4. Discuss the properties of recycled concrete aggregate and PCC mixtures containing recycled concrete aggregate, and how those properties affect their potential applications.
5. Describe mix design and pavement structural design implications of using recycled concrete aggregate.

2. INTRODUCTION

At some point near the end of the life of an existing rigid pavement, the pavement is so badly deteriorated that total reconstruction becomes a more cost-effective alternative than resurfacing or restoration. Existing rigid pavement conditions that may indicate the need for reconstruction include:

1. Little or no remaining structural life, as evidenced by extensive slab cracking.
2. Extensive slab settlements, heaves, or cracking due to foundation movement (caused by swelling soil or frost heave).
3. Extensive joint deterioration (particularly for short-jointed pavements, since full-depth repair would require replacement of a large percentage of the concrete surface).
4. Extensive concrete deterioration due to poor durability (D-cracking or reactive aggregate distress over the length of the project).
5. Outdated geometric design standards (lane widths, bridge clearances, curve superelevations).

Typically, the outermost lane of a multilane highway carries the most truck traffic, and often reaches the end of its structural life much sooner than the inner lane. When this occurs, the option of reconstructing only the outer lane should be considered.[1,2] Although the initial per-lane cost of reconstruction is higher than that of resurfacing, it will provide a longer service life and a lower overall life-cycle cost (LCC) than resurfacing alone.

In most cases where pavement reconstruction is justified, the existing concrete can be recycled to reduce the cost of reconstruction. In rigid pavement recycling, the existing rigid pavement essentially becomes an aggregate source for use in some part of the reconstructed new pavement, or in some part of a new pavement constructed elsewhere. However, the recycled aggregate has physical and mechanical properties that are slightly different than virgin materials, and these properties must be recognized and considered in both the mix design and pavement design phases of the reconstruction project.

3. DEFINITION

Concrete pavement recycling involves breaking up the old pavement on grade, loading and hauling the material to a crushing plant, and processing it at the plant to produce RCA of a specified size. The product of this process is an aggregate that can be used in place of virgin aggregate in any component of the pavement structure. The recycled coarse aggregates are more useful than the recycled fine aggregates (those passing the 9.5 mm sieve), primarily because the angularity and high absorption capacity of recycled fines can adversely affect the workability of the resulting mix.

Over the years, several advancements in rigid pavement recycling technology have made it a more economically feasible rehabilitation alternative. These advancements include:[(3)]

- Development of equipment for effectively breaking and crushing the existing rigid pavement on grade.
- Development of steel removal methods that minimize hand labor.
- Use and application of crushing equipment that can accommodate steel reinforcement.

4. PURPOSE AND APPLICATIONS

Existing rigid pavements that have reached the end of their useful life are candidates for rigid pavement recycling. Rigid pavement recycling produces an aggregate for use in the reconstruction of the pavement. The primary reasons for considering rigid pavement recycling as part of a rigid pavement reconstruction project include the following:[(4)]

- Dwindling landfill space.
- Increased disposal costs.
- Conservation of materials.
- Scarcity of high-quality, virgin aggregates.
- Overall reduction in project costs.

Many of these factors become even more acute in urban reconstruction settings. For example, disposal of demolished PCC material in an urban area may require long hauls in heavy traffic to the nearest landfill accepting such waste. Furthermore, bringing in quality virgin aggregates for new construction projects in urban settings may require hauls of 80 to 100 km.[(3)]

RCA can be used as an aggregate source for any component of the pavement structure, including the following:[(5,6)]

- Untreated, dense-graded aggregate base.
- Cement- and asphalt-stabilized bases.
- Lean concrete base.
- Portland cement concrete surfacing.
- Asphalt concrete surfacing.
- Fill.
- Filter material.
- Drainage layer or edge drains.

Rigid pavement recycling can be performed on any existing rigid pavement type: jointed plain concrete pavements (JPCP), jointed reinforced concrete pavements (JRCP), and continuously reinforced concrete pavements (CRCP). While the productivity and effectiveness of many early rigid pavement recycling projects were hampered by the presence of reinforcing steel (deformed reinforcing bars in CRCP and mesh reinforcing in JRCP), the development of innovative equipment for use on site has virtually eliminated the presence of steel as a problem. For example, on CRCP, a backhoe or front-end loader armed with a rhino horn attachment (a 760-mm curved and pointed steel pick) is extremely effective in hooking steel and pulling it free from the concrete fragments, although some handwork to cut the steel is still required.(3) On JRCP, rigid pavement pulverizing equipment is available that effectively severs the mesh reinforcement, allowing it to be easily removed by a rhino horn.

Recycling of rigid pavements is not limited to those pavements that contain sound aggregate. Wyoming and some other western States have recycled pavements containing reactive aggregate into new concrete using flyash to control expansion of the reactive aggregate.(7,8) These projects are generally performing well, although a recent performance evaluation of one recycled project in Wyoming indicated the recurrence of alkali-silica products.(9)

D-cracked pavements have also been recycled into new rigid pavements. See references 10, 11, 12, 13, and 14. However, it is generally recommended that severely D-cracked aggregates not be recycled back into a PCC paving course. In using aggregate from recycled D-cracked pavement, many agencies have limited the recurrence of D-cracking in the new pavement by using flyash to increase durability and by limiting the maximum size of the recycled concrete aggregate (typically to a maximum top size of 19 mm).

5. LIMITATIONS AND EFFECTIVENESS

Rigid pavement recycling can be a cost-effective component of a pavement reconstruction project. By reusing the existing rigid pavement as an aggregate source for the construction of the new pavement, significant savings in aggregate materials, hauling, fuel, and disposal costs can be realized, particularly in an urban setting. Whether or not a particular project should be recycled depends on numerous factors, including suitability of the pavement for recycling, availability and cost of virgin aggregate, cost of disposing of old pavement material if it is not recycled, approximate cost of recycling, agency policy toward recycling, and the extent of local contractors' experience with recycling.

RCA does have slightly different physical properties than virgin aggregate, and these must be accounted for in the PCC mix design. For example, the angularity and higher absorption capacity of recycled fines result in a very harsh mix when they are used as a complete replacement for virgin fines. Also, concrete strengths of RCA mixtures are typically 5 to 10 percent lower than those of conventional concrete at the same water-cement ratios.(6) Finally, concrete containing RCA has higher shrinkage and greater thermal expansion properties, which has been cited as a contributing factor in the development of excessive mid-panel cracking on some pavements.(4)

Rigid pavements constructed with RCA also exhibit different mechanical properties than conventional rigid pavements. Of particular concern is the load transfer capability of transverse cracks that occur (as expected) on JRCP and CRCP. Because RCA is less resistant to abrasion (due to the increased quantity of soft paste that is present on the aggregate), the crack faces are more sensitive to abrasion under repeated traffic loads.(4,15) Furthermore, concrete containing RCA often fractures along the old paste-aggregate interface.(15) Together, these factors contribute to create very smooth crack faces with poor load transfer capacity that cause excessive vertical deflections under traffic loadings, resulting in failure of the reinforcing steel and deterioration of the crack.(4) Snyder and Raja(15) have postulated that the load transfer capabilities of transverse cracks are strongly related to the crack face texture, which is a function of the type, size, and number of coarse aggregate particles at the crack face and the mode of fracture.

Similar behavior can occur at transverse joints of pavements containing RCA if load transfer devices are not provided. For example, a recent field study on the performance of concrete pavements, containing RCA, indicated that all RCA pavements with nondoweled joints exhibited poor load transfer and significant levels of joint faulting.(9) However, the addition of virgin aggregate to the mixture can contribute to the performance of the pavement. For example, an existing CRCP was recycled in 1984 by the North Dakota Department of Transportation into a short-jointed JPCP.(16) The new pavement was constructed using 60 percent RCA and 40 percent virgin materials, and is showing good performance after 12 years of service.(16)

A rigid pavement that has an HMA overlay can be recycled, but it is generally recommended that the two layers be recycled separately. When asphalt concrete is used as an aggregate in a PCC mix, the asphalt cement inhibits entrainment of air in the concrete mix.(17) However, it is reported that Austria has recycled concrete with up to 10 percent asphalt without any apparent detrimental effects.(18) RCA can be used as an aggregate in an AC mix, although the asphalt demands will be greater due to the increased porosity of the aggregate.

6. THE RECYCLING PROCESS

This section presents an overview of the rigid pavement recycling process, from the breakup and demolition of the existing pavement to the production of RCA.

Demolition and Removal

Before demolition of the pavement, any AC patches should be removed from the existing pavement.(19) Consideration should also be given to the removal of joint sealant material,(3) although some agencies elect to leave existing joint sealants in place during demolition.

As mentioned previously, it is generally recommended that an HMA overlays be recycled separately from the rigid pavement. The most efficient means of removing the asphalt overlay is through cold milling, which is described in module 3-4.

After conducting these preliminary steps, the recycling process begins with the on-site demolition of the rigid pavement. The PCC material is broken into pieces small enough (about 460 mm to 600 mm on a side) to be lifted and loaded into trucks. Several types of PCC demolition equipment are available, including the following (see references 3, 6, 20, 21, and 22):

- Drop Ball. A heavy steel ball (1.8 to 6.3 metric tons) is hoisted by a crane and dropped on the pavement. The use of this equipment is not recommended because it breaks the concrete into excessively small pieces that are less salvageable.

- Gravity Drop Hammers. This equipment operates by lifting a mass mechanically or hydraulically and releasing the mass. The impact force of the falling mass is delivered to the pavement through an impact foot.

- Hydraulic or Pneumatic Hammer. This type of equipment has been used extensively for localized removal of pavement. These hammers are commonly mounted on backhoes and come in a variety of sizes, ranging from 610 J at 200 blows/min to 2.7 kJ at 600 blows/min.

- Trailer-Mounted Diesel Hammers. These are currently the most widely used equipment for pavement breaking on large reconstruction projects. The diesel pile-hammers deliver between 50 and 90 blows/min with the impact energy ranging from 24.4 to 40.7 kJ. The high impact forces delivered by the hammers thoroughly break the bond between the concrete and the reinforcing steel, making the steel removal easier and less costly; however, this equipment has the potential to damage underground utilities or culverts. The major advantage of diesel hammers is that the size of the rubble can be easily controlled by altering both the width of each pass and speed of travel. Production rates of 840 to 1,000 m^2/hr for 203 mm thick pavement have been achieved with this type of equipment, using a typical pass width of 457 mm.[21]

- Spring-Arm Whiphammers. A whiphammer uses a flexible arm made up of leaf springs that increase the velocity of the tool head and also insulate the machine from reverse shock. The production rates vary widely, depending on existing conditions; the range is from 200 to 400 m^2/hr for 229 to 254 mm thick JRCP.

- Vibrating-Beam Breaker. Also called a resonant breaker, this equipment uses power transmitted through a 163- by 450-mm by 3.7-m forged steel beam to break up concrete. The resonant frequency of the beam is used to deliver a high-frequency, low-amplitude impact force to a 305-mm square breaker plate mounted at the end of the beam. This equipment does not disturb underground utilities, produces smaller-sized rubble (90 percent of the rubble is 200 mm or less in diameter), and is relatively quiet, all of which make it particularly suited to use in urban areas. Production rates of up to 670 m^2/hr for 225-mm thick pavement have been achieved.

- High-Pressure Water Jets. This equipment is used to selectively remove deteriorated concrete without damaging other areas on bridge decks. Water pressures of 69 to 103 MPa are used. The major advantage of this system is the selective removal of concrete, but it has seen limited application on pavements.

As previously described, a backhoe or front-end loader equipped with a rhino horn attachment can be an effective means of handling reinforcement in the concrete pavement. Where continuous steel reinforcing is present, the rhino horn can hook and pull the steel free from the concrete rubble, although some hand work (workers with torches or hydraulic shears) may still be required to cut the reinforcing to manageable sizes.[3] Where wire mesh is encountered, the demolition operation generally is enough to sever the mesh such that the rhino horn can then easily lift and pile small concrete pieces containing the wire mesh.[3] Any pieces too large for the primary crusher can also be broken by the rhino horn.

Dowel bars and tie bars are generally not removed on site but rather are removed during the crushing operation. However, during the demolition of the concrete many of these pieces may become loose and pop out.

The pavement rubble is loaded by front-end loaders into dump trucks and hauled to a crushing plant for processing into recycled concrete aggregate. Not all of the concrete is recovered in the demolition and removal operation. Contractors typically will not make an effort to pick up pieces smaller than 152 mm to reduce the amount of dirt or other contaminants picked up with the concrete rubble. Guide specifications for removing and crushing old rigid pavement are provided in references by Yrjanson[6] and ACPA.[3]

At least one highway agency is using an on-grade "recycling train" to recycle deteriorated rigid pavements. Since 1993, the Iowa Department of Transportation has used a recycling train that breaks up the rigid pavement and loads the PCC rubble in a rock crusher on site, which then crushes the material into two sizes, depositing the larger size (nominal size 19 mm) back on grade and the smaller size on the shoulder. The larger size RCA is then used as a granular base course for the reconstructed pavement.

Aggregate Processing

The pavement rubble is processed at the crushing plant to produce recycled concrete aggregate. A nearby crushing plant may be used, or a "portable" plant may be set up at a location close to the project site. In urban areas, a recycling plant can even be set up in the center of one quarter of a cloverleaf interchange on the project. A conventional crushing plant, such as those used at quarries, can be adapted for concrete recycling by adding an electromagnet for removing steel. A schematic of the sequence of operations at a concrete pavement recycling plant is shown in figure 4-12.1. The aggregate processing operation consists of crushing, removing steel, and sizing.

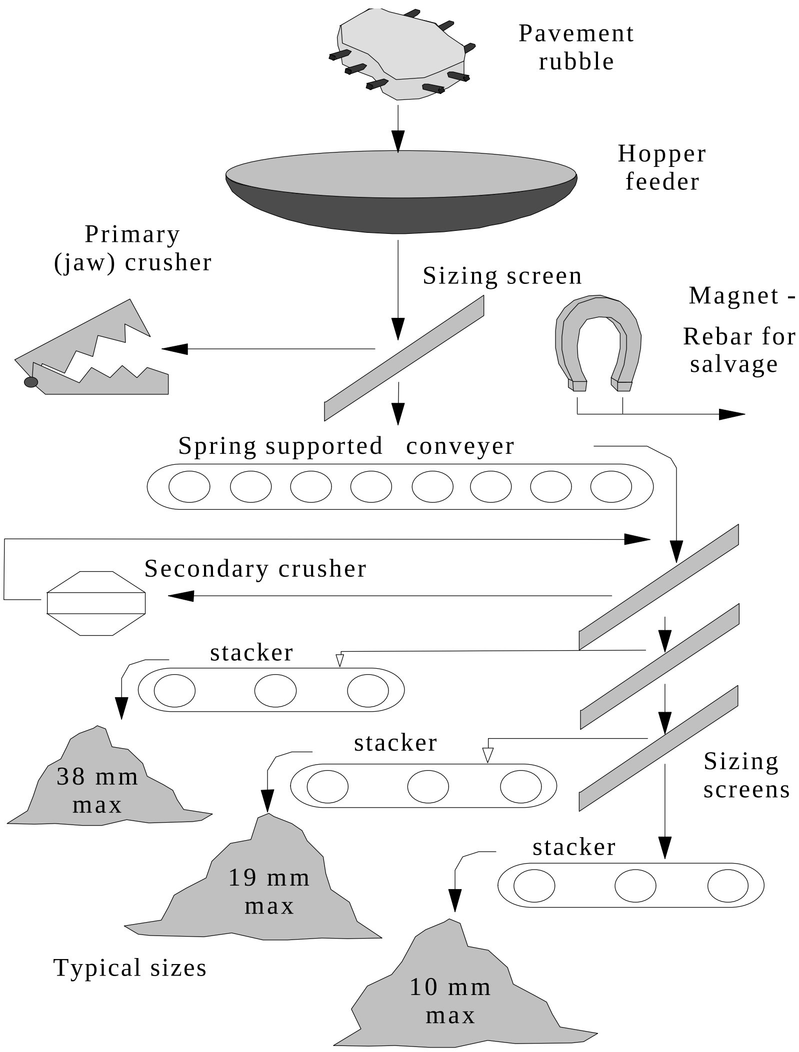


Figure 4-12.1. Schematic illustration of the sequence of operations at a concrete recycling plant.

Crushing

A front-end loader feeds the concrete rubble into a hopper, which regulates the flow of the rubble onto the sizing screen. Pieces larger than 102 mm are sent to the primary crusher, which breaks the concrete away from the reinforcing steel and into small pieces (less than 152 mm). This process is very effective in separating steel from concrete for later removal by an electromagnet.

A heavy-duty jaw crusher, which uses a cyclic compression force to fracture the concrete, should be used for primary crushing operations. Impact-type crushers tend to produce excessive amounts of fines.[6] Schematic diagrams of the main types of crushers are shown in figure 4-12.2.[3]

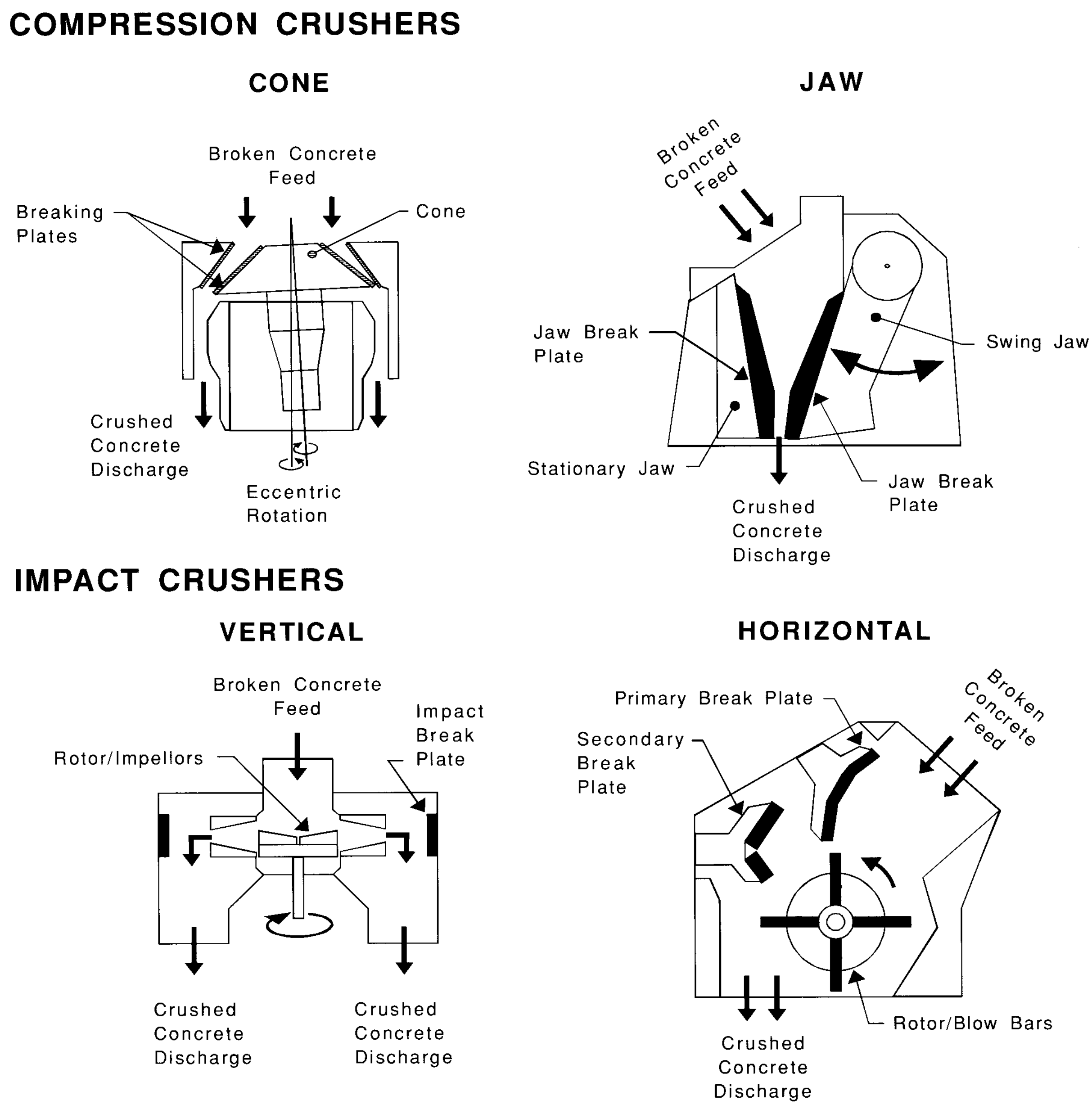


Figure 4-12.2. Types of crushers used for concrete recycling.[3]

The material processed by the primary crusher is discharged onto the conveyor belt and sent to a screening deck. Occasionally, long pieces of steel may get stuck between the discharge opening and the conveyor, slowing the flow of material out of the crusher and even punching holes in the conveyor belt. To alleviate this problem, the conveyor should be located 0.6 to 0.9 m below the discharge opening. The conveyor may also be supported on springs.

Removing Steel

One of the most significant advancements in aggregate processing is the development of economical procedures for removing steel. Steel removal had been a major roadblock to concrete recycling in the past. Now, an electromagnet placed over the conveyor belt removes virtually all steel, leaving only a minor amount of hand work to ensure that no steel enters the sizing screens. Recovered steel can be sold for about $44 per metric ton.

Sizing

The crushed concrete is passed through a set of sizing screens to separate it into different top size material. Pieces larger than the desired top size are recirculated through a secondary cone crusher until all material passes through the sizing screens. Commonly two sizes of crushed aggregate are obtained, depending on the anticipated use for the material:

- Granular base
  Size 1: < 76 to > 38 mm
  Size 2: < 38 mm

- Concrete pavement
  Size 1: < 38 to > 9 mm
  Size 2: < 9 to > 4.75 mm

- Asphalt concrete
  Size 1: < 25 to > 4.75 mm
  Size 2: < 4.75 mm

When recycling rigid pavements, the grading bands should be adjusted to maximize the recovery of coarse aggregate.[6] The recycled fine aggregate (< 4.75 mm material) is normally not used in a concrete mixture because of its angular shape and high absorption (about 7 percent). The angular shape makes the mixture harsh and difficult to finish, and high absorption makes the control of water content very difficult.

Yield

The coarse aggregate produced through recycling is more useful and attempts are generally made to maximize the amount of coarse aggregate recovered. More coarse aggregate can be recovered by crushing to a larger top size. With a 19-mm top size, a recovery of 55 to 60 percent is typical, while with a 38-mm top size, up to 80 percent or more can be recovered.

Contaminants

Various contaminants can be found in concrete pavement rubble. Table 4-12.1 lists the various contaminants and the potential problems that they pose to PCC mixtures containing RCA. Note that if the RCA is to be used in an application other than a PCC surface mixtures, the effects of any of the contaminants are not considered harmful.

Table 4-12.1. Contaminants in concrete pavement rubble.

Contaminant	How Removed	Effect of Contaminant on Concrete Mixtures with RCA
Reinforcing Steel	• On grade • Electromagnet during crushing operations	• No effect (steel effectively removed)
Dowel Bars/Baskets	• On grade • Electromagnet during crushing operations	• No effect (steel effectively removed)
Chemical Admixtures in Original PCC Mix	• Not removed	• Entrained air in original PCC dictates greater air contents in RCA concrete • Other effects not known
Deicing Salts	• Not removed	• Not known
Oil	• Not removed	• Small quantity believed to have no effect
Joint Sealant	• Removed prior to demolition by some; not removed by others	• Small quantity believed to have no effect
Soil/Base Course Materials	• Careful loading operator • Scalping screen ahead of primary crusher	• May introduce clay balls in mix, reducing PCC strength

Reinforcing steel and dowel assemblies are easily removed with magnets during the aggregate crushing operation. Excess soil and base material can be picked up during the loading activities, but experienced operators generally are able to prevent this; if a concern, an additional scalping screen can be placed ahead of the primary crusher to remove soil and base course materials.

The surfaces of pavements are usually contaminated with oil, but the amount is such a small percentage of the total volume of the rubble that oil is not a serious concern. The amount of joint sealant materials present in the total volume of concrete rubble is also very small, and might not justify the labor intensive task of hand removal (although some agencies do elect to remove it prior to recycling).

Chlorides and sulfates are the most undesirable chemical contaminants because of their potential for adversely affecting concrete durability: chlorides will accelerate corrosion in reinforcement, and sulfates will cause expansion damage.(23) However, studies conducted to date have shown that the levels of chlorides typically found in recycled concrete are not detrimental to performance, although excessive amounts of sodium chloride have been found in the fines of some study samples that were obtained within 38 mm of the original surface.(5,24) The average chloride content in the coarse fraction (> 4.75 mm) of recycled aggregate is about 1.2 kg/m^3, compared to an average chloride content of virgin aggregate of about 0.6 kg/m^3.[6,20,23] Typical limits for highly reinforced concrete are approximately 2.4 kg/m^3, meaning that most RCA is within acceptable limits.

It is suggested by some that contaminants are only a concern for RCA that will be used in a new PCC mixture.[3] That is, if the RCA is slated for use in a different application (as a base course, for example), the issue of the effect of contaminants is irrelevant. In any event, in order to ensure the suitability of a PCC mix incorporating RCA, trial mixes should be prepared and tested for workability, strength, and durability.

Although not really a contaminant, some concern has been expressed about leachates clogging the pipes and filter fabrics of drainage systems when RCA is used in a nonstabilized permeable base.[3] These leachates occur when dust adheres to the larger coarse aggregate particles during the crushing of the RCA, and then are washed away when water flows through the permeable base. The use of stabilized permeable bases is one means of preventing this problem.

7. MIX DESIGN CONSIDERATIONS

A significant amount of information has been obtained about the performance of RCA in PCC and AC mixes from both laboratory and field studies. Preparation and testing of trial mixes containing RCA is considered essential in order to ensure the adequacy of the mix.

Properties and Characteristics of RCA

Typical properties of RCA as compared to virgin materials are summarized in table 4-12.2.[4] This table shows that RCA do possess different properties than conventional virgin aggregates, and these properties must be considered in the development of an effective mix design.

Table 4-12.2. Comparison of typical virgin aggregate and RCA properties.[4]

Property	Virgin Aggregate	RCA
Particle Shape and Texture	Well rounded, smooth (gravels) to angular and rough (crushed stone)	Angular with rough surface
Absorption Capacity	0.8 – 3.7 percent	3.7 – 8.7 percent
Specific Gravity	2.4 – 2.9	2.1 – 2.4
L.A. Abrasion Test Mass Loss	15 – 30 percent	20 – 45 percent
Sodium Sulfate Soundness Test Mass Loss	7 – 21 percent	18 – 59 percent
Magnesium Sulfate Soundness Test Mass Loss	4 – 7 percent	1 – 9 percent
Chloride Content	0 – 1.2 kg/m^3	0.6 – 7.1 kg/m^3

<u>Particle Shape and Texture</u>

Both coarse and fine RCA particles are highly angular and have rough surfaces.[6] The irregular shape and texture of the recycled coarse particles are not known to have caused significant problems in the past, but the use of recycled fines (which are far more angular than typically used fines such as natural sands) greatly increases the harshness of the mix and can make it very difficult to work.[4] For this reason, many highway agencies do not use or severely limit the amount of recycled fines that can be used in a recycled PCC mix.[4]

Absorption Capacity

Absorption capacity is a measure of the amount of water that an aggregate can absorb. RCA generally have higher absorption capacities than virgin materials, primarily due to the porous nature of the cement paste fraction of the particles.[4] These higher absorption capacities can lead to a loss of workability, which can limit the time available for placing the concrete.[4] Laboratory work on RCA found that absorption capacity increases as particle size decreases, presumably due to the increased surface area of smaller particles.[25]

Specific Gravity

Specific gravity is a measure of the density of an aggregate relative to the density of water. RCA particles generally have lower specific gravity values than virgin materials, attributed to the RCA containing crushed mortar, which is less dense than most virgin aggregates because of its porosity and entrained air structure.[4] Specific gravity has been observed to decrease as particle size decreases.[25]

Abrasion and Soundness

Several tests are often conducted to assess an aggregate's abrasion resistance (L.A. abrasion test, ASTM C 131) and its soundness, or resistance to weathering (sodium and magnesium sulfate soundness tests, C 88). Typically, abrasion and soundness results for RCA are higher than for virgin aggregates, which is often attributed to the presence of the soft mortar fraction on the RCA particles.

Most RCA pass the Los Angeles abrasion test (falling below the upper limit of 50 percent abrasion loss), but it is not uncommon for RCA to fail the sodium sulfate test while passing the magnesium sulfate test; because of this, many agencies waive these tests for RCA.[4] At least one agency has attributed the failure of the RCA in the sodium sulfate test to an adverse reaction between the sodium sulfate and the mortar fraction of the RCA.[26]

Portland Cement Concrete Mix Design

Mix proportioning of recycled aggregate concrete is a critical item that requires careful consideration of the following key design variables:

- The amount of recycled fines to be permitted in the new concrete.
- The use of flyash.
- The amount and type of chemical admixtures (e.g., air-entraining agent, water reducer).
- The type of aggregate used in the original pavement.
- The maximum aggregate size.

Mix Design

RCA concrete mix design follows conventional procedures. However, the properties of recycled concrete mixes are difficult to predict. To obtain a mix with good workability, strength, and durability, mix proportions should be based on trial mixes. Guidelines on the development of competent PCC mix designs utilizing RCA are currently under development at the University of Minnesota.

In general, if only the recycled coarse aggregate is used in the new mix, no significant changes are necessary in the mix design and workability should be unaffected. However, if recycled fines (< 4.75 mm material) are used, the resulting mix can be quite harsh due to the angular nature of the recycled fines.[6] Recycled fines also have high absorption, with a consequent high demand for water which produces a more porous, weaker cement paste if not accompanied by a proportionate increase in cement.[4] However, increases in cement make the mix less economical.

To produce workable RCA mixes, it is necessary to replace all or most of the recycled fines with natural sand. The common practice is to use all recycled coarse aggregate and to replace all or most of the fine fraction with virgin fines (e.g., natural sand). If recycled fines are used, they are normally limited to 30 percent of the fine aggregate portion of the mixture.[6]

Workability

The restriction on the amount of recycled fines in the RCA mix is a significant factor influencing the workability of the resulting material. Other ways to improve the workability of the mix are to add a water reducer or to add flyash as a partial cement replacement. In fresh concrete, flyash acts as a plasticizer to improve workability without the need for additional water. Up to 15 percent of portland cement may be replaced with 20 percent of flyash. As discussed later, flyash is also often added to improve the durability of the RCA mix.

Gradation

Table 4-12.3 shows typical specifications for gradation of coarse aggregate obtained from recycled concrete. The coarse aggregate fraction of a typical concrete mixture utilizing RCA ranges from 50 to 60 percent. Note that in all three examples, the top size of the material is less than 25 mm. In order to improve aggregate interlock properties (particularly on reinforced pavements), it may be desirable to add a larger fraction of virgin coarse aggregate to the RCA.

Table 4-12.3. Typical specifications for coarse aggregate gradation.[6]

Sieve Size	**Percent Passing**		
	Wisconsin	**North Dakota**	**Oklahoma**
25.0 mm	100	100	100
19.0 mm	90 – 100	90 – 100	90 – 100
12.5 mm	–	–	25 – 60
9.5 mm	20 - 55	20 – 25	0 – 25
4.75 mm	0 – 10	0 – 10	0 – 10
2.36 mm	0 – 5	0 – 5	0 – 5
75 µm	–	0 – 1	–

Air Content

Air contents of fresh concrete containing RCA are higher and contain greater variation than the air contents of fresh concrete containing conventional aggregates because of the higher porosity of the RCA themselves and the entrained air in the original mortar.[4] Thus, total air contents for RCA mixtures must be higher than for conventional concrete mixtures.[4]

Alkali-Reactive Aggregate

Several western States have recycled concrete pavements containing alkali-reactive aggregate using pozzolanic admixtures (flyash) to control expansion of the recycled concrete aggregate in the new pavement. Pozzolans such as flyash are highly reactive and consume much of the excess alkali in the cement while the concrete is still plastic, thus reducing the amount of alkali available to react with the aggregate. Type F flyash is more effective than Type C flyash in controlling the alkali-reactive aggregate problem. Flyashes also positively affect the workability of the fresh concrete as well as the ultimate strength.

It is recommended that the flyashes used be tested by the procedures outlined in ASTM C 441. Producing concrete with alkali-reactive aggregate is no different with recycled concrete aggregate than with virgin aggregate. The recycled concrete aggregate tends to be less reactive; hence, recycling will have a beneficial effect.

The State of Wyoming has recycled several rigid pavements containing reactive aggregate.[8] On one project on I-80, the following modifications were made to the concrete mix design:[8]

- Use of low alkali type II cement in RCA mix.
- Blending of RCA with quality virgin aggregate.
- Use of Type F flyash.

A recent performance evaluation of this project indicated that the pavement was performing well after 9 years of service; however, laboratory testing of pavement samples suggested that the alkali-silica reactivity was beginning to redevelop.[9]

Mitigation of D-Cracking

Many highway agencies have recycled D-cracked pavements. The common practice in recycling D-cracked pavement is to reduce the top size of the coarse aggregate to 19 mm or less.[6] However, rigid pavement produced with the small top-size aggregate can have poor capacity to transfer load through aggregate interlock at cracks and joints. To avoid problems with faulting and spalling at these locations, only plain, short-jointed pavement should be constructed using this type of aggregate, and dowels should be provided at transverse joints for heavy traffic. Alternatively, the recycled aggregate can be supplemented with virgin aggregate that has a larger maximum size.

The use of flyash as a partial replacement for cement has been effective in reducing the D-cracking potential of RCA rigid pavements.[4] A study in Minnesota found that a 20 percent flyash replacement for a 15 percent cement reduction provided greatly reduced D-cracking potential.[10] The improved durability is attributed in part to the improved workability of the flyash concrete, which allows the use of less mix water and renders the mix less permeable.[4]

Hardened Concrete Properties

Several studies have been conducted comparing the properties of hardened RCA concrete to those containing virgin material. The following summarizes typical hardened concrete properties for RCA and virgin aggregate concrete:

1. One study indicated that the compressive strength of RCA concrete is between 60 and 100 percent of the compressive strength of conventional concrete at the same water-cement ratio.[27] Another study found that RCA concrete consistently exhibited strengths that are 10 percent lower than conventional concrete.[28] These strength reductions could be due to:[4]

    - Inherently weaker structure of the RCA (due to its cement paste–aggregate structure).
    - Greater porosity of RCA concrete due to the presence of the porous mortar component.
    - Greater number of bonded interfaces in RCA concrete, including interface areas between the natural aggregate and the mortar (both old and new), and also between the new and old mortars.
    - Lower resistance of RCA concrete to mechanical action.

    One way of achieving strength in RCA mixtures without compromising workability is to use water-reducing admixtures at the same cement content.

2. The static modulus of elasticity of recycled concrete is typically 20 to 40 percent lower than that of conventional concrete at the same water–cement ratio.[4] This reduction is attributed to the lower effective elastic modulus of the RCA (which consists of both aggregate and mortar fractions).

3. At the same water–cement ratio, the flexural strength of RCA concrete is generally about 8 percent less than that of conventional concrete.[4] Greater reductions in the flexural strength of RCA concrete are expected if recycled fines are used.

4. The freeze-thaw resistance of RCA concrete is often higher than that of the original concrete, although the results can vary depending on the quality of the original aggregate.[4] And, as previously mentioned, the durability of RCA concrete made with D-cracking susceptible aggregate can be substantially increased by limiting the top size of the aggregate.

5. RCA concrete typically exhibits higher levels of drying shrinkage than conventional concrete; increases from 14 to 95 percent have been documented.[4] The increase in drying shrinkage may be due to lower effective aggregate moduli, higher amounts of mortar components, and increased amounts of free water.[4] The highest levels of drying shrinkage were noted when recycled fines were included in the mix.

While most of this discussion has focused on RCA concrete mixes for use in the surface course, it applies to the use of RCA in lean concrete base courses as well. Recycled concrete aggregate was used in lean concrete bases before it was used in concrete surfaces.

Asphalt Concrete Mix Design

RCA can also be used in the development of AC mixes. However, due to the porosity and higher absorption capacity of RCA, it tends to absorb more asphalt cement than conventional aggregates. This must be considered in the development of the mix design and the economics of the resultant mix. For example, even though RCA may cost much less than virgin aggregate, asphalt cement is expensive enough that a small increase in the asphalt requirement for RCA mixtures may negate any economic advantages of using RCA in AC mixes.[29]

One major advantage of using RCA in AC mixtures is its effect on reducing the occurrence of stripping.[29] This is attributed to the presence of the portland cement mortar on the RCA, as portland cement is sometimes added to bituminous mixes to reduce stripping of the asphalt cement from the aggregate.

In a demonstration project on I-57 in Illinois, a badly D-cracked CRCP pavement was recycled and the aggregate was then used in the AC.[26] The new pavement consisted of five lifts of AC, ranging in thickness from 51 to 102 mm with a 38 mm thick surface lift. The recycled mixture was only used in the northbound direction, and then only in the five base lifts (i.e., not in the surface course). In the southbound direction, where virgin aggregate was used in the base mixture, the binder was 4.8 percent AC-20. This was increased to 5.4 percent in the recycled mixture to account for the increased air voids and absorptiveness of recycled concrete aggregate. A 4-year performance evaluation of this project indicated that the performance of the recycled asphalt concrete mix has been as good as, if not better than, the performance of the virgin aggregate mix.[26]

8. RIGID PAVEMENT DESIGN CONSIDERATIONS USING RCA

Conventional rigid pavement design procedures can be followed for rigid pavements utilizing RCA concrete. However, as described previously, the low abrasion resistance of RCA and its typically smaller top size (used to reduce durability problems) can create poor load transfer conditions at non-doweled joints or at transverse cracks, a condition that can lead to pumping, faulting, and steel rupture.

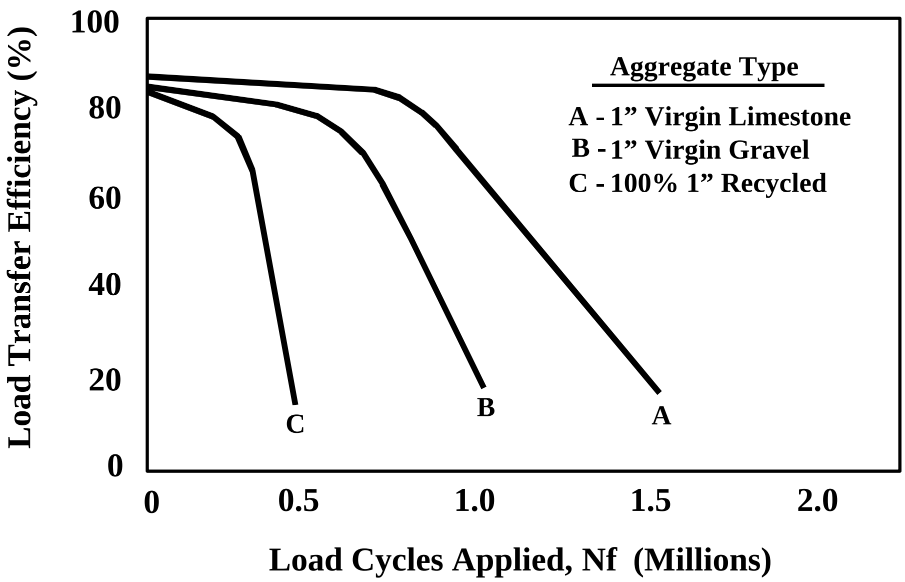
For example, the development and rapid deterioration of transverse cracks on several JRCP designs containing RCA has been documented.[30] Although intermediate slab cracking is expected to develop on JRCP, the inclusion of the reinforcing steel is intended to hold those cracks tightly together so that good aggregate interlock load transfer can be achieved. However, the load transfer across the cracks was compromised by the small top size RCA (19 mm) and the accompanying smooth crack faces.[30]

A laboratory research program on various material and structural design parameters was recently conducted.[4] Through a large testing apparatus, the passage of 40-kN wheel loads could be simulated across cracks or joints in a concrete slab. The slabs were instrumented to measure changes in the joint or crack width and relative vertical deflections across the joint or crack. Pertinent findings included:[4]

- PCC slabs containing recycled aggregate have a lower load transfer endurance than PCC slabs containing virgin materials (see figure 4-12.3).

- PCC slabs with greater foundation stiffness have an increased load transfer endurance than PCC slabs on a lower foundation stiffness.

- PCC slabs with higher reinforcement contents have greater load transfer endurance than PCC slabs with lower reinforcement contents. Deformed welded wire fabric performed significantly better than smooth wire mesh.



1 in = 25.4 mm

Figure 4-12.3. Effect of coarse aggregate type on load transfer endurance.[4]

These laboratory results and field performance observations suggest if JRCP or CRCP designs are being contemplated, consideration should be given to the use of higher reinforcing contents, greater foundation stiffness, and larger top size aggregate.[4] Consideration can also be given to the blending of large virgin aggregate with the RCA and the use of shorter joint spacing for JRCP (to reduce crack openings). Also, dowel bars should be included in all transverse joints (JPCP and JRCP).

9. CONSTRUCTION CONSIDERATIONS FOR RCA RIGID PAVEMENTS

No special techniques or paving equipment are needed for the construction of rigid pavements containing RCA. Conventional mix plants and paving equipment are used following normal paving and finishing practices.

With the recent development of two-lift concrete pavers, consideration can be given to the construction of a concrete pavement containing two layers: a thin PCC surface layer containing durable virgin aggregate and a thicker PCC underlayer containing RCA. The two layers are placed simultaneously in a “wet on wet” process, ensuring good bond between the materials. This procedure is used in Europe as a matter of economics and has reportedly performed well.[18,31]

10. COST BENEFITS OF RIGID PAVEMENT RECYCLING

The principal incentives for recycling rigid pavements are the increasing scarcity of high-quality virgin aggregates and the decreasing availability and increasing cost of disposal sites for concrete rubble. While there is an adequate total supply of aggregate in the United States, the distribution of sources is such that there are localized shortages. The cost of obtaining and transporting high-quality aggregate is very high in many regions of the country.

Recycling is not only useful in projects for which there is no nearby aggregate source. Even when acceptable virgin aggregate is readily available, it constitutes one of the greatest costs of highway construction: between 20 and 30 percent of the cost of materials and supplies, and between 10 and 15 percent of the total construction cost, excluding engineering and right-of-way acquisition.(32) When the labor and fuel costs of obtaining this aggregate from distant suppliers and the cost of the consequent project delay are added to this, recycling becomes an increasingly attractive alternative to conventional reconstruction.

Recycled concrete aggregate can be produced for about $7.70 to $11 per metric ton.(6) About half of the cost of recycling is attributable to hauling and crushing. The cost of virgin aggregate typically ranges from about $13.20 to $15.40 per metric ton. The actual savings by recycling is even greater, because the cost of disposing of old pavement material is eliminated.

Significant cost savings involving the use of recycled concrete have been reported on several projects. For example, Michigan reported about 50 to 65 percent savings on some of their projects.(33) Oklahoma reported savings of $700,000 on a $5.2-million project to reconstruct 12.5 km of I-40 east of Oklahoma City in 1983.(12) Wyoming reported savings of anywhere from $21,875 to $31,250/km on two-lane PCC roadways.(6) In North Dakota, the use of recycled pavement resulted in savings of $21,700 to 31,000 per km of concrete pavement.(6) Of course, the magnitude of actual savings will be variable, depending on local conditions.

11. NOTABLE RIGID PAVEMENT RECYCLING PROJECTS

Recycled concrete has been used extensively in Europe since the 1940s in building and pavement reconstruction. Now, recycled concrete is extensively being used in the United States for reconstructing pavements. Following are descriptions of some of the first rigid pavement recycling projects, which have contributed greatly to the current state of knowledge about the potential applications, limitations, and benefits of concrete recycling.

First Use of Recycled Concrete Aggregate in the United States

One of the earliest uses of recycled concrete in pavement construction was on U.S. Route 66 in Illinois. During World War II, construction of four-lane highways was not permitted. Illinois built a two-lane highway next to the existing narrow road and then demolished the old pavement.(34) The concrete was crushed and stockpiled for use as aggregate in the two additional lanes that were built after the war.

First Use of Recycled Concrete in an Econocrete Base

The first use of recycled concrete in an econocrete base for a new rigid was in California in 1974. The material was crushed to 38-mm maximum size and used in a mix that contained 140 kg/m^3 cement.

Cores taken from the econocrete base gave the average 28-day compressive strength as 5.1 MPa. The econocrete base was judged to perform exceptionally well under traffic.[29]

Recycling of Rigid Pavement with an HMA Overlay

In 1976, Iowa selected 2.4 km of U.S. Route 75 in Lyon County for an experimental project in PCC recycling.[17] The existing rigid pavement was built in 1934, widened in 1936 and 1958, and overlaid with 76 mm of AC in 1963. The objectives of the experiment were to determine if the PCC could be stripped of its HMA overlay, crushed, and used in a new concrete mix, and if a composite pavement could be constructed of two layers of concrete, using recycled concrete aggregate in different mix proportions, and placed monolithically using conventional slipform paving equipment. This experiment provided Iowa with a valuable experience in pavement recycling, and the success of this project encouraged Iowa to use concrete recycling on two projects in the following year.[6]

Four different mixes were produced, of which two utilized aggregate of crushed PCC only and two utilized crushed AC and PCC. The crushed PCC aggregate mixes contained 6.5 percent air with no air entraining agent added. The air content of the mixes containing crushed PCC and AC aggregate was only 3.5 percent, and was not significantly increased even by the addition of large quantities of an air entraining agent (this is consistent with laboratory studies, which have shown that asphalt inhibits the ability to entrain air in the concrete).

While it is not common practice to prepare new PCC with a combination of recycled PCC and AC, in Austria the AC portion of overlaid rigid pavements is routinely recycled with the PCC.[18] The AC typically comprises about 10 percent of the recycled mix.[18]

Recycling of an Urban Freeway

Beginning in 1978, concrete recycling was used in the reconstruction of the Edens Expressway (I-94) in the northern suburbs of Chicago, the first total reconstruction of a major urban freeway in the United States.[6,35,36] At that time, the total cost of $113.5 million for the project constituted the largest single highway contract ever awarded in the United States. In addition to being the largest highway project on which recycling had been used, the Edens reconstruction was the first which involved a mesh-reinforced pavement.

At that time, Illinois had not approved recycled concrete aggregate for use in PCC surfaces on Interstates, but it was permitted in base layers, and recycling of the existing pavement for use in the base was an option in the Edens contract. The Chicago area in general suffers no shortage of quality aggregate, but the recycling option was selected because of difficulties in hauling material through congested urban areas. The 29-km distance to the nearest aggregate stockpile was a 3-hour trip for large trucks in daytime urban traffic.

A crushing plant was set up in the cloverleaf of an interchange. The limited space available required that a careful balance be maintained between rubble coming into the plant and crushed concrete aggregate leaving the plant. Also, the area around the plant was heavily populated. To minimize noise, the plant operated in two 9-hour shifts, 7 days a week, and shut down every night at midnight for maintenance. A total of 318,000 metric tons of old pavement rubble was crushed on the site. About 85 percent of the crushed concrete aggregate produced was used in fill areas, and the other 15 percent was used as a 76-mm granular subbase on the stabilized subgrade. An asphalt-treated base was placed on the subbase, and a 254-mm CRCP was placed over this.

The Edens project demonstrated the feasibility of complete reconstruction of a heavily trafficked urban expressway using concrete recycling. Among the many savings achieved was an estimated 757,000 L of fuel. The 12 km reconstruction of Chicago's 10-lane Kennedy Expressway in the late 1980s was also accomplished with recycled material from the original pavement.

Recycling of D-Cracked Rigid Pavement

The reconstruction of a 26-km section of U.S. Route 59 in Minnesota in 1980 was the first major concrete recycling project in the United States to use D-cracked rigid pavement as a source of aggregate for a new concrete pavement.[6,10,37] The existing pavement was crushed to a maximum size of 19 mm. Several different trial mixes were developed, containing the following variations: cement and no flyash, 10 percent flyash substituted for cement, and 20 percent flyash substituted for 15 percent cement.[10] In laboratory tests, the concrete made with the 20 percent flyash mix was found to be the least susceptible to D-cracking. The aggregate in the mix consisted of 60 percent coarse aggregate from the crushed concrete and 40 percent natural sand. The mix contained 276 kg/m^3 cement and 64.8 kg/m^3 flyash. The crushed concrete fines were used in a 25 mm lift on the existing grade. The concrete slab was placed 203 mm thick and 7.3 m wide.

Recycling was estimated to have saved 572,000 L of fuel on the project, including 154,000 L of gasoline. The Minnesota DOT estimated that recycling resulted in a 27 percent savings in the total cost of the project.

A recent performance evaluation of this project indicated that significant faulting (average of 7.4 mm) had developed on this non-doweled pavement after about 1 million 80-kN ESAL applications.[9] This faulting was attributed to the lack of dowel bars at the transverse joints, although the reduced abrasion resistance of the RCA may have contributed partially to the development of the faulting.[9] However, the condition survey indicated no linear slab cracking and no signs of recurrent D-cracking.[9]

Recycling of Rigid Pavement Containing Reactive Aggregate

Wyoming was the first State to recycle rigid pavements suffering from alkali-silica reactivity. The Wyoming Highway Department successfully recycled several projects on I-80, in the Pine Bluffs area, that contained reactive aggregate.[8] The first reconstruction project in this area was in 1984, involving an 11-km stretch of I-80. The rehabilitation problem centered on countering the reactive aggregate mechanism in the existing pavement. The old pavement rubble was crushed to a 25-mm maximum size and only the coarse fraction (> 4.75-mm) was used in the new concrete mixture. Natural limestone aggregate was blended with the recycled aggregate to temper the reactive constituents of recycled material. A low-alkali, Type II cement, and a flyash meeting the requirements of C 618, were used to mitigate the reactive aggregate problem.

A performance evaluation of the Pine Bluffs project showed the pavement to be in good condition, although some joint faulting (average 2 mm) had begin to develop, suggesting the need for dowels on this pavement.[9] Small, localized areas of recurrent alkali-silica reaction products were scattered along the RCA portion of the project, and laboratory testing indicated the possible presence of moderate amounts of ASR products.[9]

12. SUMMARY

This module presents recycling as a rehabilitation technique for rigid pavements. Concrete recycling involves breaking up an existing concrete pavement to produce aggregate for use in a new pavement. In many cases in which reconstruction of a rigid pavement is warranted, recycling can be performed to significantly reduce the cost of reconstruction.

1. Reconstruction is sometimes a more cost-effective rehabilitation alternative for a concrete pavement than either resurfacing or restoration, particularly where the existing pavement is severely deteriorated.

2. In most cases where reconstruction is justified, the existing concrete can be recycled to reduce the cost of reconstruction. This is particularly true when high-quality virgin aggregate is expensive or not locally available, disposal sites for pavement rubble are difficult to find, or when reconstruction is on a tight schedule.

3. Rigid pavement recycling produces an aggregate for use in any component of the pavement structure (surface coarse, base, subbase) or in a related application (fill, filter layer, riprap).

4. Pavements containing reactive aggregate and D-cracked aggregate have been recycled into new pavements. Type F flyash is effective in controlling the expansion of the reactive aggregate. The common practice in recycling D-cracked pavement is to limit the maximum size of the recycled concrete aggregate (e.g., 19 mm).

5. The steps in the concrete recycling process are on-site breakup and removal, hauling to a crushing plant, crushing and steel removal, and sizing as appropriate for the intended use of the aggregate. Recent advancements in breaking and steel removal equipment have made rigid pavement recycling more economical.

6. Recycled concrete aggregate performs as well as virgin aggregate in PCC and lean concrete, as long as the mix design is modified to achieve desired strength and workability. Use of a water-reducing admixture and exclusion of crushed concrete fines may be necessary. Extensive testing of trial mixes is required to ensure that the desired results from the recycled mixture are obtained.

7. Recycled concrete aggregate performs well in AC and is less susceptible to stripping than virgin aggregate. However, the porosity of recycled concrete aggregate necessitates an increase in asphalt cement content, which may negate the cost benefit of recycling.

8. No specialized mixing or paving equipment are required for construction of rigid pavements containing RCA.

9. Innovative uses of concrete recycling on a number of projects have advanced the state of knowledge on this technique and demonstrated its versatility and cost-effectiveness.

13. REFERENCES

1. Calvert, G. 1983, “Portland Cement Concrete Inlay Work in Iowa,” Transportation Research Record 924, Transportation Research Board, Washington, DC.

2. Van Matre, F. R. and A. M. Schutzbach. 1989, “Illinois’ Experience with a Recycled Concrete Inlay,” Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafayette, IN.

3. American Concrete Pavement Association (ACPA). 1993, “Recycling Concrete Pavement,” Technical Bulletin TB-014P, American Concrete Pavement Association, Arlington Heights, IL.

4. Snyder, M. B. 1994, “Effect of Reinforcement Design and Foundation Stiffness on the Deterioration of Transverse Cracks in Jointed Concrete Pavements,” Proceedings, Third International Workshop on the Design and Evaluation of Concrete Roads, Vienna, Austria.

5. Forster, S. W. 1985, “The Use of Recycled Portland Cement Concrete (PCC) as Aggregate in PCC Pavements,” Public Roads, Volume 49, No. 2, Federal Highway Administration, Washington, DC.

6. Yrjanson, W. A. 1989, “Recycling of Portland Cement Concrete Pavements,” NCHRP Synthesis of Highway Practice 154, Transportation Research Board, Washington, DC.

7. Abel, et al. 1983, “Rehabilitation of Concrete Pavements,” CDOH-83-1, Colorado Department of Highways, Denver, CO.

8. Horan, R. 1986, “Recycled Concrete Pavement,” Demonstration Project 47, Wyoming Department of Transportation, Cheyenne, WY.

9. Wade, M. J., G. D. Cuttell, J. M. Vandenbossche, H. T. Yu, K. D. Smith, and M. B. Snyder. 1997, “Performance of Concrete Pavements Containing Recycled Concrete Aggregate,” FHWA-RD-96-164, Federal Highway Administration, Washington, DC.

10. Halverson, A. D. 1981, “Recycling Portland Cement Concrete Pavements,” Transportation Research Record 853, Transportation Research Board, Washington, DC.

11. Haas, S. 1985, “Recycling D-Cracked Portland Cement Concrete Pavements in North Dakota,” Transportation Research Record 1040, Transportation Research Board, Washington, DC.

12. Hankins, R. B. and T. M. Borg. 1984, “Recycling PCC Roadways in Oklahoma,” Transportation Research Record 986, Transportation Research Board, Washington, DC.

13. Parry, J. M. 1990, “A Summary of Wisconsin Experience with D-Cracking and PCC Pavement Recycling,” Proceedings, National D-Cracking Workshop, Kansas Department of Transportation, Topeka, KS.

14. Cross, S. A., M. N. Abou-Zeid, J. B. Wojakowski, and G. A. Fager. 1996, “Long-Term Performance of Recycled Portland Cement Concrete Pavement,” Transportation Research Record 1525. Transportation Research Board, Washington, DC.

15. Raja, Z. I. and M. B. Snyder. 1991, "Factors Affecting Deterioration of Transverse Cracks in Jointed Reinforced Concrete Pavements," Transportation Research Record 1307, Transportation Research Board, Washington, DC.

16. Kuhlmann, B. 1996. "Recycling of Continuously Reinforced Portland Cement Concrete Pavement." Final Report, IR-029-2(52)100. North Dakota Department of Transportation, Bismarck, ND.

17. Bergren, J. V. and R. A. Britson. 1977, "Portland Cement Concrete Utilizing Recycled Pavement," Proceedings, International Conference on Concrete Pavement Design, Purdue University, West Lafayette, IN.

18. Federal Highway Administration (FHWA). 1992, "Report on the 1992 U.S. Tour of European Concrete Highways," FHWA-SA-93-012, Federal Highway Administration, Washington, DC.

19. Federal Highway Administration (FHWA). 1985, Pavement Rehabilitation Manual, FHWA-ED-88-025, Federal Highway Administration, Washington, DC (Manual supplemented April 1986, July 1987, March 1988, February 1989, October 1990).

20. Strand, D. L. 1985, "Designing for Quality: Concrete Pavement Rehabilitation and Recycling on Wisconsin's Interstate Highways," Proceedings, Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafayette, IN.

21. Dykins, S. E. and J. A. Epps. 1987, "Portland Cement Concrete Pavement Pulverizing Equipment," Transportation Research Record 1126, Transportation Research Board, Washington, DC.

22. Federal Highway Administration (FHWA). 1987, "Pavement Recycling Guidelines for Local Governments," FHWA-TS-87-230, Federal Highway Administration, Washington, DC.

23. Young, J. F. 1980, "Contamination Problems in the Recycling of Concrete," Proceedings, NATO Advanced Research Institute on Adhesion Problems in the Recycling of Concrete, Paris, France.

24. Lane, K. R. 1980, "Construction of a Recycled Portland Cement Concrete Pavement," FHWA/CT-80-12, Connecticut Department of Transportation, Hartford, CT.

25. Fergus, J. S. 1981, "Laboratory Investigation and Mix Proportions for Utilizing Recycled Portland Cement Concrete as Aggregate," Proceedings, National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, Federal Highway Administration, Washington, DC.

26. Schutzbach, A. M. 1992, "Case Study of a Full-Depth Asphalt Concrete Inlay," Transportation Research Record 1337, Transportation Research Board, Washington, DC.

27. Frandistou-Yannas, S. 1980, "Economics of Concrete Recycling in the United States," Proceedings, NATO Advanced Research Institute on Adhesion Problems in the Recycling of Concrete, Paris, France.

28. Sri Ravindrarajah, R. Y. H. Loo, and C. T. Tam. 1987, “Recycled Concrete as Fine and Coarse Aggregate in Concrete,” Magazine of Concrete Research, Volume 39, No. 141, Cement and Concrete Association, Buckinghamshire, England.

29. Berger, R. L. and S. H. Carpenter. 1980, “Recycling of Concrete into New Applications,” Proceedings, NATO Advanced Research Institute on Adhesion Problems in the Recycling of Concrete, Paris, France.

30. Darter, M. I. 1988, “Initial Evaluation of Michigan JRCP Crack Deterioration,” Technical Report, Michigan Concrete Paving Association, Lansing, MI.

31. Larson, R. M., S. Vanikar, and S. Forster. 1993, “Summary Report—U.S. Tour of European Concrete Highways (U.S. TECH) Follow-up Tour of Germany and Austria,” FHWA-SA-93-080, Federal Highway Administration, Washington, DC.

32. Halm, H. J. 1980, “Concrete Recycling,” Transportation Research News, Volume 89, Transportation Research Board, Washington, DC.

33. McCarthy, G. J. 1985, “Recycling of Concrete Freeways by Michigan Department of Transportation,” Transportation Research Record 1040, Transportation Research Board, Washington, DC.

34. Epps, J. A., D. Little, R. Holmgren, and R. Terrell. 1980, “Guidelines for Recycling Pavement Materials,” NCHRP Report 224, Transportation Research Board, Washington, DC.

35. Dierkes, J. H., Jr. 1981, “Urban Recycling of Portland Cement Concrete Pavement—Edens Expressway, Chicago, Illinois,” Proceedings, National Seminar on Portland Cement Concrete Pavement Recycling and Rehabilitation. FHWA-TS-82-208. Federal Highway Administration, Washington, DC.

36. Krueger, O. 1981, “Edens Expressway Pavement Recycling—Urban Pavement Breakup, Removal, and Processing,” Proceedings, National Seminar on Portland Cement Concrete Pavement Recycling and Rehabilitation, FHWA-TS-82-208, Federal Highway Administration, Washington, DC.

37. Nelson, L. A. 1981, “Rural Recycling,” Proceedings, National Seminar on Portland Cement Concrete Pavement Recycling and Rehabilitation, FHWA-TS-82-208, Federal Highway Administration, Washington, DC.

MODULE 4-13

RIGID PAVEMENT OVERLAYS

1. INSTRUCTIONAL OBJECTIVES

This module describes the design and use of rigid pavement overlays for existing rigid and flexible pavements. Upon completion of this module, the participants shall be able to:

1. List the types of rigid pavement overlays.
2. Discuss the importance of the bonding condition for each rigid pavement overlay type.
3. Identify the conditions for which each rigid pavement overlay type is best suited and is most cost-effective.
4. Recognize the different design methodologies for rigid pavement overlays.
5. Describe the level of preoverlay repair required for each rigid pavement overlay type and its relative importance.

2. INTRODUCTION

Overlays are one of the most popular rehabilitation strategies for deteriorated pavements. Although HMA overlays have historically seen the greatest use in the pavement rehabilitation arena, the use of rigid pavement overlays on both existing rigid and flexible pavements has increased significantly in the last few years. Contributing factors to the increased use of rigid pavement overlays include more effective pavement evaluation procedures, improvements in portland cement concrete (PCC) paving materials (particularly high-early strength materials), and advancements in rigid pavement paving technology. Furthermore, the long life and low maintenance requirements of rigid pavement overlays make them an attractive rehabilitation alternative for many highway agencies.

Like new rigid pavement construction, rigid pavement overlays are constructed using conventional paving equipment and materials, although unique paving equipment and mixes (for example, zero-clearance pavers and fast track mixes) are available for certain applications. Special design considerations for rigid pavement overlays include the condition of the underlying pavement, the bonding requirements between the existing pavement and the new rigid overlay, and the jointing requirements of the new pavement. A variety of rigid pavement overlays are available for different applications, each of which is discussed in this module.

As the use of rigid pavement overlays has increased, substantial insight has been gained on their design, construction, and performance. Such information is well summarized in a recent synthesis(1) and that document should be referred to for more detailed information on rigid pavement overlays.

3. DEFINITIONS

A variety of rigid pavement overlays are used on existing pavements, depending primarily upon the type of existing pavement and the proposed bonding condition between the rigid overlay and the existing pavement. These different rigid pavement overlay types are summarized in table 4-13.1 and described in the following sections.

Table 4-13.1. Summary of rigid pavement overlay types.

Bonding Condition	How Condition Achieved	Type and Condition of Existing Pavement	Preoverlay Repair	Critical Design Considerations	Overlay Pavement Types
Rigid Overlays of Rigid Pavements					
Bonded	• Surface preparation (e.g., shotblasting) • Bonding agent	All rigid pavement types in relatively good condition	Most deteriorated cracks, joints, punchouts	• Bonding • Match joints	JPCP, JRCP, CRCP
Partially Bonded[1]	• No special preparation	All rigid pavement types in moderate condition	Limited repair and surface sweeping	• Match joints	JPCP, JRCP
Unbonded	• Separation layer	All rigid pavement types in deteriorated condition	Limited repair of deteriorated cracks, joints, punchouts	• Achieve layer separation • Mismatch joints	JPCP, JRCP, CRCP
Rigid Overlays of Flexible Pavements (whitetopping)					
Semi-Bonded[2]	• No special preparation *(except for ultra-thin whitetopping overlays that require bonding between the PCC and HMA)*	All flexible pavements in deteriorated condition	Limited repair of areas of inadequate support and possible cold milling to restore cross profile	• Uniform support	JPCP, JRCP, CRCP

[1] Partially bonded overlays not recommended for highway applications.
[2] Rigid overlays of flexible pavements designed as unbonded, but some degree of bonding occurs between the two materials. In ultra-thin overlays, bond between the two materials is critical to performance.

Rigid Overlays of Existing Rigid Pavements

The bonding condition (sometimes referred to the interface condition) between the new rigid overlay and the existing rigid pavement is a primary design concern for rigid overlays of existing rigid pavements. The bonding condition, which is generally selected based on the condition of the existing rigid pavement, may be one of the following:

- <u>Bonded</u> indicates that specific measures are taken to enhance the bonding between the rigid overlay and the existing slab. This usually includes extensive surface preparation of the existing rigid pavement (e.g., shotblasting) and often the placement of a cement grout on the existing rigid pavement just ahead of the paver. The intent of bonding an overlay to the existing pavement is to obtain a pavement system that behaves monolithically, as shown in figure 4-13.1.

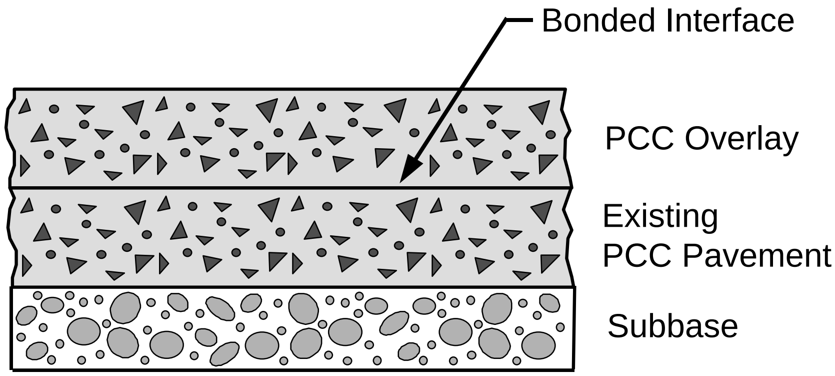


Figure 4-13.1. Bonded rigid pavement overlay.[1]

Bonded overlays are used to increase the structural capacity of the pavement, or to improve its overall ride quality. They are typically used where the underlying pavement is in relatively good condition.[1,2] If distressed pavements are overlaid without substantial preoverlay repair, significant reflection cracking will occur in the new overlay because of bonding between the two layers. Typical bonded overlay thicknesses range from about 75- to 150-mm.

The use of a bonding agent has become the topic of some debate among design engineers. At least one research study has indicated that a bonding agent is not needed for a bonded rigid overlay.[3] In some cases, it has been shown that a bonding agent actually inhibits bond, but this is most likely due to the bonding agent drying significantly before the actual paving. Agencies must determine the efficacy of using a bonding agent for their own local conditions.

- Partially bonded indicates that no real measures are taken to either enhance or prevent bonding from occurring between the rigid overlay and the existing rigid slab. Although used in airport applications, this bonding condition has not seen widespread use in the highway field and, therefore, is not discussed further.

- Unbonded indicates that special actions are taken to prevent bonding between the rigid overlay and the existing rigid pavement, effectively separating them so that they move independently from one another (see figure 4-13.2). Various materials may be used as a bondbreaker; a thin (25- to 50-mm) interlayer of hot-mix asphalt (HMA) covered with a membrane curing compound placed on the existing pavement is most commonly used.

Unbonded overlays are used when the existing pavement distress is so advanced that it cannot be effectively corrected prior to overlaying.[4] The expectation is that the placement of the separation layer will prevent the development of reflection cracking in the new overlay. A small amount (if any) of preoverlay repair may be required and mismatching of the overlay joints and the existing rigid pavement joints is generally recommended. In some cases, cracking/breaking and seating of the rigid pavement is performed prior to the placement of the unbonded overlay. Typical unbonded overlay thicknesses range from about 150 to 300 mm, depending on conditions.

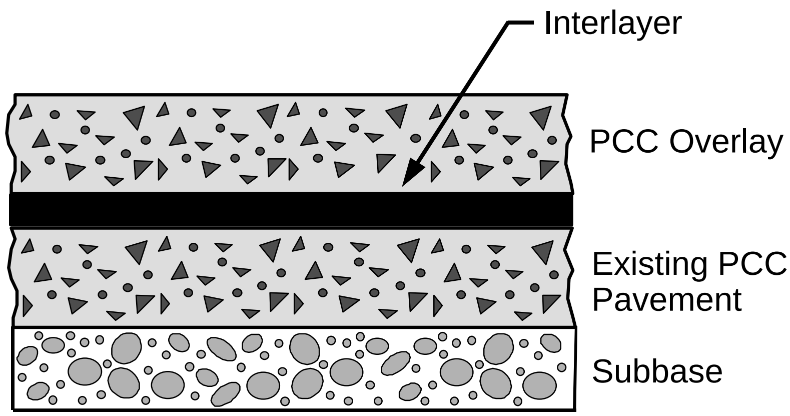


Figure 4-13.2. Unbonded rigid pavement overlay.[1]

As in new construction, rigid overlays of existing rigid pavements may be either jointed plain concrete pavements (JPCP), jointed reinforced concrete pavements (JRCP), or continuously reinforced concrete pavements (CRCP). JPCP is the most common overlay type and may be placed in either a bonded or unbonded condition. JRCP overlays, containing distributed steel throughout the new slab, can also be either bonded or unbonded, although their use in an unbonded condition is more common.[1] CRCP overlays contain heavy reinforcing steel (typically 0.7 to 0.8 percent of the cross-sectional area) and are generally placed in an unbonded condition. A fourth rigid pavement type, fibrous PCC overlays—rigid pavement overlays containing steel or synthetic fibers distributed in the PCC mix—have seen some use in the highway field.[1] Although very few projects have been constructed since 1980, the last few years have seen a number of fibrous PCC overlays constructed under the provisions of the 1991 surface transportation legislation.

Rigid Pavement Overlays of Existing Flexible Pavements (Whitetopping)

Rigid pavement overlays of existing flexible pavements, also known as whitetopping, have become a common rehabilitation practice for some agencies.[1] Whitetopping is considered an ideal solution to rehabilitating deteriorated flexible pavements that exhibit such distresses as rutting, shoving, and alligator cracking.[5] A limited amount of preoverlay repair is generally required, with many cold milling the flexible surface to remove ruts or surface irregularities; on the other hand, some agencies place the new rigid overlay directly on the existing flexible pavement without any treatment. A typical cross section of a whitetopping paving project is shown in figure 4-13.3.

Most design procedures for whitetopping consider the existing flexible pavement as a base course and slab thicknesses are developed accordingly, generally assuming an unbonded condition. However, in the design of whitetopping overlays, it is important to adequately characterize the support provided by the flexible pavement through deflection testing. JPCP, JRCP, and CRCP have all been used successfully in whitetopping applications.

A subset of the rigid pavement whitetopping family currently generating considerable interest in the pavement community is ultra-thin whitetopping (UTW). UTW is a thin, high-strength PCC overlay placed on an existing flexible pavement. Intended for parking lots, residential streets, and low volume

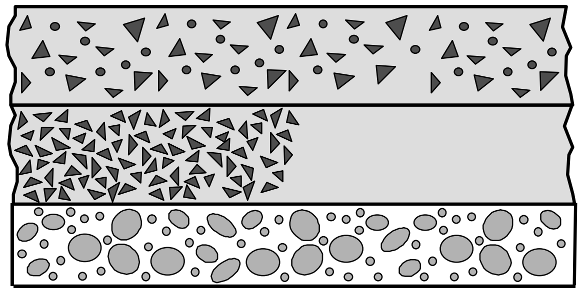


Figure 4-13.3. Rigid pavement overlay of flexible pavement (whitetopping).[1]

roads (design ESALs less than 1 million), UTW slabs are generally less than 100 mm thick and typically jointed at 2 m or less; they also often contain steel or synthetic fibers in the PCC mix.[5] Much of the effectiveness of UTW applications is dependent upon the condition of the underlying AC and on the bond developed between the two materials.[6]

4. PURPOSE AND APPLICATIONS

As the preceding discussion indicates, a variety of rigid pavement overlays may be appropriate rehabilitation techniques, depending on the type and condition of the existing pavement. In general, bonded rigid pavement overlays are most suited to existing rigid pavements in relatively good condition. These overlays are most appropriate when the existing rigid pavement is in need of structural improvement for the current or projected traffic loadings; by bonding the rigid pavement overlay to the existing pavement, a monolithic structure of increased load-carrying capacity is achieved.

Unbonded rigid pavement overlays are best suited to existing rigid pavements that are showing significant levels of deterioration. By separating the overlay from the existing pavement, the two slabs can act independently and the rigid pavement overlay will be unaffected by the condition and action of the underlying pavement.

Whitetopping (rigid pavement overlays of flexible pavements) are an attractive alternative for badly distressed flexible pavements. In lieu of removing the flexible pavement and reconstructing, the existing flexible pavement is treated as a base course for the new rigid pavement. Some preoverlay repair may be required, particularly in areas of localized support failures or in areas of severe rutting or shoving. On the other hand, some agencies do not perform any preoverlay repair for their whitetopping projects, instead assuming that any ruts or depressions will be filled with concrete, which provides additional pavement thickness in the wheelpath areas where it is most needed.

UTW overlays are best suited to flexible pavements on local roads, intersections, or parking lots that frequently experience severe rutting, shoving, and pothole problems. The placement of the UTW will provide a durable surface more resistant to the forces that create those distresses. In order for the UTW to perform as intended, it is essential that adequate bonding be achieved between the new rigid pavement overlay and the existing HMA pavement; this is usually achieved through milling and sweeping of the flexible surface.

Although flexible overlays could be considered an alternative for any of the above scenarios, rigid pavement overlays offer the advantage of longer service lives, lower maintenance requirements, and reduced occurrences of future traffic disruptions.

5. LIMITATIONS AND EFFECTIVENESS

Bonded Overlays

The field performance of bonded overlays has been mixed.[1,7,8] While many projects have provided good long-term performance, a few have failed in the first few years after construction. These failures are often characterized by reflective cracking and corner breaks (in areas where bond has been lost). Most of these failures, however, were attributed to the application of the bonded overlay to a pavement that was too far deteriorated.[1] Others are attributed to inadequate or ineffective preoverlay repair.[8] It is emphasized that bonded concrete overlays must be placed on concrete pavements still in good condition, but in need of additional structure.

The establishment of an effective bond between the two PCC layers is also critical to the performance of the bonded overlay. Shotblasting and sandblasting are most commonly used, with shotblasting being judged more effective in producing a textured surface.[1] Cold milling has been used on some projects, but some engineers believe that it creates micro-cracks in the existing rigid pavement.[1]

The bonding of the rigid pavement overlay is also greatly affected by the prevailing climatic conditions at the time of construction, such as ambient temperature, humidity, and wind speed.[9] If significant stresses develop during the first 72 hours following PCC placement, debonding of the overlay from the underlying pavement may occur. Researchers have developed a program that predicts the development of interface bond stresses and strengths to assess the possibility of early-age failures of the overlay.[9]

The issue of whether a bonding agent is required for bonded overlays is still being debated by design engineers. Research on this topic has also shown mixed results with at least one study indicating that a bonding agent is not required.[3] If a bonding agent is used (such as a cement grout material), it is critical that the bonding agent is placed just in front of the paver and that the material not dry before placement of the rigid pavement overlay.[1]

Unbonded Overlays

Unbonded rigid pavement overlays have generally provided good performance. For this type of overlay, the performance and effectiveness of the overlay is less affected by the condition of the underlying pavement and more by a competent overlay thickness design. Some unbonded rigid overlays have provided 30 or more years of performance.[1]

Where the performance of unbonded rigid pavement overlays has been compromised, factors that have been identified as adversely affecting the performance of the overlay include:[1]

- Inadequate thickness of separator layer.
- Unanticipated bonding at the interface.
- Nonuniform support conditions.

- Inadequate load transfer.
- Inadequate drainage.

Current practice is to include a thicker separator layer (e.g., 25 to 50 mm of HMA) to ensure that the two slabs act independently.

Random slab cracking has developed on many unbonded rigid overlay projects, and this is often attributed to the use of excessive joint spacing.[7] For JPCP designs, the current recommendation is that the maximum joint spacing of the unbonded rigid pavement overlay should be limited to 21 times the slab thickness; that is:

$$S_{max} = 21 * t \tag{4-13.1}$$

where:

S_{max} = Maximum joint spacing, mm
t = Slab thickness, mm

For example, if an unbonded overlay is 250 mm thick, the maximum joint spacing should be limited to 21* 250 = 5,250 mm (or 5.25 m). The joint spacing for JRCP unbonded overlays may be the same as used in conventional JRCP construction, but the joints should be mismatched.

Whitetopping

The number of whitetopping projects has increased dramatically over the past few years as more and more agencies consider the use of a rigid overlay on a distressed flexible pavement. Although many projects are too new to assess their effectiveness, those that have been in service for several years are generally providing good to excellent performance.[1] The success of this design is often attributed to the uniform support and bond provided by the underlying flexible pavement.[1,10]

As discussed previously, the performance of UTW projects has been promising, with a recent evaluation of ten UTW projects revealed that all projects are in very good condition.[11] A greater percentage of cracking is observed at the approach and leave ends of the UTW paving area, perhaps due to debonding and vehicle impact loading at those locations.[11] A preliminary procedure for the design of UTW overlays has been developed that considers critical tensile stresses at the top corner of the UTW and in the underlying HMA layer.[12]

6. CONSIDERATIONS IN RIGID PAVEMENT OVERLAY SELECTION

In evaluating the suitability of a rigid overlay for an existing pavement, a detailed evaluation of the pavement must be performed. Information on the type, extent, and severity of key distresses must be collected through a pavement condition survey. Where a structural deficiency is suspected, additional information on the structural integrity of the existing pavement and the condition of the underlying layers should be obtained through a combination of deflection testing and subsurface boring.

A thorough examination of pavement deficiencies and the cause of deterioration must be made prior to selecting a rigid overlay as a candidate repair alternative. There is a point where the extent of pavement deterioration could make reconstruction more cost-effective than placing a thick overlay. For example, poor drainage and poor base or subbase support can not be corrected through the placement of a rigid overlay. Also, reconstruction may be the desired alternative when serious concrete durability problems are present in the existing concrete. Figure 4-13.4 shows the conceptual relationship between a pavement's condition and the optimum rehabilitation activity.[13] This figure shows that bonded overlays are most applicable when the pavement is still in good condition, whereas unbonded overlays are more suitable when the pavement is deteriorated. Although this figure was developed for rehabilitation of rigid pavements, the concept is applicable for flexible pavements as well.

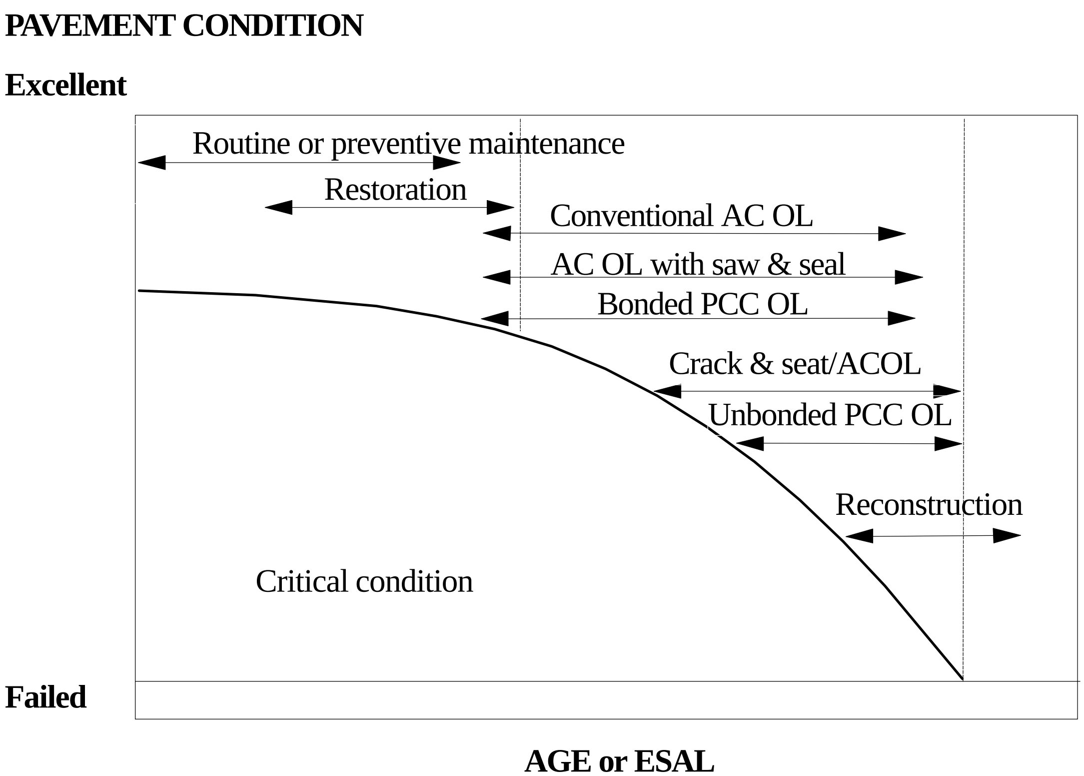


Figure 4-13.4. The spectrum of pavement rehabilitation alternatives.[13]

If a rigid overlay has been selected as an alternative for a particular project, it is important that its overall feasibility be evaluated. This determination is based on consideration of construction feasibility, desired performance period, and available funding. These critical aspects are described below and are summarized in table 4-13.2.

Construction Feasibility

The first step in determining if a rehabilitation alternative is feasible is to determine if it is constructible. The major factors that determine construction feasibility are described below.

<u>Traffic Control</u>

Generally, rigid overlays have significant traffic control requirements, although the introduction of fast-track paving, zero-clearance pavers, and unique traffic control plans have greatly increased their construction feasibility. Furthermore, with additional life of rigid overlays, a reduction in the number of future traffic delays and/or detours can be expected.[1]

Table 4-13.2. Feasibility guidelines for rigid overlays.[13]

	BONDED	UNBONDED	WHITETOPPING
CONSTRUCTIBILITY			
Vertical Clearance	Usually not a problem	Can often be a problem.	Can be a problem.
Traffic Control	Somewhat difficult to construct under traffic.	Somewhat difficult to construct under traffic.	Somewhat difficult to construct under traffic
Construction	Trained personnel required. Special cleaning equipment required. Achievement of bond critical. Construction experience limited.	Does not require special equipment.	Does not require special equipment.
PERFORMANCE PERIOD			
Existing Condition	No D-cracking or extensive cracking.	Can be applied to very deteriorated rigid pavements.	Can be applied to very deteriorated flexible pavements.
Extent of Repair	Must repair deteriorated cracks and joints.	Very little repair is needed. Crack/break and seat may be considered.	Very little repair is needed.
Structural Adequacy	Thickness can be increased to provide structural adequacy.	Substantial thickness must be provided for structural adequacy.	Substantial thickness may be required for structural adequacy.
Future Traffic	Used under any traffic level.	Used under any traffic level.	Used under any traffic level.
Reliability	Fair to poor (reflection cracking may be a problem).	Very good.	Very good.
COST-EFFECTIVENESS			
Initial Cost	Usually relatively high. Depends on preoverlay repair.	Higher than conventional flexible overlay.	Higher than conventional flexible overlay.
Life-Cycle Cost	Competitive if future life is substantial.	Competitive.	Competitive.

Constructibility

Bonded rigid pavement overlays require special care during preparation and placement to ensure good long-term performance. Unbonded and whitetopping overlays utilize conventional paving equipment and procedures.

Overhead Vertical Clearances and Other Surface Elevation Change Problems

All rigid overlay types have the disadvantage of decreasing overhead clearances by adding additional thickness to the surface. However, correcting the same deficiency might require different thicknesses for different types of overlays. Where clearance problems might exist, they must be considered during initial design and steps must be taken to provide adequate clearance. This can be accomplished by using thinner overlays under structures (undesirable), by reconstructing the pavement directly under the structure with tapers to the overlaid sections on either side, or by milling off additional thickness of the existing flexible pavement (for whitetopping). In rare instances, it may be more cost-effective to raise the structures. If many overhead bridges exist along the project, a thick overlay may result in prohibitively high costs.

Overlaying traffic lanes also necessitates overlaying shoulders, which may be in good condition and not in need of an overlay. Overlays may also require the raising of guard rails and the placement of additional fill material adjacent to shoulders. Overlays also decrease curb height and could disturb drainage patterns. One innovative method that has been used in this situation is a PCC inlay, in which the existing pavement is milled or removed and a new rigid pavement overlay is placed between the existing shoulders.

Performance Period

The AASHTO Guide defines a rehabilitation's performance period as the amount of time that a rehabilitation will last before the pavement will again require some type of rehabilitation.[14] Because of the difficulty, hazards, and costs involved with closing traffic lanes, some minimum life must be attainable with the proposed rehabilitation. The life of the rehabilitation alternative depends upon many factors:

- Existing pavement type and design.
- Existing pavement condition.
- Structural adequacy to carry future traffic.
- Material deterioration within the pavement structure.
- Climate (temperature, moisture, freeze-thaw cycling).
- Subdrainage adequacy.
- Presence of swelling soils or frost heave.
- Extent of repair performed.

The extent of preoverlay repair performed may be the single most important factor in determining how well the overlay will perform and is discussed in a later section. One other factor that may influence the performance of the rigid overlay is the occurrence of reflection cracking. Reflection cracking is the development of cracks in the overlay caused by horizontal and vertical movement of cracks and joints in the underlying existing pavement. It can be a severe problem in bonded rigid overlays, but is typically not a problem in unbonded rigid overlays. Methods currently used to prevent or control reflection cracking are discussed in detail in module 4-14 in relation to flexible pavement overlays of rigid pavements; for rigid overlays, reflection crack control is best achieved through overlay type selection and joint matching. Consideration can also be given to retrofitting load transfer devices at joints and cracks in the existing pavement to reduce vertical movements beneath the overlay (see module 4-9).

Funding

If the predicted performance period and reliability are acceptable, an estimate of an overlay's initial construction cost is made to ensure that it is within the funds available for the project. If so, then the rehabilitation alternative is feasible. A comparison of life-cycle costs of all feasible alternatives to identify the most cost-effective alternative can then be made.

7. RIGID PAVEMENT OVERLAY DESIGN PRINCIPLES

Slab Thickness Design

There is no universally accepted design procedure for rigid overlays. Although not discussed in detail in this course, several general approaches to overlay design are briefly described below. Overlay thickness design procedures either assume that adequate preoverlay repair and reflective crack control actions are taken, or they permit different levels of repair to be considered. If preoverlay and reflective crack control treatments are not adequate, then the overlay will likely fail prematurely and not provide the desired structural service. Each agency is encouraged to calibrate or modify the design procedures to suit local conditions.

Engineering Judgment

A number of agencies rely on the judgment and experience of their engineers in determining the required overlay thicknesses and bonding conditions. Some agencies have monitored the performance of previous overlays and can estimate how selected overlays might perform. There are obvious deficiencies to this approach, because very few engineers have adequate experience to determine the required overlay thickness for a given traffic and design life. The development of an overlay design procedure that quantitatively considers the important design factors is strongly recommended.

Structural Deficiency

In the structural deficiency approach to overlay design, the required overlay thickness is equal to the difference between the structural capacity of a new pavement and the structural capacity of the existing pavement. The difference in structure represents the theoretical structural deficiency that must be met by the overlay. If both the pavement structures are made of concrete, then the difference is the thickness of concrete directly. If not, equivalency factors must be used to convert the thickness of the existing HMA into an equivalent thickness of PCC. This is by far the most common approach for the design of overlays, and may be expressed mathematically as follows:

$$SC_{OL} = SC_f - SC_{eff} \tag{4-13.2}$$

where:

SC_{OL} = Required structural capacity of rigid overlay.
Sc_f = Structural capacity of new rigid pavement for design conditions.
SC_{eff} = Effective structural capacity of existing pavement.

For example, assume a bonded rigid overlay is being designed for an existing rigid pavement. If it is determined that the existing concrete pavement has an effective "structure" of 175 mm, and the thickness of a new rigid pavement designed for the current design conditions and projected traffic loadings is 275 mm, then the required rigid overlay thickness is (275 – 175) = 100 mm.

In 1993, AASHTO released a new overlay design methodology based on the structural deficiency approach.[14] In the new procedure, the existing or effective structural capacity of the pavement is evaluated (using visual surveys and materials testing, nondestructive testing, or remaining life procedures) and this effective structure is subtracted from the required structural capacity of the "new" pavement. The reliability of the design is based on the ability to accurately determine the effective structural capacity of the existing pavement. Key design steps in the AASHTO procedure include:

- Collection of existing pavement design and construction information.
- Development of traffic projections.
- Conduct condition survey.
- Perform deflection testing.
- Perform destructive testing (coring and boring).
- Determination of structural capacity of new pavement for given design conditions.
- Determination of effective structural capacity of existing pavement.
- Determination of overlay thickness.

The most difficult step in this process is the determination of the effective structural capacity of the existing pavement. Because that pavement has been subjected to traffic and environmental loadings over its life, its effective structure is not equivalent to its slab thickness, but instead will typically be something less. Where practical, the AASHTO procedure provides several methods for determining the effective structural capacity of the existing pavement, and encourages the use of deflection testing to assist in its evaluation.

For whitetopping projects, the rigid overlay is designed as a new rigid pavement using the AASHTO procedure. Thus, the structural deficiency approach does not apply to this overlay type. The key aspect of the whitetopping designs is adequately characterizing the underlying flexible pavement and the support that it provides to the rigid pavement.

<u>Mechanistic Fatigue Damage Approach</u>

The basic concept of the mechanistic fatigue damage approach to overlay design is that the required overlay thickness is one that will limit fatigue damage in the overlay or the existing pavement to an acceptable level over the design period. The existing pavement and overlay may be modeled using plate theory or finite element analysis. The amount of fatigue damage expected in the overlay depends primarily on the trial overlay thickness and the number of expected load repetitions. The total fatigue damage expected in the existing pavement not only depends on those factors, but also on the past fatigue damage accumulated prior to the overlay. Whether fatigue damage in the overlay or in the existing pavement should control the design depends on the type of overlay and type of existing pavement.

The mechanistic fatigue damage approach to rigid pavement overlay design has not been widely validated by field performance data and so far has seen only limited use. One of the major limitations of this approach is that it only considers one failure mode (fatigue cracking).

Other Design Considerations

There are many other factors besides slab thickness to consider in the design of rigid pavement overlays, including bonding condition, drainage, reinforcement, and joint design. While a complete discussion of these factors is not warranted here, a brief discussion is presented below.

Bonding Condition

As discussed previously, the bonding condition between the rigid overlay and the underlying pavement greatly affects performance. For bonded overlays, a strong bond is counted on in order to elicit monolithic behavior from the two layers; if such bonding is not achieved, cracking of the overlay will result. Shotblasting appears to provide the most effective surface texture for bonded rigid overlays.

Although whitetopping overlays are generally designed assuming an unbonded condition, some bonding between the rigid overlay and the flexible pavement will occur and actually contributes to the performance of the overlay; some significant level of bonding is particularly important for UTW overlays. Unbonded rigid pavement overlays, on the other hand, require total separation between the two layers in order to prevent reflection cracking from occurring.

Drainage

As with any pavement structure, adequate drainage is important to ensure that the rigid pavement overlay achieves its design life. If poor drainage conditions are contributing to the deterioration of the existing pavement, adding an overlay will not correct the problem. Most overlay design procedures assume that the existing pavement has good drainage.

As part of the pavement evaluation process, it is always advisable to conduct a thorough drainage survey to identify drainage-related distresses and develop solutions that address these distresses as part of the overlay design process. While constraints exist as to what can be done to improve drainage on an overlay project, consideration should be given to the installation of an edge drain system and to the establishment of adequate drainage ditches for the pavement. On unbonded rigid pavement overlays, a few agencies have experimented with the use of an open-graded drainage material as an interlayer.[1]

Reinforcement

Longitudinal distributed reinforcing steel is placed in JRCP and CRCP overlays to help control transverse cracks that develop in those designs. Reinforcing steel design for these overlay types is the same as for conventional reinforced designs, with minimum steel contents of 0.1 and 0.6 percent recommended for JRCP and CRCP, respectively;[15] typically, steel contents for JRCP range between 0.1 and 0.25 percent and for CRCP between 0.65 and 0.85 percent. In addition, consideration should be given to providing sufficient slab thickness to ensure a minimum steel cover of 75 mm.

Joint Design

For rigid pavement overlays, the location of the joints is a function of the overlay bonding condition. For bonded rigid overlays, joints in the overlay must match those of the underlying rigid pavement in order to prevent reflection cracking. Furthermore, it is recommended that the sawcut depth be through the entire thickness of the bonded overlay to prevent the buildup of compressive stresses and to prevent the development of any secondary cracking.[1]

For unbonded rigid overlays, the joints are typically mismatched by about 0.6 to 0.9 m in order to provide a localized strong area of support beneath the rigid overlay joints.[4] It is recommended that the overlay joint be placed on the approach side of an existing joint to inhibit the typical pumping phenomenon at the existing joint.[1] As previously discussed, the maximum joint spacing for JPCP is limited to less than 21 times the slab thickness.

Overlay joints in whitetopping pavements generally use the same spacing used by the agency for their conventional PCC paving. In the case of UTW projects, a maximum joint spacing of 1.2 m by 1.2 m is currently recommended.[12]

Mechanical load transfer devices (dowel bars) are generally recommended for most rigid pavement overlays, except for those on low volume roadways. Depending on the slab thickness, the use of dowel bars may not always be practical for bonded overlays. If load transfer is a concern, retrofitted load transfer devices may be installed in joints and cracks of the existing rigid pavement.

Construction Guidelines

Although conventional PCC paving practices are used for most rigid pavement overlay applications, each has unique construction requirements. Appropriate design and construction guidelines for rigid pavement overlays should be consulted for additional information. See references 2, 4, 5, 16, and 17.

8. PREOVERLAY TREATMENT AND REPAIR

The amount of repair and treatment that is performed to a pavement prior to receiving a rigid pavement overlay can be a key factor influencing the future performance of the overlay, particularly for bonded rigid overlays. The amount and type of preoverlay restoration needed on an existing pavement must be carefully determined by considering the following factors:

- Type of overlay.
- Structural adequacy of the existing pavement.
- Distress type exhibited by the existing pavement.
- Future traffic loadings.
- Physical constraints such as traffic control.
- Overall costs (preoverlay repair and overlay).

Depending on the type of rigid overlay, a certain amount of preoverlay treatment and repairs is expected to be performed as part of the overlay design. However, for the given application, the relative amount of preoverlay repair must be evaluated to ensure that excessive repairs are not being made. For example, if a bonded rigid overlay is being contemplated, but will require a significant amount of preoverlay repair in order to bring it to an acceptable condition level, the existing pavement may be better suited to an unbonded rigid overlay.

The following are examples of preoverlay repairs that might be required for each rigid overlay type:[1]

- Bonded Rigid Overlays
  - Full-depth repair of severely deteriorated joints.
  - Load transfer restoration or full-depth repair of working cracks (tight cracks left alone).
  - Grinding of minor joint faulting and possible slab stabilization.
  - Cross stitching of longitudinal cracks (if the cracks are working).

- Unbonded Rigid Overlays
  - Full-depth repair of severely deteriorated joints and cracks.
  - Milling of joint faulting greater than 6 mm.
  - Full-depth repair of punchouts.
  - Crack/break and seat of existing rigid pavement to provide more uniform support.

- Whitetopping
    - Localized repair of failed areas.
    - Cold milling to restore profile and remove rutting/shoving.
    - Leveling course to produce uniform surface for paving.

Detailed information on reflection crack control, a major design concern for bonded rigid overlays, is provided in module 4-14.

9. SUMMARY

This module summarizes the different types of rigid pavement overlays. Three primary rigid overlay types are used on existing rigid pavements—bonded, unbonded, and partially bonded—but only the bonded and unbonded cases are described in this module due to their greater use in highway applications. Rigid pavement overlays of existing flexible pavements (whitetopping) are also described in this module.

Bonded overlays are used on pavements where additional structure is required, and as such are generally placed on pavements in relatively good condition. Unbonded pavements are used on deteriorated rigid pavements and require little preoverlay repair. Whitetopping projects are effective treatments for deteriorated flexible pavements exhibiting severe structural deterioration.

Bonding is a design element that is critical to the performance of rigid overlays. Bonded overlays require an intimate bond between the rigid overlay and the existing rigid pavement in order for the resulting pavement structure to react monolithically. This is generally achieved through aggressive surface preparation and perhaps the use of a bonding agent.

In unbonded rigid overlays, specific steps are taken to ensure that the rigid overlay and the existing rigid pavement remain unbonded and act independently. This is to reduce the possibility of cracks in the underlying pavement reflecting through the rigid overlay. This separation is generally achieved through the use of an HMA material placed on the existing rigid pavement.

Whitetopping pavements are designed assuming an unbonded condition. However, research studies have indicated that bonding between the rigid overlay and existing flexible pavement does occur and contributes positively to the performance of the pavement.

There are three approaches for determining the thickness required for a structural rigid overlay, with the structural deficiency procedures the most commonly used. The 1993 AASHTO procedure provides a step-by-step procedure for the design of rigid overlays, but each agency is advised to calibrate the design procedure to reflect local conditions. It is also suggested that the design engineer utilize more than one procedure to check the reasonableness of the resulting design.

In addition to slab thickness, there are other factors to consider in the design of rigid overlays. These include, among other things, the bonding condition, the prevailing drainage conditions, reinforcement requirements, and joint design considerations.

The preoverlay repair performed on the existing pavement has a major effect on the performance of the overlay. The amount of preoverlay repair to be conducted on a pavement depends on such factors as the type of overlay being constructed, the condition of the existing pavement, the future traffic loadings, and the overall costs of the preoverlay repair.

10. REFERENCES

1. McGhee, K. H. 1994, "Portland Cement Concrete Resurfacing," NCHRP Synthesis of Highway Practice 204, Transportation Research Board, Washington, DC.

2. American Concrete Pavement Association (ACPA). 1990a, "Guidelines for Bonded Concrete Overlays," Technical Bulletin TB-007P, American Concrete Pavement Association, Arlington Heights, IL.

3. Delatte, N. J., D. W. Fowler, and B. F. McCullough. 1996, "Full-Scale Test of High Early Strength Bonded Concrete Overlay Design and Construction Methods," Transportation Research Record 1544, Transportation Research Board, Washington, DC.

4. American Concrete Pavement Association (ACPA). 1990b, "Guidelines for Unbonded Concrete Overlays," Technical Bulletin TB-005P, American Concrete Pavement Association, Arlington Heights, IL.

5. American Concrete Pavement Association (ACPA). 1991, "Guidelines for Concrete Overlays of Existing Asphalt Pavements (Whitetopping)," Technical Bulletin TB-009P, American Concrete Pavement Association, Arlington Heights, IL.

6. Mack, J. W., L. W. Cole, and J. P. Mohsen. 1994, "Analytical Considerations for Thin Concrete Overlays on Asphalt," Transportation Research Record 1388, Transportation Research Board, Washington, DC.

7. Voigt, G. F., M. I. Darter and S. H. Carpenter. 1987, "Field Performance of Bonded Concrete Overlays," Transportation Research Record 1110, Transportation Research Board, Washington, DC.

8. Peshkin, D. G. and A. L. Mueller. 1990, "Field Performance and Evaluation of Thin Bonded Overlays," Transportation Research Record 1286, Transportation Research Board, Washington, DC.

9. Rasmussen, R. O., B. F. McCullough, D. G. Zollinger, and S. Yang. 1997, "A Foundation for High Performance Bonded Concrete Overlay Design and Construction Guidelines," Proceedings, Sixth International Purdue Conference on Concrete Pavement Design and Materials for High Performance, Purdue University, West Lafayette, IN.

10. Grove, J. D., G. K. Harris, and B. J. Skinner. 1993, "Bond Contribution to Whitetopping Performance on Low-Volume Roads," Transportation Research Record 1382, Transportation Research Board, Washington, DC.

11. Cole, L. W. 1997, "Pavement Condition Surveys of Ultrathin Whitetopping Projects," Proceedings, Sixth International Purdue Conference on Concrete Pavement Design and Materials for High Performance, Purdue University, West Lafayette, IN.

12. Mack, J. W., C. L. Wu, S. Tarr, and T. Refai. 1997, "Model Development and Interim Design Procedure Guidelines for Ultra-thin Whitetopping Pavements," Proceedings, Sixth International Purdue Conference on Concrete Pavement Design and Materials for High Performance, Purdue University, West Lafayette, IN.

13. Darter, M. I. and K. T. Hall. 1990, "Structural Overlay Strategies for Jointed Concrete Pavements, Volume IV—Guidelines for the Selection of Rehabilitation Alternatives," FHWA-RD-89-145, Federal Highway Administration, Washington, DC.

14. American Association of State Highway and Transportation Officials (AASHTO). 1993, "Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials, Washington, DC.

15. Yu, H. T., M. I. Darter, K. D. Smith, J. Jiang, and L. Khazanovich. 1996, "Performance of Concrete Pavements, Volume III—Improving Concrete Pavement Performance," FHWA-RD-95-111, Federal Highway Administration, Washington, DC.

16. Voigt, G. F., S. H. Carpenter, and M. I. Darter. 1989, "Rehabilitation of Concrete Pavements, Volume II—Overlay Rehabilitation Techniques," FHWA-RD-99-072, Federal Highway Administration, Washington, DC.

17. Yu, H. T., D. G. Peshkin, K. D. Smith, M. I. Darter, D. Whiting, and H. Delaney. 1994, "Concrete Rehabilitation—Users Manual," SHRP-C-412, Strategic Highway Research Program, Washington, DC.

MODULE 4-14

HOT-MIX ASPHALT CONCRETE OVERLAYS FOR RIGID PAVEMENTS

1. INSTRUCTIONAL OBJECTIVES

This module describes the use of hot-mix asphalt concrete (HMA) overlays of rigid pavements. The appropriateness of HMA overlays as a rehabilitation technique is described, along with the importance of preoverlay repair and general design and construction information. This module also discusses the problem of reflection cracking, the propagation of cracks or joints from an existing pavement through an overlay. The factors influencing the development of reflection cracking are described, along with an overview of the various treatments that have been used in an attempt to prevent, retard, or control the development of reflection cracking.

Upon completion of this module, the participant should be able to:

1. List the functional and structural deficiencies that can be corrected with HMA overlays.
2. Describe the conditions that affect the performance of an HMA overlay.
3. List the causes of reflection cracking.
4. Discuss the various treatments that have been used to reduce reflection cracking and their relative effectiveness.

2. INTRODUCTION

HMA overlays are the most popular method of rehabilitating existing pavements. They are a cost-effective means of correcting surface deficiencies and increasing a pavement's structural capacity. However, many HMA overlays fail to provide the desired level of performance or service life, due to one or more of the following reasons:

- Improper selection of an HMA overlay as the appropriate rehabilitation method.
- Selection of the wrong type of overlay.
- Inadequate overlay thickness design, mixture design, and joint design.
- Insufficient preoverlay repair of deteriorated areas.
- Lack of direct consideration of reflection cracking.

It is important that the need for an overlay be accurately identified. A realistic evaluation of the existing pavement condition is critical in selecting the most appropriate resurfacing alternative, and in determining the type of preoverlay repair required to prepare the existing pavement for an overlay.

The design of the overlay is also very important. Minimum overlay thicknesses are applied for specific problems, such as polishing or wear in the wheelpaths. A more detailed thickness design procedure is necessary when resurfacing is intended to increase the structural capacity of the existing pavement structure.

As discussed in module 3-10, engineers are encouraged to use pavement survey and evaluation results in the overlay design process. However, the design of HMA overlays is an involved topic and is only introduced in this text. Procedures for selecting the overlay type best suited for a particular situation and the limiting criteria associated with the use of specific types of overlays are discussed in module 4-15. Additional information on the design of HMA overlays for existing rigid pavements is available from other sources as well.[1,2,3]

3. DEFINITIONS

In this discussion, HMA overlays refer to hot-mixed bituminous mixtures placed on an existing rigid pavement for functional or structural improvements. HMA is used to indicate high-quality bituminous mixtures produced in a facility that proportions, blends, and heats aggregate and asphalt to produce a material meeting the requirements of the project. Other bituminous mixtures (e.g., chip seals) could be used to address certain functional deficiencies, but HMA materials are more versatile and are the material of choice for addressing structural deficiencies.

The development of either functional or structural deficiencies in a rigid pavement are cause for consideration of the placement of an HMA overlay. For definition purposes, functional deficiencies in a pavement refer to its inability to provide a safe and comfortable ride to the traveling public. Examples of functional deficiencies in an existing rigid pavement include:

- Polishing of the surface in the wheelpaths, resulting in decreased surface friction.
- Roughness resulting from nonload-associated distress, such as joint spalling.
- Inadequate cross slope that results in poor surface drainage.
- Excessive noise levels due to the surface texture.

Structural deficiencies refer to the pavement's inadequacy to sustain current or future traffic loadings. Examples of structural deficiencies (distresses) in rigid pavements include:

- Deteriorated reflection cracking, particularly transverse cracking over joints and existing cracks, in HMA overlays of rigid pavements.
- Corner breaks, transverse working cracks, shattered slabs, and patches of these distresses in jointed concrete pavements (JCP).
- Punchouts, and patches of punchouts in continuously reinforced concrete pavements (CRCP).

4. PURPOSE AND APPLICATIONS

As mentioned above, HMA overlays can be used to improve the functional or structural performance of a pavement. Many functional deficiencies can be partially or totally corrected with a nonstructural overlay, such as a thin HMA overlay. Several European countries even use surface treatments on rigid pavements to address functional deficiencies.[4] While the focus of this module is on structural overlays, it is important to recognize that nonstructural overlays are often appropriate repair alternatives. A thin overlay can be successfully used to rehabilitate a pavement exhibiting the functional deficiencies listed above. However, they should not be considered for a pavement that is structurally deficient.

A pavement is structurally deficient if it is experiencing structural distress, when nondestructive testing (NDT) indicates that a deficiency exists, or when future traffic loadings are expected to exceed design levels. The presence of structural distresses in significant amounts indicates that the structural capacity of the pavement has been approached or exceeded. An overlay that is placed to correct structural deficiencies must address the cause of the structural deterioration of the pavement and be able to provide sufficient strength to resist further deterioration resulting from future traffic and environmental loadings.

Detailed distress definitions and illustrations useful in determining the condition of a pavement are provided by SHRP[5] and are presented in module 2-3. Where extensive distresses exist, a structural evaluation of the pavement should be conducted to determine the extent of the structural deficiency (see module 2-6).

5. HMA OVERLAYS

The most common type of overlay is constructed with dense-graded, hot-mixed asphalt (HMA) concrete, although the type of mixes for binder or surface layers vary from agency to agency. Depending upon the problem in the existing pavement and on the purpose of the overlay, the thickness of HMA overlays may vary considerably. For example, HMA overlays placed to address functional deficiencies are typically minimum thickness overlays; that is, they are constructed to the minimum thickness that is practical given equipment limitations, coarse aggregate size, and projected traffic loadings. Minimum thickness overlays generally range from about 25 to 100 mm.

HMA overlays placed to address structural deficiencies are typically thicker than functional overlays, but will vary depending on the condition of the existing pavement and on the projected traffic volumes. Structural HMA overlay thicknesses are the primary output of most overlay design methodologies, and typically range from about 75 to 200 mm for most highway applications.

6. CONSIDERATIONS IN OVERLAY SELECTION

A thorough examination of pavement deficiencies and the cause of deterioration must be made prior to selecting an overlay as a candidate repair alternative. There is a point where the extent of pavement deterioration could make reconstruction more cost-effective than placing a thick overlay. For example, poor drainage and poor base or subbase support cannot be corrected through the placement of an overlay alone. Also, reconstruction may be the desired alternative when there are serious concrete durability problems. Figure 4-14.1 shows a conceptual relationship between a pavement's condition and the optimum rehabilitation need. In general, HMA overlays fall in the middle range of options; they are thought of as appropriate after restoration techniques have been tried, but before unbonded concrete overlays and reconstruction. Alternatively, HMA overlays have been successfully used on baldy deteriorated rigid pavements that are first fractured or rubblized.[3]

Once an overlay has been identified as a candidate alternative, an examination of the overlay types is made to determine which are feasible for a particular situation. This determination is based on the consideration of construction feasibility, desired performance period, and available funding, as summarized in table 4-14.1.[6] Each of these critical aspects is described below.

Construction Feasibility

Traffic Control

Traffic control delays with HMA overlay construction are shorter than most others, since the overlay can easily be constructed one lane at a time, and the project can be opened to traffic much sooner than portland cement concrete (PCC) overlays or other repairs. On roads with high traffic volumes, the ability to minimize closures and the attendant disruptions translate into considerable cost savings and a reduction in user delays. However, comparisons between alternate designs should be made on a life-cycle cost basis. Although a given alternative may have a lower initial cost and less closure time, it may still not be the best alternative when all costs and delays throughout the analysis period are considered.

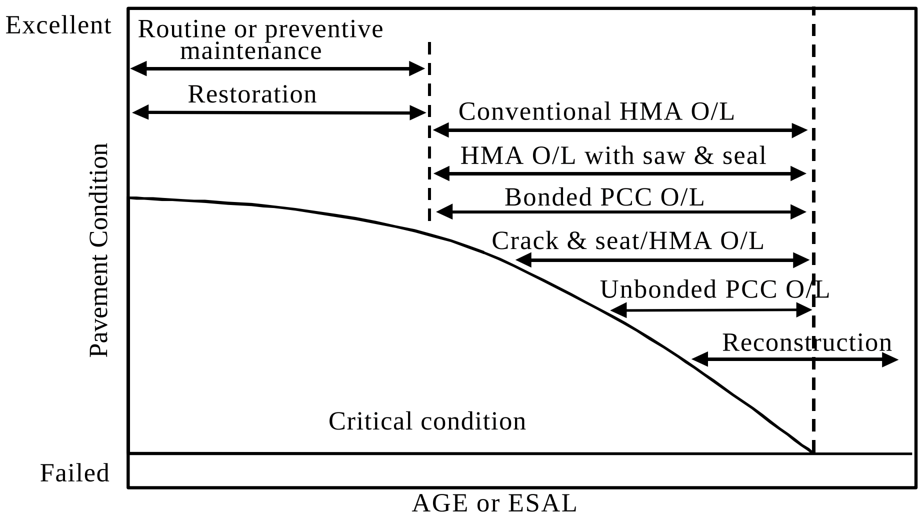


Figure 4-14.l. Conceptual relationship for selecting rehabilitation alternatives.

Constructibility

Conventional HMA overlays are a common rehabilitation procedure and therefore require no special construction considerations. Key issues regarding their constructibility are discussed under the various reflection crack control headings.

Overhead Vertical Clearances and Other Surface Elevation Change Problems

Any overlay has the effect of decreasing overhead clearances by adding additional thickness to the surface. Where there is the possibility for clearance problems, this must be considered during initial design and steps must be taken to provide adequate clearance. This can be accomplished by using thinner overlays under structures (undesirable), or by reconstructing the pavement directly under the structure with tapers to the overlaid sections on either side. In rare instances, it may be more cost effective to raise the structures. If many overhead bridges exist along the project, a thick overlay may result in prohibitively high costs.

Overlaying traffic lanes also necessitates overlaying shoulders, which may be in good condition and not in need of an overlay. Overlays may also require the raising of guard rails and the placement of additional fill material adjacent to shoulders. Overlays also decrease curb height and could disturb drainage patterns. One innovative method that has been used in this situation is an inlay, in which the existing pavement (but not the shoulders) is milled or removed and a new HMA overlay is placed between the shoulders.

Table 4-14.1. Feasibility guidelines for HMA overlays.[6]

	HMA over PCC	**HMA over Rubblized PCC**	**HMA over Cracked and Seated PCC (JPCP only)[1]**	**HMA over PCC with Sawed and Sealed Joints**
CONSTRUCTIBILITY				
Vertical Clearance	Required thickness may pose a problem.	Required thickness usually a problem.	Required thickness usually a problem.	Required thickness may pose a problem.
Traffic Control	Not difficult to construct under traffic. Can be opened to traffic quickly.	Not difficult to construct under traffic. Can be opened to traffic quickly. Fracturing operations require additional time.	Not difficult to construct under traffic. Can be opened to traffic quickly. Fracturing operations require additional time.	Not difficult to construct under traffic. Can be opened to traffic quickly. Joint sawing requires additional time.
Construction	Common rehabilitation procedure. HMA mix design and density critical.	Special equipment recommended for rubblizing. HMA mix design and density critical.	Fairly difficult to fracture existing slab sufficiently. HMA mix design and density critical.	Common rehabilitation procedure, except for joints, which must be sawed very accurately. HMA mix design and density critical.
PERFORMANCE PERIOD				
Existing Condition	The more deterioration present, the thicker the overlay for a given performance period.	Can be applied to more deteriorated PCC pavements.	Can be applied to more deteriorated PCC pavements.	The more deterioration present, the thicker the overlay required for a given performance period.
Extent of Repair	Must repair deteriorated cracks and joints and restore load transfer.	Very little repair is needed.	Must repair badly deteriorated cracks and joints (with HMA).	Must repair deteriorated cracks and joints and restore load transfer.
Structural Adequacy	Thickness can be increased to provide structural adequacy, but may be substantial.	Thickness may be substantial, since it is essentially a reconstructed HMA surface on a granular base.	Reduction in structural integrity from slight to substantial. Thickness can be increased to provide structural adequacy.	Thickness can be increased to provide structural adequacy, but may be substantial.

Note 1: For JRCP reinforcement must either be broken or yielded to provide any crack relief benefit. Therefore, rubblizing is recommended for badly deteriorated JRCP.

Table 4-14.1. Feasibility guidelines for HMA overlays.[6] (continued)

	HMA over PCC	HMA over Rubblized PCC	HMA over Cracked and Seated PCC (JPCP only)[1]	HMA over PCC with Sawed and Sealed Joints
Future Traffic	High traffic level may result in permanent deformation.	High traffic level may result in permanent deformation.	High traffic level may result in permanent deformation.	High traffic level may result in permanent deformation.
Reliability	Fair (reflection cracking and permanent deformation may be a problem.	Fair to good (permanent deformation may be a problem).	Fair to poor (reflection cracking and permanent deformation may be a problem).	Good (permanent deformation may be a problem).
COST-EFFECTIVENESS				
Initial Cost	Depends greatly on preoverlay repair.	Higher than HMA overlay due to rubblizing and thicker overlay required.	Higher than HMA overlay due to cracking and seating and thicker overlay required.	Somewhat higher than conventional HMA overlay due to joint sawing. Depends greatly on preoverlay repair.
Life-Cycle Cost	Competitive, if future life is substantial.	Not competitive unless preoverlay costs can be reduced to offset the cost of rubblizing and thicker HMA.	Not competitive unless preoverlay costs can be reduced to offset the cost of cracking and seating and thicker HMA.	Competitive if future life is substantially greater than conventional HMA overlay.

Note 1: For JRCP reinforcement must either be broken or yielded to provide any crack relief benefit. Therefore, rubblizing is recommended for badly deteriorated JRCP.

Performance Period

The AASHTO Design Guide[1] defines a rehabilitation's performance period as the amount of time that a rehabilitation will last before the pavement will again require some type of rehabilitation. Due to the difficulty, hazards, and costs involved with closing traffic lanes, some minimum life must be attainable with the proposed rehabilitation. The life of an HMA overlay depends upon many factors:

- Condition of the underlying rigid pavement.
- Structural adequacy.
- Material deterioration within the pavement structure.
- Climate (temperature, moisture, freeze-thaw cycling).
- Subdrainage adequacy.
- Presence of swelling soils.

The dominant factor that affects the performance of HMA overlays of existing rigid pavements is reflection cracking, discussed later. Where reflection cracking is not a problem, performance will be controlled by permanent deformation and aging. There is no point in attempting to specify a typical performance period for HMA overlays because it is dependent on so many factors. Even when such information is available in an agency's pavement management system (PMS), any projection of performance should be based on actual or anticipated site conditions and not some average results.

Permanent Deformation

Permanent deformation (rutting) may occur in HMA overlays, sometimes within the first few years of overlay construction. This has become a growing concern in many highway agencies, as they move toward thicker HMA overlays. In addition to construction concerns related to the thicker overlays, this increase in rutting has been attributed to increases in axle loads, traffic volumes, and tire pressures. Improvements in mix design and construction are necessary to minimize rutting. Critical factors to consider to reduce rutting include, among others, the angularity of the coarse and fine aggregate, the rut resistance of the mix design, the proposed field compaction efforts, and the asphalt content, gradation, and volumetric properties.

Several States have experimented with stone matrix asphalt (SMA), which is purported to have increased rutting resistance due to a greater percentage of quality coarse aggregate in the mix that ensures stone-on-stone contact. SMA mixtures have a high proportion of asphalt, mineral filler, fine aggregate, and a stabilizing agent to prevent asphalt cement drainage. In addition, SMA has been reported to provide high wear resistance and good durability.

7. PREOVERLAY TREATMENT AND REPAIR

A certain amount of preoverlay treatment and repair should be performed as part of an HMA structural overlay design. This is to repair deteriorated and weakened areas in the existing rigid pavement that would otherwise cause premature failure of the overlay.

As noted in module 3-10, there is a trade off between the amount of preoverlay treatment and the required structural thickness (and hence cost) of the overlay. That is, as more preoverlay treatments are performed, the thickness of the overlay is reduced. However, this reduction generally levels off after some level of preoverlay repair per treatment has been reached. On the other hand, it is generally not feasible to construct thicker HMA overlays without performing any preoverlay repairs/treatments due to increased costs and associated problems with thick overlays (e.g., bridge clearances, rutting potential).

Preoverlay techniques available for use on existing rigid pavements include full-depth repair, undersealing, load transfer restoration, and retrofitted subdrainage. The appropriateness of these preoverlay treatments is described below, with the exception of full-depth repairs which is discussed later in the reflection cracking section of this module.

Restoring Support Under Rigid Pavements

When loss of support exists beneath the corners or joints of rigid pavement slabs, the corner deflections will be higher than normal. This can lead to the rapid formation and progression of reflective cracks in HMA overlays. Loss of support is usually associated with excess water and erosion problems.

On pavements exhibiting loss of support, removal of water by pavement drainage and restoration of support through slab stabilization (see module 4-7) should be considered. Overlays by themselves do little to address the problem of loss of support.

Restoring Load Transfer Across Transverse Joints and Cracks

Many transverse joints and cracks lose their ability to transfer loads from one side of the joint or crack to the other. This results in the development of a large differential deflection between each side, such that only a small portion of the applied load is transferred to the other side. This loss of load transfer in an existing pavement will lead to the deterioration of cracks in an HMA overlay or bonded concrete overlay. Two solutions to this problem exist. One is a full-depth repair of the joint or crack, and the provision of doweled transverse joints (see module 4-5). The other is to restore load transfer across the joint or crack using dowels grouted into slots (see module 4-9).

Drainage Corrections

One factor that should always be addressed in overlay design is drainage, both of the surface and of the subsurface. If poor drainage conditions are contributing to the deterioration of the existing pavement, adding an overlay will not correct the problem. Most overlay design procedures assume that the existing pavement has good drainage. It is always advisable to conduct a thorough drainage survey, identify drainage-related distresses, and develop solutions that address these distresses as part of the overlay design process. Improving poor subdrainage conditions will have a beneficial effect on the performance of the overlay. Removal of excess water from the pavement cross section will reduce erosion and increase the strength of the base and subgrade, which in turn will reduce deflections. In addition, stripping in the HMA overlay and D-cracking in the underlying rigid pavement may be slowed by improved drainage.

8. HMA OVER PCC OVERLAY DESIGN

There is no universally accepted overlay design procedure, but three major approaches to overlay design are discussed below. Overlay thickness design procedures either assume that adequate preoverlay repair and reflective crack control actions are taken, or they permit different levels of repair to be considered. Fractured slab techniques can also be employed on rigid pavements in poor condition.[1,3] If preoverlay and reflective crack control treatments are not adequate, then the overlay will likely fail prematurely and not provide the desired structural service. Each agency is encouraged to calibrate or modify overlay design procedures to suit local conditions.

Engineering Judgment

A number of agencies rely on the judgment and experience of their engineers in determining overlay thicknesses. Some agencies have monitored the performance of previous overlays and have an approximate estimate of how selected standard overlays will perform. This is particularly true for HMA overlays of rigid pavements. An example of an application of engineering judgment is the specification of a standard overlay, such as 50 mm HMA overlays for certain classes of roads, 76 mm HMA overlays for other classes, and so on. These overlays may be applied at a certain age or when specific distresses start to appear. There are obvious deficiencies to this approach, because very few engineers have adequate experience to determine the required overlay thickness for a given traffic and design life. The development of an overlay design procedure that quantitatively considers the important design factors is strongly recommended. However, engineering judgment is often used when designing overlays to correct functional deficiencies.

Structural Deficiency

The basic concept of the structural deficiency overlay design approach is that an overlay is required that is equal to the difference between the structural capacity of the existing pavement and the structural capacity of a new pavement designed for the prevailing conditions. The difference in structure represents the theoretical structural deficiency that must be provided by the overlay. If both the existing pavement and the overlay pavement are made of the same material, the difference in structural capacity represents the required thickness of concrete or asphalt directly. However, with an HMA overlay equivalency factors must be used to convert different materials into one material, or the American Association of State Highway and Transportation Officials (AASHTO) structural number approach must be used.

The structural deficiency approach is by far the most common approach for the design of overlays. This procedure involves evaluating the existing or effective structural capacity of the pavement (using visual surveys and materials testing, nondestructive testing, or remaining life procedures) and subtracting it from the required structural capacity of the "new" pavement (see figure 4-14.4). The reliability of the design is based on the ability to accurately determine the effective structural capacity of the existing pavement.

Mechanistic Fatigue Damage Approach

The basic concept of the mechanistic fatigue damage approach to overlay design is that the design overlay thickness is one that will limit fatigue damage in the overlay or the existing pavement to an acceptable level over the design period. This is discussed in greater detail in module 3-10. An existing pavement and overlay may be modeled using elastic layer theory, plate theory, or finite element analysis, as appropriate for the pavement type and overlay type. The amount of fatigue damage expected in the overlay depends primarily on the trial overlay thickness and the number of expected load repetitions. The total fatigue damage expected in the existing pavement not only depends on those factors, but also on the past fatigue damage accumulated prior to the overlay. Whether fatigue damage in the overlay or in the existing pavement should control the design depends on the type of overlay and type of existing pavement.

Mechanistic fatigue damage procedures for overlay design have been developed in several research studies. However, these procedures are not generally applicable for HMA overlays of rigid pavements, as the primary load-related failures are due to either rutting or reflection cracking. Seeds et al.[7] developed a mechanistic-based approach that considers the potential for reflection cracking to develop due to horizontal and vertical differential movement. The accumulated fatigue damage in the HMA may be due as much to environmental loadings as to vehicles causing vertical movement. However, this type of mechanistic approach is what is required for HMA overlays of rigid pavements.

9. FACTORS INFLUENCING CRACK PROPAGATION

Reflection cracks appear on the surface of an overlay above joints or cracks in the underlying pavement layer. Reflection cracking occurs in nearly all types of overlays, but is most often a problem in HMA overlays of HMA pavements with thermal cracks, and in HMA overlays of jointed concrete pavements. Although attempts have been made to develop a comprehensive design procedure that addresses the problem of reflection cracking, to date no procedure has received widespread acceptance.[7,8]

Reflection cracking is a result of horizontal and vertical movements at the joints and cracks in the underlying pavement that create high stress concentrations in the overlay. These movements at joints and cracks are caused by a combination of the following:[9]

- Low temperature.
- Traffic loads.

Low temperatures cause the underlying pavement to contract, increasing joint and crack openings. This horizontal movement in the base pavement creates tensile stress in an overlay, as illustrated in figure 4-14.1. In addition, the overlay is subjected to further tensile stress as the overlay material itself also contracts in response to low temperatures, shown in figure 4-14.2.

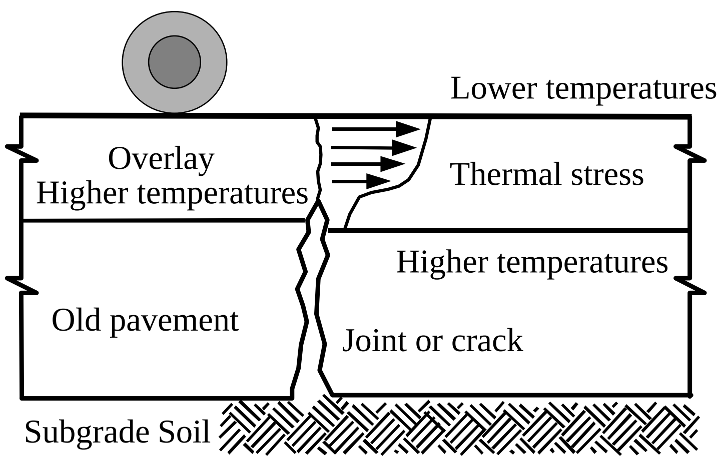


Figure 4-14.2. Stresses in overlay caused by low temperatures.

Traffic loadings produce a completely different type of movement, shown in figure 4-14.3. Differential vertical deflection created by traffic passing over a joint or working crack creates shearing and bending stress in the overlay. Three distinct load pulses are produced by a moving wheel load:[10]

> *As the wheel load approaches the crack, the shear stress in the overlay above the crack will reach a maximum illustrated as point A . . . When the wheel is directly above the crack, the maximum bending stress will occur as illustrated by point B . . . As the wheel load crosses the crack, a second maximum shear stress in the reverse direction will occur as illustrated by point C . . .*

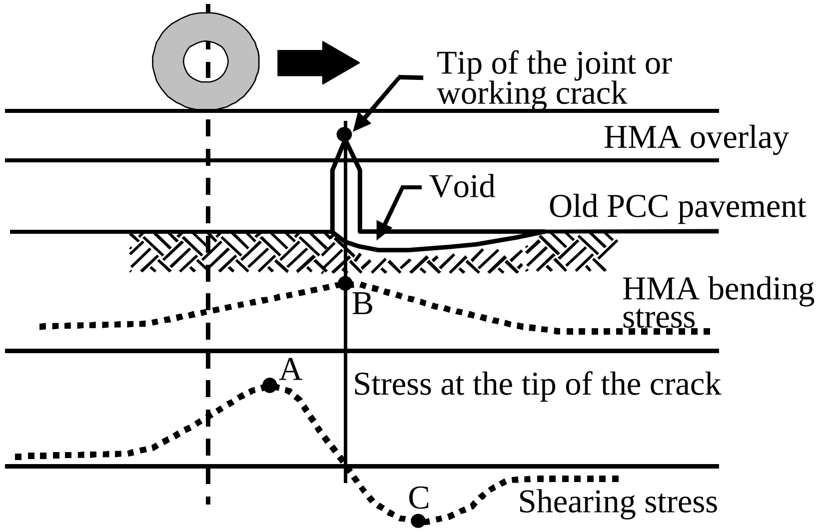


Figure 4-14.3. Shearing and bending stress in an HMA overlay created by a moving load.[10]

Stresses from traffic loadings occur far more often than temperature stresses, but are typically much smaller in magnitude except where poor load transfer exists. The poorer the load transfer, the more rapidly the crack will reflect through the overlay, and the more severe the deterioration will become. It is critical that the expected deflections before and after overlays be evaluated so that the most cost-effective rehabilitation or reconstruction method may be selected.

As an example of the effect of load transfer on reflection cracking, consider the data measured on a JRCP with a 100-mm HMA overlay, shown in table 4-14.2. The load transfer measurements were made with an FWD at a 40-kN load at over 50 transverse reflection cracks (caused by both underlying cracks and joints) after the overlay had been in service for about 8 years under heavy traffic.[11] These results show that poor load transfer across the underlying crack or joint results in deterioration of reflected cracks in the HMA overlay.

For both temperature- and load-induced reflection cracking, crack initiation usually begins at the bottom of the overlay. Each temperature cycle and traffic load causes damage that contributes to the propagation of the reflection crack further up into the overlay. The different types of horizontal and vertical movements do not propagate the crack equally, and the number of movements is not in itself a criterion for predicting the rate of reflection cracking. While further studies into the influence of each type of load damage are needed, much is already known today that can be applied to this problem.

Table 4-14.2. Relation between existing pavement discontinuity, reflected crack, and measured load transfer.[11]

Underlying Location	HMA Reflected Crack/Joint Severity*	Measured Mean Load Transfer**
Cracks	Low	77%
Joints	Low	79%
Cracks	Medium/High	31%
Joints	Medium/High	52%

* Low is non-spalled, narrow crack; Medium/High is spalled and deteriorated crack.
** (Unloaded side/loaded side deflection), measured on top of the HMA overlay.

The key to eliminating reflection cracking is to eliminate the deformations and stresses produced in the overlay at existing joints and cracks. However, it is highly unlikely that these deformations and stresses can ever be completely eliminated; the most that can be achieved is a reduction in the rate of appearance and the severity of the reflection cracking. This is the primary reason that repairs prior to overlaying are encouraged and various preoverlay treatments are used.

The results of different treatments used around the United States to reduce reflection cracking are compiled in the NEEP-10 study reports.[12] Taken as a whole, the results of the treatments are inconclusive. Some individual case studies have shown excellent results, whereas others have shown poor results. The main reason for this great disparity is the inability of the performing agency to adequately define the horizontal and vertical deformations that existed in the pavement prior to and following the overlay.

10. DESIGN ISSUES RELATED TO REFLECTION CRACKING

Reflection cracking leads to increased infiltration of surface water into the pavement system, which in turn weakens the supporting layers. Over time, reflection cracks will also deteriorate and spall, decreasing the pavement's serviceability. Design issues regarding reflection cracking consider the following factors:

- The rate of reflection cracking through the overlay.
- The amount and rate of deterioration of the cracks after cracking occurs.
- The amount of water that can infiltrate through the cracks.

While each of these is important, the second factor is the most cost-effective to address. If the crack severity can be limited, sealing the crack is easier and the contribution of crack deterioration to roughness and potholes is greatly reduced.

11. APPROACHES TO REDUCE THE SEVERITY OF REFLECTION CRACKING

A number of techniques have been tried in order to reduce the rate or severity of reflection cracking. They can be grouped into the following categories:

- Fabrics.
- Stress-relieving interlayers.
- Crack-arresting interlayers.
- Preoverlay treatments.
- Slab repair and replacement.
- Reflection crack severity control.
- Increased overlay thickness.

The success of these treatments lies, in part, on how their performance is measured. Some believe that any attempt to mitigate reflection cracking is successful only if the cracks do not appear at all. The other way to look at such treatments is not whether they allow the crack to appear, but also:

- Do they retard the occurrence of the reflection crack?
- Do they control the severity of the crack once it occurs?
- Do they provide other benefits, such as reducing the overlay thickness or enhancing the waterproofing capabilities of the pavement?

The latter, less constrictive definition should probably be applied to measuring the effectiveness of approaches used to reduce the severity of reflection cracking, providing that such measures can also be shown to be cost-effective.

Fabrics

Woven or non-woven synthetic fabrics made of polypropylene, polyester, fiberglass, nylon, or combinations of these materials, have been used as reinforcing layers in HMA overlays. Common examples of these fabrics are Petromat, Bidim, Typar, Reepav, Mirafi, Amopave, and Trevira. The purpose of a fabric is to physically resist the formation of the cracks in the overlay as the cracks and joints in the underlying pavement open.

Most typical fabric treatments are placed directly on the pavement, following the application of a tack coat to the existing pavement surface.[13,14] After the fabric is rolled or brushed into the tack coat, the overlay is placed on top of the fabric. However, the best results have been achieved when the fabric is placed within the overlay, which requires two lifts of HMA and a minimum overlay thickness of about 76mm.[15,16] This concept is illustrated in figure 4-14.4. The surface of the leveling course should be unblemished, reducing the potential for wrinkling the fabric. A light application of tack coat should be applied to both the leveling course and on the fabric once it is placed. The surface course should then be placed directly on the fabric and compacted.

The ability of fabrics to prevent reflection cracking has been varied. Although some reduction in the initial onset of reflection cracking is commonly reported, the cost-effectiveness of the treatment has been questioned where substantial horizontal or differential vertical movement at the joint or crack is encountered. See references 13, 17, 18, 19, and 20. Predoehl also reported that fabrics could provide reflection crack control equivalent to approximately 30.5 mm of additional HMA (or a delay of a little over 1 year), but did not prevent it from occurring.[20] Fabrics have been found to be more effective at preventing reflection cracking at longitudinal joints, where smaller horizontal movements are recorded, and in warm and mild climates.

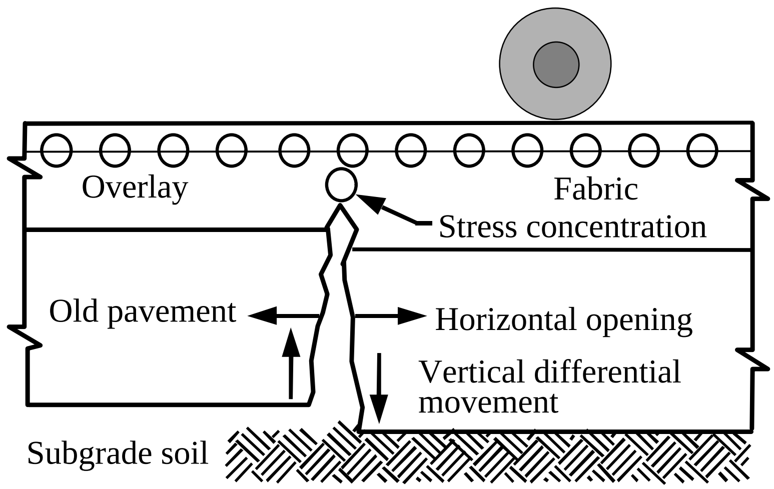


Figure 4-14.4. Proper placement of a geotextile fabric.

The effectiveness of fabrics has commonly been related to the magnitude of differential vertical movement at the joint or crack. California reports that fabrics are effective if the differential vertical movement at a rigid pavement joint is between 0.0762 mm and 0.203 mm.[20] It is believed that fabrics are ineffective if the differential vertical deflection is greater than 0.203 mm, and are unnecessary if it is less than 0.0762 mm. The results of a study conducted in Virginia are presented in table 4-14.3. This study showed a dramatic increase in the rate of reflection cracking propagation as a result of increasing differential vertical deflections.[21] The Asphalt Institute also recommends limiting the differential vertical deflection at the joints to 0.0508 mm to achieve good performance from an overlay if no treatment is used.[2]

Table 4-14.3. Influence of differential vertical deflections on rate of reflection cracking.[21]

Differential Vertical Deflection, mm	Percent of Joints Cracked	
	With Fabric	Control
0	0	44
0.050	29	54
0.100	88	74
0.150	88	100
0.200	100	100

Where existing joints or cracks experience large horizontal or differential vertical movement, cracks tend to reflect through the fabric, eliminating its effectiveness. If the fabric does not rupture, some degree of waterproofing will be provided even if the overlay cracks, although the long-term performance has not been substantiated.

Another consideration when using fabric within HMA layers is that future recycling projects may not be practical due to the presence of the fabric. The effects of fabrics in recycled asphalt pavement (RAP) is an area that needs to be studied further as projects containing fabrics begin to need rehabilitation.

Stress-Relieving Interlayer

A stress-relieving interlayer dissipates movements and stresses developed at the joint or crack before they create stresses in the overlay. These installations generally include a rubber- or polymer-modified asphalt as the stress-relieving material, and can be constructed directly on the original pavement surface, or applied through the use of a proprietary material.

Constructing a stress-relieving interlayer involves a spray application of rubber-asphalt over the existing joints and cracks, followed by the placement and seating of aggregate chips. This type of treatment is commonly referred to as a stress-absorbing membrane interlayer (SAMI). A SAMI, illustrated in figure 4-14.5, may be up to 13 mm thick. These have worked effectively in reducing the rate and severity of reflection cracking in overlays of HMA pavements with fatigue cracking; however, they have been ineffective on the linear or opening mode of cracking found in rigid pavements, or where movements have been very large, such as on lane widenings.[22]

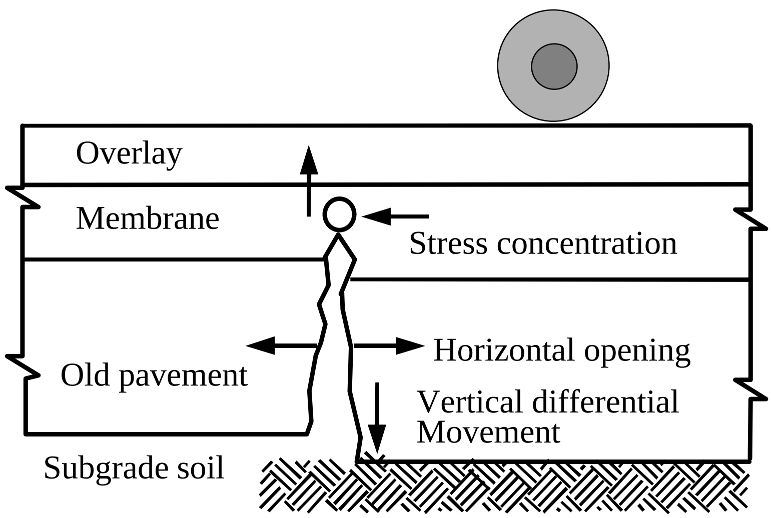


Figure 4-14.5. Illustration of the placement of a stress-absorbing (membrane) interlayer.

Proprietary stress-relieving interlayers are available that consist of one or two fabrics and a modified asphalt cement layer. Some of these products use a dense mastic asphaltic binder to absorb the stress. PavePrep, Bituthene, Petrotac, Polyguard 665, and Roadglas are examples of proprietary stress-relieving interlayers that have shown promise in reducing the rate of reflection cracking. Most of these products are “Band-Aid” treatments that are applied directly over the joint or crack, and not over the entire pavement surface.

Most of the proprietary stress-relieving interlayers are applied to the existing pavement using a modified asphalt cement tack coat. Several States have had success with the Band-Aid treatments on both longitudinal and transverse joints and cracks; however, the cost-effectiveness of this approach has not yet been established.[23,24,25]

Table 4-14.4 summarizes the results of Pennsylvania Department of Transportation Research Project 79-6.[26] This study reports that the application of proprietary stress-relieving interlayers showed promise in the reduction in the rate of reflection crack propagation. But in contrast to the Pennsylvania results, research conducted in Arizona using PavePrep, Glassgrid, and Tapecoat found no statistical difference in the occurrence of reflection cracking compared to a control section, with the majority of cracks reflecting through within 6 months of placement of the treatment.[25]

Table 4-14.4. Reduction of reflective cracking in HMA overlays through the use of heavy-duty membranes.[26]

Material	Total Length, (meters)		Total Reflective Cracks	
	Transverse	**Longitudinal**	**Transverse**	**Longitudinal**
Polyguard 665	53.6	237.7	62 (35.0%)	2.5 (0.3%)
Royston #108	6.1	92.7	18 (90.0%)	0 (0%)
Royston #10AR	16.8	22.9	19.5 (35.5%)	0 (0%)
PavePrep	40.2	160.0	7 (5.3%)	9 (1.7%)
Roadglas	59.7	304.8	57.5 (29.3)	0 (0%)
Bituthene H.D.	56.7	98.8	93.5 (50.3%)	0 (0%)
Petrotac	61.0	180.4	59.5 (29.8%)	0 (0%)

Figure 4-14.6 presents the results from a study performed in Colorado that included four paving fabrics, a nonwoven glass mat, and a heavy-duty membrane (Bituthene).[11] The original pavement consisted of a narrow two-lane roadway that was widened to four lanes; block cracking was present over much of the surface of the old pavement. All of the materials were placed directly on top of an HMA leveling course with the exception of the Bituthene, which was placed directly over the cracks prior to placement of the leveling course. A 90-mm HMA overlay was placed on top of the leveling course. After 4 years, the membrane performed better than the control (standard) treatment, as did two of the other treatments. The results also show increased reflection of transverse cracks compared to longitudinal cracks.

The stress-relieving interlayers that include rubber asphalt membranes are very effective waterproofers.[23] They have worked best on jointed pavements with firm foundations. Fiberglass installations have been effective in reducing reflection cracking caused by pavement widening.[27]

The variability in results obtained from different research efforts indicates that stress-absorbing interlayers are not a universally applicable solution to reflection crack prevention, and their high cost requires that they provide greatly improved performance, which has not been demonstrated.

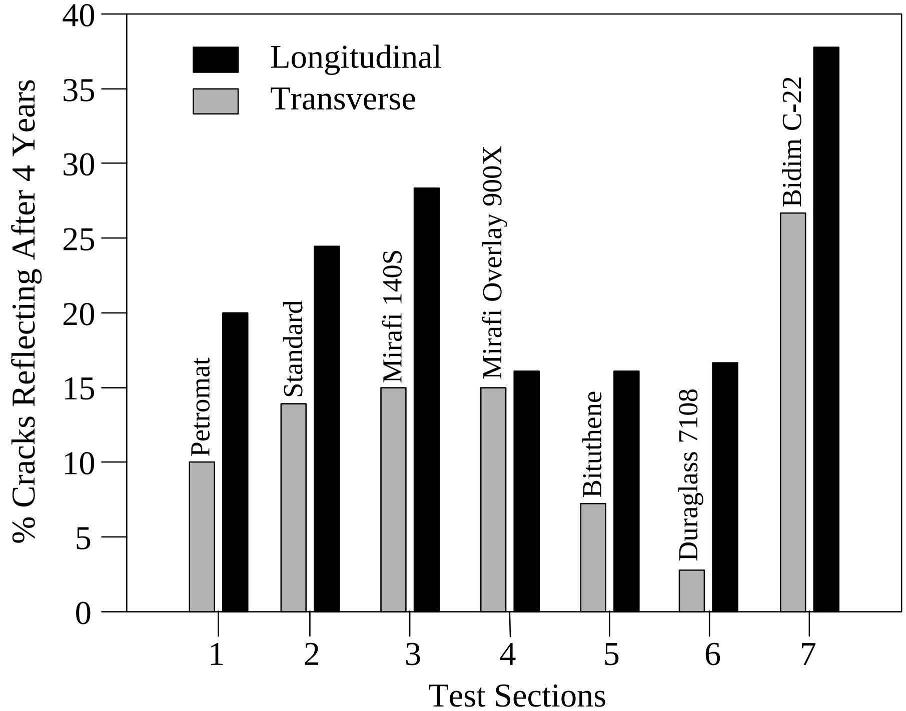


Figure 4-14.6. Effect of various fabrics and membranes on reflective block cracking.[11]

Crack-Arresting Interlayers

Crack-arresting interlayers are granular layers that arrest the development of reflection cracks by providing large void spaces that effectively blunt crack propagation, as illustrated in figure 4-14.7. Standard granular bases with low fines contents and large aggregates have been used for this purpose. Arkansas and Tennessee have pioneered a bituminous-stabilized base using large (90-mm top size) aggregate with up to 25 percent voids. The Asphalt Institute recommends two different aggregate gradations for interlayers, one with a maximum aggregate size of 75 mm and the other with a maximum aggregate size of 50 mm.[28] Special construction procedures must be observed because of the use of the one-sized aggregate. Sand/asphalt layers have also been tried with limited success.

Crack-arresting interlayers have been effective when installed properly. Their effectiveness is marginal when improperly constructed, because of the resultant instability of the mix and subsequent rutting problems. Because of the large top size of the aggregate, the crack relief interlayer usually has a minimum thickness of 90 mm, resulting in a total overlay thickness of 125 to 230 mm.[28] This can limit the use of this type of procedure to areas where bridge clearance and shoulder elevation problems are not encountered.

Preoverlay Treatments

Any form of repair or reduction in effective slab size that reduces movement at joints and cracks can potentially reduce reflection cracking. A number of techniques are used prior to overlaying to reduce this movement.

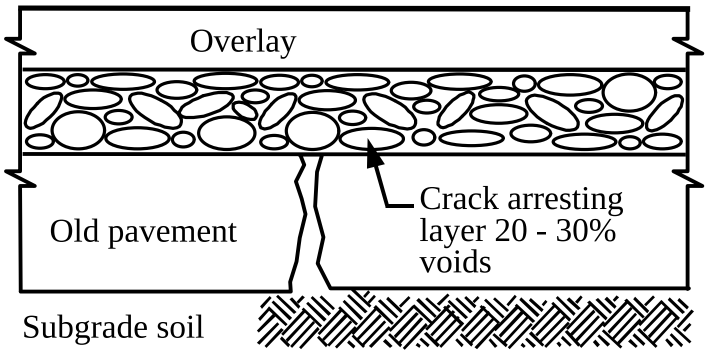


Figure 4-14.7. Cross section of a typical crack-arresting interlayer.

Slab Stabilization

Slab stabilization is performed to fill voids that may exist beneath slab corners. As a result, differential vertical deflections across a joint or crack are reduced, which in turn reduces the progressive deterioration of the reflection crack. Care must be taken to ensure that the grouting operation reduces the deflections. If proper construction procedures are not followed, it is possible to lift the slab by pumping too much grout, which will create a void elsewhere in the slab and lead to increased deflections. If this happens, the reflection cracking can be worse than if nothing had been done. The procedures for slab stabilization are discussed in detail in module 4-7.

Fractured Slab Rehabilitation Techniques

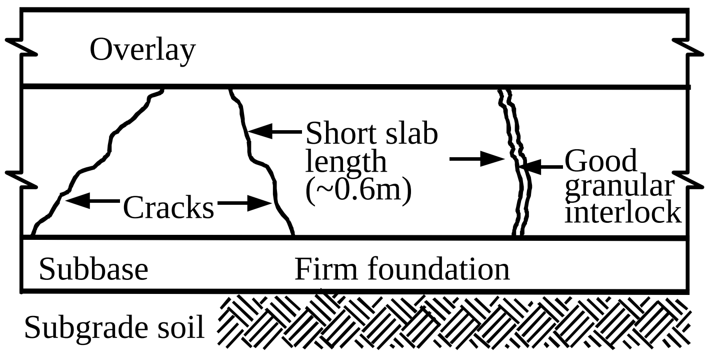
Three specific techniques are grouped together under the heading of fractured slab techniques: cracking and seating, breaking and seating, and rubblizing. Although these terms have been used loosely and somewhat interchangeably in the past, there is general agreement on the following definitions:(1)

- Cracking and seating is performed on jointed plain concrete pavements (JPCP) to reduce the effective slab length and reduce slab movement.
- Breaking and seating is conducted on JRCP, again to shorten the slab length and to reduce slab movement. However, greater impact energy is required to rupture the steel in the slab.
- Rubblizing is the fracturing of the slab into extremely small pieces; it is generally performed on badly deteriorated rigid pavements that have little or no remaining structural life.

In most cases where fractured slab rehabilitation techniques are used, some consideration should be given to whether or not edge drains will be needed. The methods of cracking, breaking, and rubblizing create substantial amounts of cracking in the rigid layers and allow water to pass more freely into the underlying pavement layers. The use of edge drains may be necessary to drain the pavement effectively, and should be considered when contemplating a fractured slab technique. However, they should be placed after the fracturing of the slab to prevent silting. Where edge drains are already in place and still functioning, fractured slab techniques may not be appropriate.

Cracking and Seating

Cracking and seating reduces joint and crack movement by shortening the effective slab length and seating the broken concrete pieces into the supporting layer. Although cracking and seating has been used for a number of years, it has recently become a standard practice in a number of States and many projects have been completed in the past 10 years. See references 3, 29, 30, and 31. As shown in figure 4-14.8, the JPCP slabs are cracked into small segments before overlaying. The intent of cracking and seating is to create concrete pieces that are small enough to reduce the movement caused by the low temperature thermal stresses and the cyclic, daily temperature stresses, while still being large enough to maintain structural stability.[3,32] Seating of the broken slabs after cracking is believed to be necessary to re-establish firm support between the subbase and the slab, thereby limiting differential vertical movement under traffic.



1 ft = .3048 m

Figure 4-14.8. Illustration of a crack and seat pavement.

One important consideration when planning a crack and seat project is the quality of subgrade materials. In situations where the support of the underlying areas is weak or questionable, a determination should be made as to whether the cracked slabs will be adequately supported, or whether the pieces will “float” on a soft subgrade layer. Floating slabs contribute to poorer performance than if nothing had been done. While sampling and testing of the subgrade support material should be performed whenever possible, it is especially important in situations where questionable subgrade support may exist.

There are several types of distress that occur in JPCP that may, when they exist in substantial quantities, justify cracking and seating of the concrete slab. Some of these distresses are listed below:[32]

- Seriously faulted transverse joints and transverse cracks.
- Presence of many working transverse cracks.
- Rocking of slabs due to the presence of voids.
- Presence of many working longitudinal cracks.
- Patch deterioration.
- Lane separation.

- Joint deterioration due to D-cracking.
- Slab deterioration due to reactive aggregates.
- Uneven slab settlement.
- Corner breaks.

Various types of equipment have been used to crack JPCP, including modified pile drivers, guillotine hammers, whiphammers, and impact hammers. Production rates are highly dependent upon the equipment type and the thickness of pavement, and can vary from 0.25 to 6.4 lane km/day.[3,29] Table 4-14.5 provides a summary of the types and production rates for equipment used for cracking and seating, breaking and seating, and rubblizing rigid pavements.[3]

Table 4-14.5. Summary of equipment types, characteristics, and productivity.[3]

Equipment Type	Applications	Energy Range Joules	Production Rate lane km/day	Advantages	Limitations
Crane and Wrecking Ball	Small areas where specialized equipment not justified.		0.40 - 1.2	Equipment generally available.	Requires skilled crane operator.
Whip Hammer	JPCP		1.6	High Productivity covers full lane width.	Not effective on JRCP.
Pavement Breakers/Drop Hammers	JPCP and wire mesh reinforced JRCP.	6,500 - 20,300	0.24 - 1.2	Effective on JRCP.	Developed unusual cracking pattern on JRCP. Several passes required.
Guillotine	JPCP and JRCP.	16,300 - 163,000	1.2 - 6.4	Versatile-effective with JPCP and JRCP: preferred by several States; high productivity; covers full lane width.	
Impact Hammer		2,700			Rubblized portions of concrete below cracks.
Pile-Driver	JRCP.	10,800 - 156,000	0.43	Covers full lane width.	Low productivity.
Resonant Pavement Breaker	JPCP, JRCP, CRCP.	<2,700	0.80	Excellent for Rubblizing.	Not effective for making transverse cracks.

Although many States are cracking and seating deteriorated JPCP, most of them, with the exception of California, have not developed set criteria for its use.[32] In New York, only badly deteriorated pavements that are no longer behaving as structural sections—as indicated by a substantial number of working and/or faulted cracks, a significant loss of load transfer with associated faulting, shearing of longitudinal tie bars, or any combination of these distresses—have been selected as candidates for cracking and seating.[15,33]

Many different overlay thickness design schemes are used with crack and seat pavement sections.[34] Some States, such as California, simply use a set minimum overlay thickness (107 mm); they also mandate the use of a fabric interlayer on top of the leveling course. Other States use various empirical methods, such as equivalent thicknesses, to determine an overlay thickness.[34] The 1993 AASHTO overlay design procedure for the crack and seat technique recommends a range of values for structural layer coefficients of cracked and seated JPCP layers.[1] In the NCHRP Synthesis on the topic, it was concluded that breaking or cracking and seating procedures were effective in alleviating or perhaps even eliminating reflective cracking in HMA overlays attributed to horizontal movement induced by cyclic temperature changes. This means that HMA reflection cracking caused by load applications then becomes the primary determinant of HMA overlay thickness.[34]

The severity of the pavement deterioration is the criterion by which the engineer can decide if cracking and seating is warranted. The cost-effectiveness of cracking and seating may be judged by comparing the potential savings in extensive preoverlay repair with the costs of cracking and seating and increased overlay costs. The crack and seat operation reduces the structural integrity of the concrete pavement, requiring thicker, more expensive overlays and limiting future rehabilitation options.

The size of the broken pieces is critical to the performance of the overlay. States that commonly use cracking and seating specify crack patterns that produce pieces that range in size from 0.50 to 1.8 m, based on the amount of temperature variation expected and the quality of support under the slab.[34] The smaller sizes require a very solid foundation due to a reduction in structural capability of the rigid pavement, but have shown better performance in studies in Minnesota and New York.[15,33] Iowa compared the performance of a range of fractured slab sizes and found that 0.6 by 0.9 m produced the least reflection cracking. However, almost 42 percent of the joints had reflected through after 6 years.[35]

One study recommends that the area of each piece be limited to a range of 0.37 to 0.56 m^2, and emphasizes the importance of controlling the ratio of length to width of the pieces.[36] It was found that reflection cracking occurs more frequently when the length of the cracked pieces is less than their width. For construction purposes, an attempt should be made to keep the dimensions of the cracked pieces approximately equal, or allow a pattern with a slightly greater length than width.

On several projects, cracking and seating did not produce full-depth cracks because either the breaker was too light, the PCC slab was too thick, or the pavement temperatures were too high to allow cracks to occur during the breaking operation. New York requires that full-depth cracking of rigid pavements be verified through coring.[37]

After the rigid pavement has been cracked, the pieces must be firmly seated into the supporting layer to prevent them from rocking under the influence of traffic. Seating is generally accomplished through the use of a pneumatic roller that “massages” each small piece into the foundation while maintaining aggregate interlock. In general, a 445-kN roller is specified for the seating operation, although a 311-kN roller has been used successfully.[29] If the foundation is weak, this procedure may not be totally effective because the slabs can rock. This is especially true in areas with heavy truck traffic.

The weight of the roller used for the seating operation has a dual influence on the resulting reflection cracking. Heavier rollers appear to reduce the rate at which low-severity cracks develop into medium- and high-severity cracks, but they also may cause more low-severity cracking initially.[36] The strength of the foundation has a direct impact on the effectiveness of the heavier rollers. To ensure the performance of the crack and seat operation, heavy rollers should not be used on pavements with a weak foundation. Localized areas with a weak foundation that can be identified should be undercut and the foundation improved.

As with most methods being investigated for reducing reflection cracking, cracking and seating has had both successes and failures. The FHWA completed a limited study of 22 projects, of which only four showed appreciably less reflective cracking in the crack and seat sections than in the control sections.(32) It was concluded that "there generally is a reduction in the amount of reflective cracks through the overlay during the first few years following construction of a crack and seat project. However, after 4 to 5 years, the crack and seat sections exhibited approximately the same amount of reflective cracks as the control sections." The study found that for two of the projects that had a significant reduction in reflection cracks, both had a cement-treated base, were located in warm climates, and were short-jointed plain concrete pavements. Similar findings were reported in another study, which indicated that crack and seat overlays can reduce reflective cracking, particularly in the early years of the overlay's life.(36) There is some evidence that after a number of years, however, the effectiveness of the crack and seat operation diminish.

Benefits from cracking and seating have also been reported. One study reports that cracking and seating is a very effective rehabilitation strategy for JPCP.(3) California has had success with cracking and seating in conjunction with an AC overlay, reporting reductions in vertical deflections and reflective cracks.(31) On crack and seat projects, only about 10 percent of the AC overlays exhibited initial cracking after 5 years of service, compared to nearly 75 percent of conventional AC overlays. California has also used a fabric within the AC overlay on crack and seat projects and report an additional delay of one year to the onset of reflective cracking.(31) The design consists of an initial 30-mm AC leveling course upon which the fabric is placed, followed by a 75-mm AC overlay.

Breaking and Seating

Breaking and seating is conducted only on existing JRCP. To be effective, breaking and seating must reduce horizontal movements by rupturing the reinforcing steel or debonding the concrete from the steel. This effort generally requires that the existing PCC be broken into smaller pieces than required for cracking and seating of JPCP, and thus there is a greater reduction in the structural capacity of the break and seat section.(3)

When the breaking and seating operation does not adequately disrupt the reinforcement, full-depth cracking does not always occur and the reinforcing continues to hold the broken pieces together. As a result, the slab continues to function as a unit. This leads to poorer performance, as large horizontal movements continue at the joints, and reflection cracking will occur as if the breaking and seating had not occurred.(36) If the steel is ruptured, the performance of the break and seat section should not differ from a crack and seat section.

An important consideration in the breaking and seating process is to make sure that the reinforcement is sheared at the pavement breaks. A breaking device that does not impart sufficient force will not shear the steel and reflection cracks can be expected to appear in the HMA overlay. A study in Indiana showed that a whip hammer was unable to produce the desired cracking, but a guillotine hammer was.(38) It is not reported whether the rupturing of the steel was verified. Kentucky also specifies an impact hammer, based on over 10 years of experience.(39) For seating, Kentucky has moved from specifying a 7.3-metric ton vibratory roller to a 32-metric ton pneumatic tire roller (with 7 passes; a 45-metric ton roller is allowed with 5 passes).(39)

The actual performance of break and seat sections is quite variable. A recent study concluded that "until further improvements are made, the use of the conventional break/seat should be viewed with extreme caution. If construction equipment other than sonic pavement breakers are used, then specified crack spacings of 150 to 460 mm (150 to 305 mm preferred) should be used."(3) Kentucky has linked limiting reflection cracking to minimizing the size of the broken pieces. Their specification requires that all pieces be smaller than 750 mm, and 80 percent must fall in the 455 to 610 mm range.(39) The effect of broken particle size on a project on I-71 in Kentucky is shown in figure 4-14.9. The HMA overlay was 180 mm thick, and after 7 years service pavement sections with small crack patterns showed the least reflection cracking.

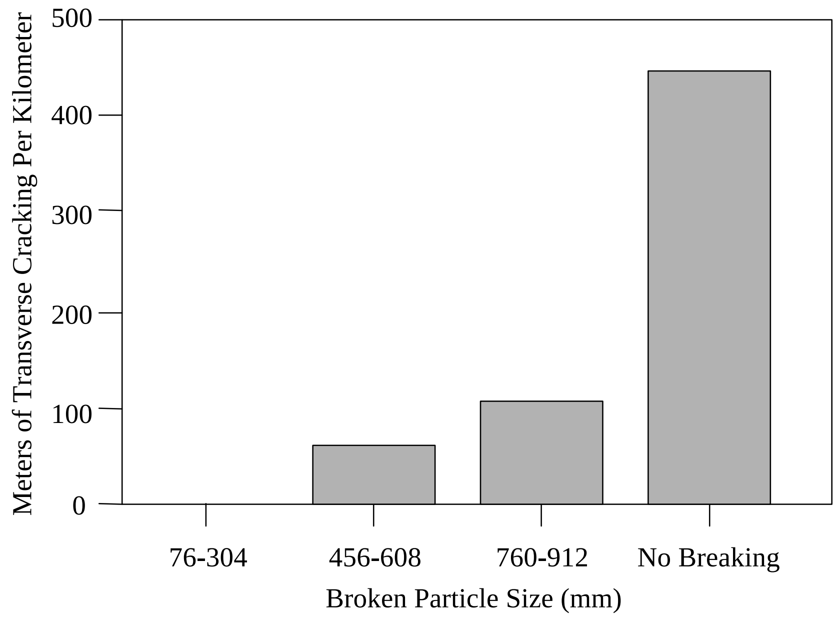


Figure 4-14.9. Comparison of reflection cracking for different crack patterns.(39)

In Indiana's study of this technique, which included a number of variables as well as a control section, researchers concluded that "the [break and seat] technique successfully delayed the crack development for 5 years."(38) When cracks did occur, it was hypothesized that they were due more to aging and weathering than to reflection cracking. After 7 years, however, the cracks on the break and seat sections were about as bad as those on the control. This is shown in figure 4-14.10, which compares crack intensities on the control sections (A), with those on break and seated sections with 130-mm overlays (B), 165-mm overlays (C), and 215-mm overlays (D).

The FHWA is currently conducting research on the effect of break patterns on JRCP, under Special Project 202. Break patterns of 150, 460, and 760 mm are being evaluated against the performance of unbroken slabs in Ohio, Kentucky, Louisiana, and West Virginia.

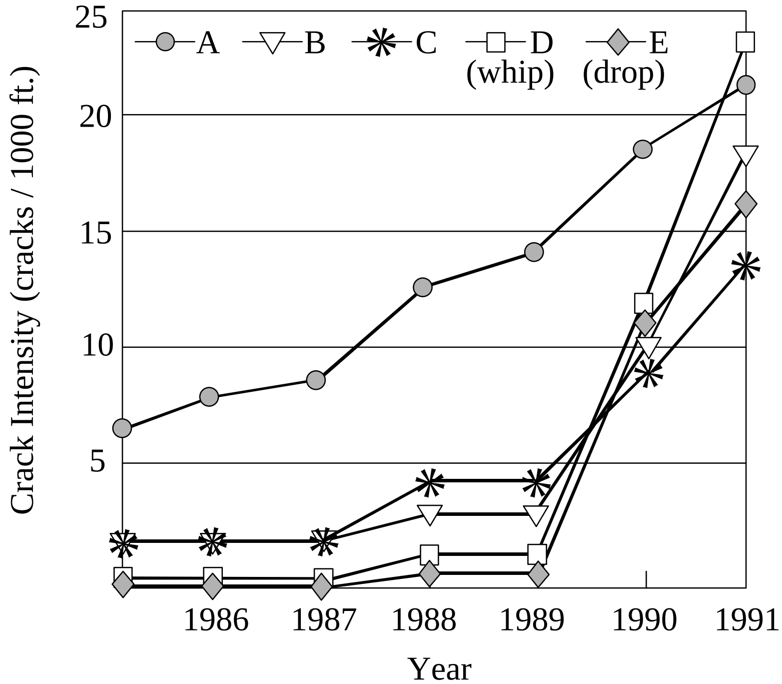


Figure 4-14.10. Comparison of reflection crack intensities on control and experimental sections.[38]

Rubblization

A variation of rigid pavement fractured slab techniques that is enjoying increased use is "rubblization." This procedure uses a resonant pavement breaker capable of reducing the existing slab into small pieces varying from sand-sized particles to pieces 150 mm across. The resonant pavement breaker accomplishes this by applying a 8.9-kN impact force at a frequency of 44 impacts per second to the pavement surface through a shoe that is attached to a massive steel beam.[40] In essence, this beam acts as a "giant tuning fork," shattering the pavement into pieces having an average size of 25 to 50 mm, with no pieces larger than 150 mm.[41] The most commonly used piece of equipment is the resonant frequency pavement breaker, Model PB-4, which is manufactured by Resonant Machines Inc. This equipment applies its impact through a 305 mm square plate, and has achieved production rates of as high as 670 m^2/hr on a 230-mm thick pavement.

After the pavement is rubblized, it is compacted with at least two passes of a vibratory roller weighing at least 89 kN.[42] One advantage that rubblization has over traditional break and seat is it is unnecessary to rupture the steel, as the concrete completely debonds from the steel during the rubblization process. Therefore, rubblization can be used effectively on JRCP and CRCP, with only the need to remove steel exposed on the surface prior to overlaying.

Another advantage of rubblization is that the impact load applied is relatively small, and thus disturbance to the support material, underground drainage structures, and utilities is minimized. Normally, water is sprayed during the rubblizing operation to suppress dust.

Rubblization is usually considered when the amount of deterioration of the existing pavement is so great that normal crack and seat or break and seat methods would not be effective, or when CRCP is being rehabilitated. It is often the preferred approach in warm climates. The reduction of the rigid pavement to aggregate-sized particles dramatically reduces the structural capacity of the pavement, thus requiring that thicker overlays be used. The relatively slower production rates and increased overlay costs can only be justified if no other rehabilitation alternative is more cost effective.

Similar design schemes are employed with rubblized pavements as with crack and seat, usually converting the rubblized layer into an equivalent thickness or assigning it a layer coefficient. The 1993 AASHTO overlay design procedure for HMA overlays of rubblized rigid pavements recommends a range of values for structural layer coefficients of the rubblized layers. The suggested values assume that the fracturing technique is performed properly, and allows a downward adjustment for deterioration in the base layer.[1]

Performance to date on rubblized sections has been good when sufficient overlay thicknesses are applied. See references 3, 29, 40, and 41. Early failures of this technique appear to be related to inadequate overlay thickness. However, this is a relatively new technique, and long-term performance data are not available. Future research will be able to address the cost-effectiveness of this approach.

Slab Repair and Replacement at Deteriorated Transverse Cracks/Joints

The deterioration of reflected cracks caused by differential vertical movement can be addressed by full-depth concrete repair (see module 4-5). This procedure permits re-working of the foundation and repair of badly cracked areas, while improving load transfer through the addition of dowels. Another option is to restore load transfer at joints and working cracks through use of dowel bars placed in slots (addressed in module 4-9).

Reflection Crack Severity Control By Sawing and Sealing Joints in HMA Overlays

In contrast to treatments that attempt to reduce the rate of reflection cracking, some agencies concede that reflection cracking will occur. Their approach, then, is to control the severity and rate of deterioration of reflection cracking. One such approach is to saw and seal joints in the overlay. Sawing and sealing joints consists of marking joints and cracks in the existing pavement prior to overlaying, and then sawing joint reservoirs in the HMA overlay directly over the underlying joints or cracks. These new reservoirs are then sealed and treated as joints. This technique is illustrated in figure 4-14.11.

New Jersey, Connecticut, New York, and other States have investigated the use of sawing and sealing joints in HMA overlays. See references 3, 32, 42, and 43. One study found that 12 States have experimented with or are using sawing and sealing as a routine procedure.[44] This technique is most commonly used with HMA overlays of jointed concrete pavements, but has been attempted in Colorado using an HMA overlay of an existing flexible pavement.[45]

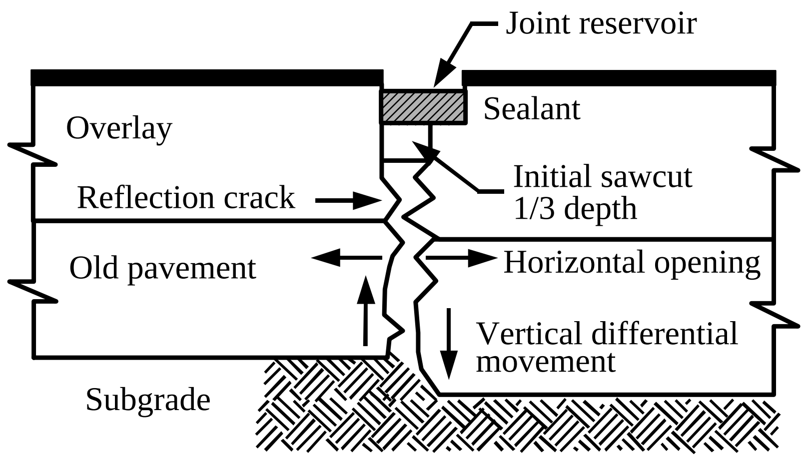


Figure 4-14.11. Illustration of sawing and sealing of joints in HMA overlay.

Sawing and sealing, when properly done, results in the formation of a reflection crack through the weakened plane created by the saw cut. This reduces spalling of the crack, and provides an easily maintained and aesthetically acceptable reflection crack. It is absolutely critical that the sawcut be placed immediately above the existing joint or crack, as a deviation of even 25 mm can lead to a secondary crack forming adjacent to the sawed joint, resulting in spalling. See references 43, 44, 45, and 46.

States that have used these techniques typically have long-jointed reinforced pavements (greater than 12 m). These long slabs experience a lot of movement at the joint and are more susceptible to reflection cracking than plain concrete pavements with shorter joint spacings. New York, for example, follows the guidelines listed in table 4-14.6.[42] Shorter-jointed pavements experience less movement but do require a proportionally larger amount of sawing. Typically, the saw cut has a width of 10 mm and a depth of 13 mm, although this varies from State to State depending on the slab length.[46]

Table 4-14.6. Recommended sawcut dimensions in New York.[42]

Slab Length	Width, mm	Depth, mm
15.2 m or less	13	16
15.3 to 18.9 m	16	16
18.9 to 22.9 m	19	16
19.0 to 26.5 m	22	19
26.5 to 30.5 m	25	22

Sawing and sealing should only be considered where the existing pavement has well defined joints; it should not be considered if excessive spalling or cracked slabs exist.[42,43] Pavements with wide joints (greater than 19 mm) or numerous full- and partial-depth repairs, misaligned slabs, and mid-slab cracking, are not good candidates for this technique.

Use of this technique means conceding the fact that reflective cracking will occur. By sawing joints in the HMA overlay, the cracks that are produced are straight and sealed to help retard the infiltration of moisture or incompressibles into the pavement structure. As long as the saw cuts are made in the proper location, this technique has been shown to be highly successful.(43,44,45)

Increased Overlay Thickness

Increased overlay thicknesses are occasionally cited as an approach to reduce the rate and severity of reflection cracking. The general rule of thumb is that a crack will reflect about 25 mm a year; that is, a crack will take 4 years to reflect through a 100-mm overlay or 8 years to get through an 200-mm overlay. And in fact, some believe that at thicknesses greater than 200 mm, reflection cracking is essentially eliminated (Walsh). This may be explained, in part, by the insulating effect of the additional thickness. A study in New York noted that "thicker overlays were found to reduce daily and seasonal temperature changes in the underlying rigid pavements."(47)

However, the cost-effectiveness of this approach should be examined in relation to the cost of the other treatments mentioned previously. In an Indiana study of reflection crack control, it was found that "thicker overlays did not improve pavement performance considerably but greatly increased construction costs."(38) Thicker HMA layers are likely to fail in other ways, such as by thermal cracking and aging, rutting, and raveling.

Georgia found that by increasing the overlay thickness from 50 to 100 to 150 mm, the incidence of reflection cracking was dramatically decreased.(19) They also found that a similar delay in the rate of reflection cracking could be obtained from use of fabrics. The cost of the additional HMA thickness should be compared to the cost of the fabric installation to determine which alternative is most cost effective. Additional considerations, such as increased structural capacity or clearance restrictions, would also contribute to this analysis.

General Comments on Reflection Crack Control

The problem of reflection cracking of HMA overlays of rigid pavements attracts the attention of many highway agencies. Many different treatments have been attempted with varying success. In extreme cases (such as large temperature variations), it seems as if no treatment offers relief and temporary mitigation is the best that can be expected. On the other hand, in mild climates where the underlying pavement does not experience large vertical movement, many treatments have the potential to succeed. In the absence of universally applicable guidelines, perhaps an approach such as that described by Moody(48) is appropriate. In Texas, the rehabilitation of a jointed concrete pavement with an HMA overlay was accompanied by the following reflection crack control techniques:

- Full-depth repair (repairs followed by a 100-mm HMA overlay).
- Crack and seat (0.6 m pieces, 100- to 140-mm HMA overlay).
- Crushed stone base interlayer (200-mm crushed stone, 50-mm HMA overlay).
- Open-graded HMA interlayer (90-mm open-graded HMA, 75-mm fine-graded HMA, 40-mm coarse surface HMA mix).
- Styrene-Butadiene-Styrene modified seal coat interlayer (25-mm SBS on existing HMA, followed by 75-mm HMA).
- Coarse surface mix HMA overlay (75-mm on the existing HMA overlay).
- Control (40-mm on the existing HMA overlay).

The percent of joints that reflected through after 2 years is shown in figure 4-14.12. While the performance of the different approaches is somewhat obscured by the different treatment thicknesses and the fact that in some cases an existing HMA overlay was left in place, it is clear that thicker treatments perform better, at least initially. However, thicker treatments cost more and are not always feasible because of construction limitations. In selecting an appropriate treatment, Moody[48] suggests that the engineer must weigh the benefits of retarding the onset and rate of reflection cracking versus the associated costs of the benefit.

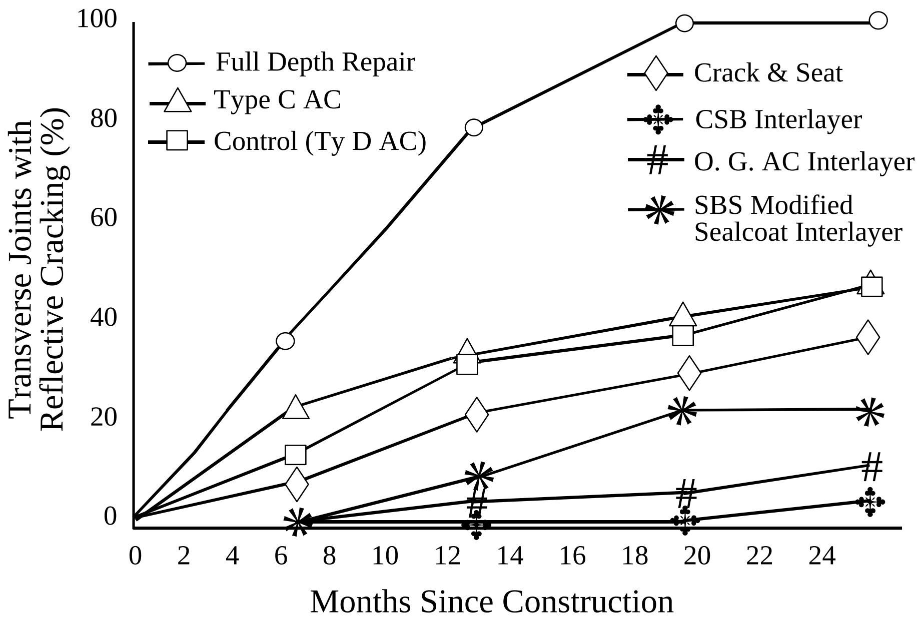


Figure 4-14.12. Reflected transverse joints with different crack control treatments.[48]

12. SUMMARY

HMA overlays are an effective and widely used rehabilitation method for rigid pavements. The determination of whether it will be more cost-effective to conduct extensive repairs on the existing pavement and use a minimum thickness of overlay, or to conduct minimum repairs on the existing pavement and use thicker overlays, requires an economic analysis for the specific project under consideration. Factors such as the rate of deterioration must be considered, and repair amounts and associated costs must be determined for a specific pavement section after a thorough analysis.

There are three approaches for determining the thickness required for a structural overlay, with the structural deficiency approach most commonly used. The 1993 AASHTO overlay design procedures provide field-tested methods that are based on deflections, conditions surveys, or remaining life, and provide a useful tool for all types of overlays. Each agency is advised to calibrate the design procedure to reflect local conditions. It is also suggested that the design engineer utilize more than one procedure to check the design, regardless of the original procedure used.

The consideration of reflection cracking is also a critical aspect of an HMA overlay project. There are several alternatives available for controlling reflection cracking, and these need to be carefully evaluated for each project. This module presents the mechanisms behind the development of reflection cracking from the different sources of deformations and stresses, in particular low temperatures and traffic loads.

Each treatment addresses these inputs in a different manner to accomplish the following:

- Reduce the severity of the reflection cracks.
- Reduce water infiltration into the cracks.
- Retard the rate of reflection cracking.

Due to the variable performance of most reflection cracking techniques, it is recommended that each agency evaluate the performance of various techniques in similar climates and pavement types to select the most promising techniques for additional testing. Full-scale tests with control sections should then be carefully designed and conducted over several years before using any of the techniques on a widespread basis.

13. REFERENCES

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

2. Asphalt Institute (AI). 1983, “Asphalt Overlays for Highway and Street Rehabilitation,” MS-17, The Asphalt Institute, College Park, MD.

3. National Asphalt Pavement Association. 1991, “Guidelines and Methodologies for the Rehabilitation of Rigid Highway Pavements Using Asphalt Concrete Overlays,” Technical Report. National Asphalt Pavement Association, Lanham, MD.

4. Permanent International Association of Road Congresses (PIARC). 1992, “Evaluation and Maintenance of Concrete Pavements,” Permanent International Association of Road Congresses, Paris, France.

5. Strategic Highway Research Program (SHRP). 1993, “Distress Identification Manual for the Long-Term Pavement Performance Project,” Report No. SHRP-P-338, Strategic Highway Research Program, Washington, DC.

6. Darter, M. I. and K. T. Hall. 1990, “Structural Overlay Strategies for Jointed Concrete Pavements, Volume IV—Guidelines for the Selection of Rehabilitation Alternatives,” FHWA-RD-89-145, Federal Highway Administration, Washington, DC.

7. Seeds, S. B., B. McCullough, and R. Carmichael. 1985, “Asphalt Concrete Overlay Design Procedure for Portland Cement Concrete Pavements,” Transportation Research Record 1007, Transportation Research Board, Washington, DC.

8. Majidzadeh, K., A. Abdulshafi, G. J. Ilves, and A. McLaughlin. 1987, “A Mechanistic Model for Thermally Induced Reflection Cracking of Portland Cement Concrete Pavement with Reinforced Asphalt Concrete Overlay,” Transportation Research Record 1117, Transportation Research Board, Washington, DC.

9. Smith, R. E., R. Palmieri, M. Darter, and R. Lytton. 1984, “Pavement Overlay Design Procedures and Assumptions, Volume II—Guide for Designing an Overlay,” FHWA/RD-85/007, Federal Highway Administration, Washington, DC.

10. Jayawickrama, P. W. and R. Lytton. 1987, “Methodology for Predicting Asphalt Concrete Overlay Life Against Reflective Cracking,” Proceedings of the Sixth International Conference on the Structural Design of Asphalt Pavements, Volume I.

11. Barksdale, R. D. 1991, “Fabrics in Asphalt Overlays and Pavement Maintenance,” NCHRP Synthesis of Highway Practice 171, Transportation Research Board, Washington, DC.

12. Wood, W. A. 1984, “Reducing Reflection Cracking in Bituminous Overlays,” FHWA-EP-85-02, National Experimental and Evaluation Program (NEEP) Project No. 10, Federal Highway Administration, Washington, DC.

13. Button, J. W. 1989, “Overlay Construction and Performance Using Geotextiles,” Transportation Research Record 1248, Transportation Research Board, Washington, DC.

14. Pourkhosrow, G. 1985, “Nonwoven Polyester and Polypropylene Engineering Fabrics in Oklahoma Pavements,” Oklahoma Department of Transportation, Norman, OK.

15. Noonan, J. E. and F. McCullagh. 1980, “Reduction of Reflection Cracking in Bituminous Overlays on Rigid Pavements,” Report No. 78, New York State Department of Transportation, Albany, NY.

16. Dykes, J. W. 1985, “The Use of Fabric Interlayers to Retard Reflective Cracking,” Proceedings, Association of Asphalt Paving Technologists, Volume 49.

17. Maurer, D.A., and G. J. Malasheskie. 1989, “Field Performance of Fabrics and Fibers to Retard Reflective Cracking,” Transportation Research Record 1248, Transportation Research Board, Washington, DC.

18. Rutkowski, T.S. 1985, “Reduction of Reflective Cracking in Asphaltic Concrete Overlays of Rigid Pavement,” FHWA/WI-85/01, Wisconsin Department of Transportation, Madison, WI.

19. Gulden, W. and D. Brown. 1984, “Overlays for Plain Concrete Pavements,” Georgia Department of Transportation, Forest Park, GA.

20. Predoehl, N. H. 1990, “Evaluation of Paving Fabric Test Installations in California—Final Report,” FHWA/CA/TL-90/02, California Department of Transportation, Sacramento, CA.

21. McGhee, K. H. 1979, “Attempts to Reduce Reflection Cracking of Bituminous Concrete Overlays on Portland Cement Concrete Pavements,” Transportation Research Record 700, Transportation Research Board, Washington, DC.

22. Morris, G. R. and C. McDonald. 1976, "Asphalt Roller Stress-Absorbing Membranes: Field Performance and State of the Art," Transportation Research Record 595, Transportation Research Board, Washington, DC.

23. Pourkhosrow, G. 1986, "Precoated Membranes," Oklahoma Department of Transportation, Norman, OK.

24. McGhee, K. H. 1982, "Control of Reflection Cracking in a Fabric-Reinforced Overlay on Jointed Portland Cement Concrete Pavement," VHTRC 83-R8, Virginia Highway and Transportation Research Council, Charlottesville, VA.

25. Arizona Transportation Research Center (ATRC). 1989, "Paving Fabrics for Reducing Reflective Cracking," ATRC Research Notes, ADOT Project RS-274-(8) P, Arizona Department of Transportation, Phoenix, AZ.

26. Knight, N. E. 1985, "Heavy Duty Membranes for the Reduction of Reflective Cracking in Bituminous Concrete Overlays—Final Report," Research Project Number 79-6, Pennsylvania Department of Transportation, Harrisburg, PA.

27. Fowler, D. 1982, "Illinois Experience with Reflection Crack Treatments," Presented at the Annual Transportation and Highway Engineering (THE) Conference, University of Illinois, Urbana, IL.

28. Asphalt Institute (AI). 1989, "Crack Relief Layer," Technical Bulletin No. 4, Asphalt Institute, College Park, MD.

29. Crawford, C. 1989, "Cracking and Seating of PCC Pavements Prior to Overlaying with Hot-Mix Asphalt," Information Series 98, National Asphalt Pavement Association, Lanham, MD.

30. Kilareski, W. P. and S. Stoffels. 1990, "Structural Overlay Strategies for Jointed Concrete Pavements, Volume II—Cracking and Seating of Concrete Slabs Prior to AC Overlay," FHWA-RD-89-143, Federal Highway Administration, Washington, DC.

31. Wells, G. K., J. B. Hannon, and N. H. Predoehl. 1991, "California Experience with Cracking and Seating of Concrete Pavements," Transportation Research Record 1307, Transportation Research Board, Washington, DC.

32. Federal Highway Administration (FHWA). 1990, "Crack and Seat Performance," Review Report, Demonstration Projects Division and Pavement Division, Federal Highway Administration, Washington, DC.

33. Vyce, J. M. 1983, "Reflection Cracking in Bituminous Overlays on Rigid Pavements," Report No. 109, New York State Department of Transportation, Albany, NY.

34. Thompson, M. R. 1989, "Breaking/Cracking and Seating Concrete Pavements," NCHRP Synthesis of Highway Practice 144, Transportation Research Board, Washington, DC.

35. Harris, G. K. 1993, "Cracking and Seating to Retard Reflective Cracking—Hamilton County," Iowa Highway Research Board Project HR-277, Iowa Department of Transportation, Ames, IA.

36. Voigt, G.F., S. Carpenter and M. Darter. 1989, "Rehabilitation of Concrete Pavement, Volume 2—Overlay Rehabilitation Techniques, FHWA/RD-88/072, Federal Highway Administration, Washington, DC.

37. McCarty, W. M. 1987, "Cracking and Seating PCC Pavement—Construction Techniques and Overlay Performance," New York Department of Transportation, Albany, NY.

38. Jiang, Y. and R. S. McDaniel. 1993, "Application of Cracking and Seating and Use of Fibers to Control Reflective Cracking," Transportation Research Record 1388, Transportation Research Board, Washington, DC.

39. Graves, R. C., D. L. Allen, and G. W. Sharpe. 1993, "Breaking and Seating of Concrete Pavements: Kentucky's Experience," Transportation Research Record 1388, Transportation Research Board, Washington, DC.

40. Bernard, D. W. 1988, "Rubblizing PCC. Pavement—Construction Techniques and Overlay Performance," Technical Report 88-2, New York State Department of Transportation, Albany, NY.

41. Asphalt Institute (AI). 1988a, "Rubblizing Prior to Overlay with Asphalt Concrete," Technical Bulletin TB-3, Asphalt Institute, College Park, MD.

42. New York State Department of Transportation (NYDOT). 1985, "Sawing and Sealing Joints in Bituminous Concrete Overlays," Construction Specification Item 18403.2501, New York State Department of Transportation, Albany, NY.

43. Hellriegel, E. J. 1987, "Second Generation Pavement Overlays," FHWA/NJ-86-013-7778, New Jersey Department of Transportation, Trenton, NJ.

44. Kilareski, W. P. and R. Bionda. 1990, "Structural Overlay Strategies for Jointed Concrete Pavements, Volume I—Sawing and Sealing of Joints in AC Overlays of Concrete Pavements," FHWA-RD-89-142, Federal Highway Administration, Washington, DC.

45. Harmelink, D. S. 1989, "Sawed Joints in AC Pavement," CDOH-DTD-R-89-14, Colorado Department of Highways, Denver, CO.

46. Asphalt Institute (AI). 1988b, "Sawcut & Seal After Overlay with Asphalt Concrete," Technical Bulletin TB-2, Asphalt Institute, College Park, MD.

47. McAuliffe, D., L. J. Bendaña, H-J. Chen, and R. Morgan. 1995, "Overlays on Faulted Rigid Pavements," Transportation Research Record 1473, Transportation Research Board, Washington, DC.

48. Moody, E. D. 1994, "Field Investigations of Selected Strategies to Reduce Reflective Cracking in Asphalt Concrete Overlays Constructed Over Existing Jointed Concrete Pavements," Transportation Research Record 1449, Transportation Research Board, Washington, DC.

MODULE 4-15

IDENTIFICATION OF FEASIBLE RIGID PAVEMENT REHABILITATION ALTERNATIVES

1. INSTRUCTIONAL OBJECTIVES

Block 4 introduces the techniques that are available to rehabilitate rigid pavements. These rehabilitation techniques are introduced separately within their own modules, which include a discussion of when the technique may be applicable to correct a specific distress or condition, as well as its limitations and effectiveness. In some cases, concurrent repairs are also recommended. For example, the module on load transfer restoration also discusses undersealing and retrofitting edge drains, which are themselves separate modules.

Having presented the various techniques for rigid pavement rehabilitation, each of these techniques can be assembled into an overall framework. The overall framework is a series of steps or processes that includes determining the distress mechanisms that apply to the particular project and identifying the most reasonable solutions (rehabilitation treatments) that meet the needs or constraints of the project. Common constraints include funding limitations, anticipated traffic loads, resulting structural condition, and desired service life. These steps are discussed in block 5. In this module, one part of the process is elaborated upon, namely identifying feasible alternatives. At the conclusion of this module, the participant shall be able to accomplish the following:

1. Recognize the importance of an overall framework for the identification of feasible rigid pavement rehabilitation alternatives.
2. Analyze information from a project and develop a list of specific treatments for that project that will best meet the needs and constraints of that project.
3. Describe what a decision tree or chart is and how it is developed and used.
4. Describe the limitations and potential problems that are associated with the strict use of a decision tree.

2. INTRODUCTION

Deciding which pavements to repair, what repairs are needed, and when these repairs should be performed is a complex process. At the network level, at which all of the pavements for which an agency is responsible are considered, such decisions are being increasingly made with the aid of pavement management systems (PMS). Such systems include a database with information about traffic, construction, and performance, as well as models, rules, and other tools that assist in decision-making.

This course focuses its efforts at the project level, the point at which a specific section of pavement is identified as a candidate for treatment, but the final treatment is not yet decided. Figure 4-15.1 shows an overall process that can be followed to take a distressed pavement from evaluation through to a completed rehabilitation project.[(1)] The first step is to collect project information. This follows the procedures described in block 2, and includes a review of pavement design information, construction records, both historical and projected traffic loadings, and environmental data. It may also include historical condition data, if available. Next, the condition of the existing project is evaluated. This includes the collection of distress data, a structural evaluation, such as with a falling weight deflectometer (FWD), a drainage survey, and an evaluation of the pavement materials.

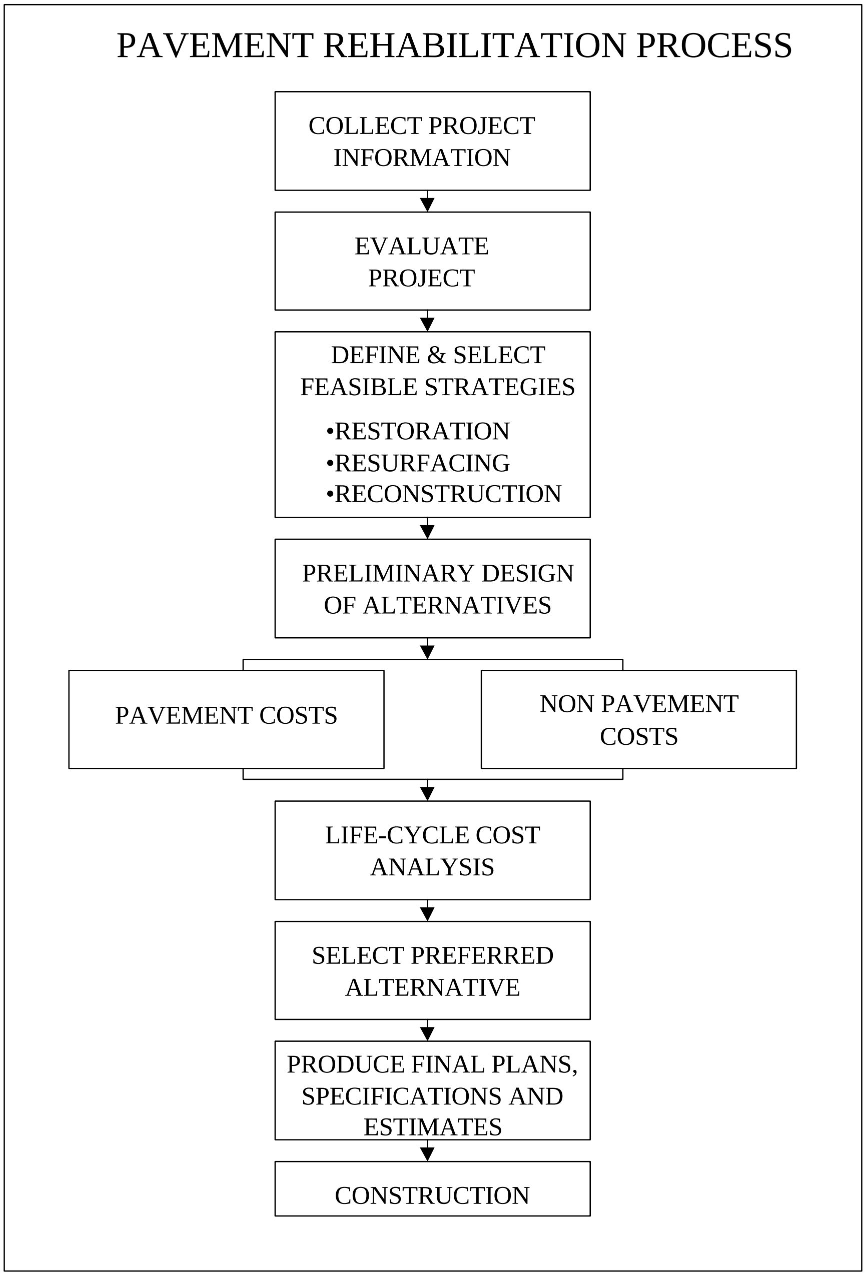


Figure 4-15.1. Overview of pavement rehabilitation strategy selection process.[1]

The third step, identifying and selecting feasible rehabilitation alternatives, is what is covered in this module. It is this process that may have the greatest effect on the overall success of a project. Yes, reconstruction will correct a joint spalling problem, but perhaps partial-depth repair is a more appropriate and cost-effective strategy. A crack can be repaired by full-depth repair techniques, but is sealing better? Decisions made at this stage can have the greatest impact on the cost-effectiveness of the resulting rehabilitation strategy and the ability of an agency to provide to the traveling public roads that perform well.

The final engineering step in the process is to complete the design of the feasible alternatives. Again, here an emphasis should be placed on completing the engineering for alternatives that not only address the existing conditions, but also recognize the limitations or restrictions that cannot be changed. These limitations might include the following:

- Available funding.
- Available closure times.
- Age of the structure.
- Current and projected traffic volumes or loadings.
- Anticipated future work activities.

The remaining steps in the rehabilitation process shown in figure 4-15.1 are addressed in later modules. Module 5-1 introduces concepts that are used to calculate the costs that are associated with a project and to perform a life-cycle cost analysis so that alternatives with different lives can be compared fairly. The process of selecting the preferred alternative is addressed elsewhere.

3. DEFINITIONS

Familiarity with several terms is essential to an understanding of the processes presented in this module. These are described below.

Decision Trees

Decision trees can be thought of as flow charts. They are a graphical representation that is assembled to represent the logic in a decision-making process. In the pavement rehabilitation process, a decision tree might have as its initial input a pavement condition or set of conditions, and the treatment as an output. The tree consists of nodes, which describe a piece of information about the pavement, and branches, which lead from the nodes and represent measured data related to the information at the node.

Decision Table

A decision table is similar to a decision tree, in that it relates pavement condition (be it distresses, deflections, or some other measure), to recommended strategies. A typical arrangement is to represent pavement condition in the leftmost column, with the recommendations for rehabilitation in the rightmost column. In between, columns might become more specific, or be used to differentiate between causes of the condition or severity levels.

Feasible Alternative

A rehabilitation alternative that addresses the observed pavement condition and meets project constraints such as budget and timing is called a feasible alternative. Cost-effective repairs are more likely to be the result if they are preceded by the development of a range of feasible alternatives.

Distress Mechanisms

The cause or causes of a pavement's deterioration is termed its distress mechanism. Identifying the correct distress mechanism(s) is important in the rehabilitation process because distresses can be thought of as symptoms, for which selecting the appropriate treatment requires identifying the underlying cause. Rigid pavement distress mechanisms are covered in module 2-5. They include the following:

- Load-related
  - Pumping
  - Faulting
  - Transverse (fatigue) cracking
  - Spalling (secondary)

- Environment-related
  - D-Cracking
  - Pumping
  - D-Cracking
  - Faulting

- Construction/Material-related
  - D-Cracking
  - Alkali-silica reactivity
  - Spalling
  - Joint sealant failure
  - Scaling
  - Cracking

When the distress mechanism is identified, it makes the task of identifying feasible alternatives easier. Some distresses are caused by more than one factor, however, and the success of a rehabilitation strategy can depend on discovering which factors are contributing to the development of the distress.

4. USE OF DECISION TREES TO DETERMINE FEASIBLE SOLUTIONS

On the face of it, matching rehabilitation treatments with a pavement's condition is not a difficult process: where there's a spall, do a partial-depth repair; where there's a crack, seal it. Unfortunately, identifying feasible strategies is not quite this simple. What if a pavement is faulting and pumping? Which of the following possible treatments are selected: undersealing, load transfer restoration, grinding, asphalt overlay, or crack and seat and overlay? Would reconstruction be more appropriate? What is an appropriate rehabilitation if transverse cracks appear within a year of construction? Is this still appropriate if the cracks appear after 15 years? Of course, there's no unique answer: every project must be engineered so that the treatments that are applied are appropriate for the existing conditions, and cost-effective within the project constraints.

As is noted in module 3-11, decisions about appropriate rehabilitation strategies appear to be made almost intuitively by experienced engineers. While they are not infallible, experienced engineers do have years of familiarity with how their pavements' perform, which materials and treatments work under what conditions, when the best time is to place different treatments, and so on. It is a widely held perception that this experience base is disappearing due to a number of forces at play within highway agencies (such as retirement, a move away from specialization, and downsizing). Some recent efforts have attempted to capture this experience so that it is accessible to the next generation of pavement engineers. For example, module 3-11 notes the role of pavement management systems which, at a network level, link up treatments with pavements, and knowledge-based expert systems (KBES), which are a tool that links up specific pavement conditions with appropriate treatments in the same way that an experienced engineer approaches the problem.

PMS and expert systems can be extremely useful tools, but even where they are used routinely, the practicing engineer should have a good grasp of their pavements' condition and what treatments will address that condition.

A general framework for selecting the appropriate strategy involves matching distresses with possible repairs. Such a relationship is shown in table 4-15.1.[1] Note that this table presents a range of repairs; the appropriate repair for a particular pavement clearly depends upon the extent and severity of the actual distresses, as well as a host of other factors. The information presented in this table is similar to what

Table 4-15.1. Rigid pavement distresses and possible rehabilitation treatments (modified from reference 1).

Jointed Concrete Pavements (JCP)		**Continuously Reinforced Concrete Pavements (CRCP)**	
Distress/Problem	Rehabilitation Method/Treatment	Distress/Problem	Rehabilitation Method/Treatment
Pumping	Slab Stabilization Joint Reseal Full-Depth Repair Load Transfer Restoration Retrofitted Drainage Retrofitted Portland Cement Concrete (PCC) Shoulders	Structural Deficiency	Slab Stabilization Full-Depth Repair Retrofitted PCC Shoulders Replace/Recycle Lanes Bonded Overlay Unbonded Overlay Hot-Mix Asphalt (HMA) Overlay
Faulting	Diamond-Grinding Slab Stabilization Joint Reseal Full-Depth Repair Load Transfer Restoration Retrofitted Drainage Retrofitted PCC Shoulders Bonded Overlay Unbonded Overlay HMA Overlay	Faulting (at cracks)	Diamond-Grinding Slab Stabilization Full-Depth Repair Retrofitted Drainage Retrofitted PCC Shoulders Bonded Overlay Unbonded Overlay HMA Overlay
Slab Cracking	Slab Stabilization Full-Depth Repair Load Transfer Restoration PCC Recycled Lane Bonded Overlay Unbonded Overlay HMA Overlay	Crack Deterioration	Slab Stabilization Full-Depth Repair Replace/Recycle Lane Unbonded Overlay HMA Overlay
Joint Spalling	Joint Reseal Partial-Depth Repair Full-Depth Repair	Punchouts & Pumping	Slab Stabilization Full-Depth Repair

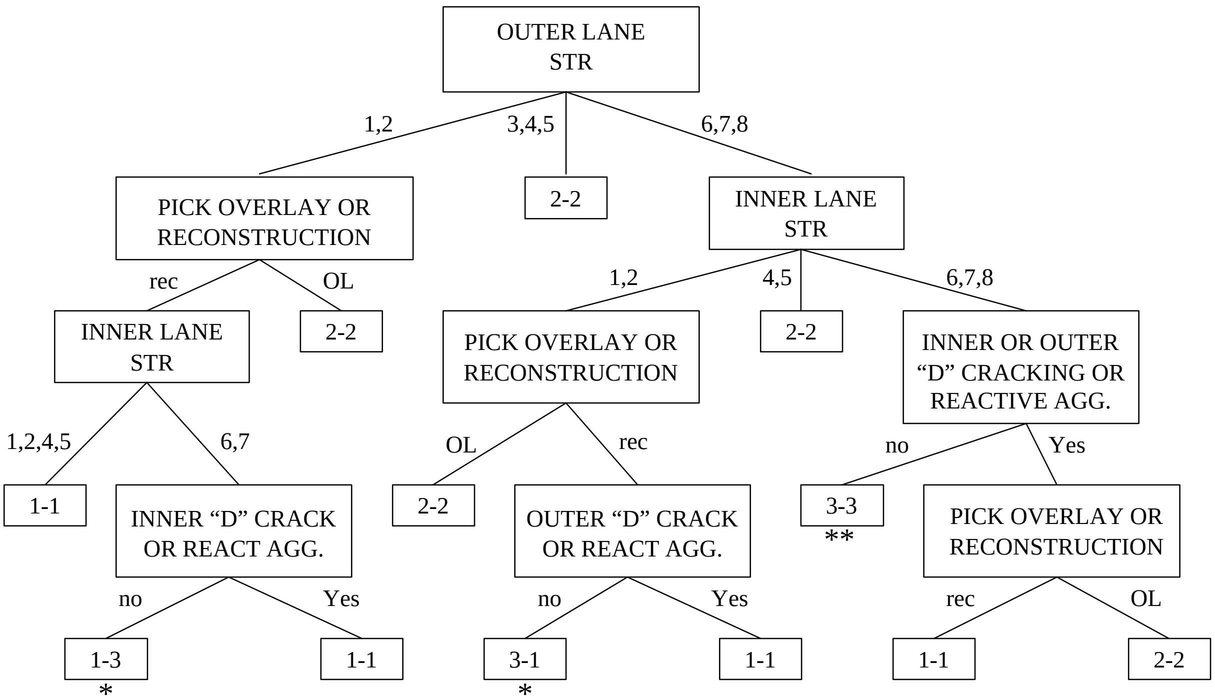
might be found in a pavement management system at the network level, although a good PMS would go beyond linking distresses to potential repairs to consider at least some of the other factors.

Table 4-15.1. Rigid pavement distresses and possible rehabilitation treatments (modified from reference 1) (continued).

Jointed Concrete Pavements (JCP)		**Continuously Reinforced Concrete Pavements (CRCP)**	
Distress/Problem	Rehabilitation Method/Treatment	Distress/Problem	Rehabilitation Method/Treatment
D-Cracking	Partial-Depth Repair Full-Depth Repair Unbonded Overlay Replace/Recycle Lanes	D-Cracking	Partial-Depth Repair Full-Depth Repair Unbonded Overlay Reconstruction
Polishing/Scaling	Diamond-Grinding Bonded Overlay HMA Overlay	Polishing/Scaling	Diamond-Grinding Bonded Overlay HMA Overlay
Blow-up	Full-Depth Repair		
Structural Deficiency	Slab Stabilization Full-Depth Repair Load Transfer Restoration Retrofitted PCC Shoulders Replace/Recycle Lanes Bonded Overlay Unbonded Overlay HMA Overlay		

How is the transition made from the generic approach suggested by table 4-15.1 to a specific list of feasible alternatives for a given project? One approach to narrowing from feasible alternatives to realistic approaches for an actual project is through the use of decision trees. Decision trees can be incredibly complex, however, especially if they attempt to cover all possible situations. Consider, for example, figure 4-15.2.[2] This flow chart or decision tree presents a methodology that is used to select a main rehabilitation approach for jointed plain concrete pavements (JPCP) (separate charts are presented for jointed reinforced concrete pavements [JRCP] and CRCP). It is based on the range of structural deficiencies that are defined in table 4-15.2, which also relates the structural condition to feasible rehabilitation strategies. The different structural conditions themselves are also identified by the decision tree shown in figure 4-15.3, which brings in some of the additional factors that are considered in selecting appropriate strategies, such as environment, pavement design, and annual equivalent single-axle loads (ESAL) volumes. These examples show the complexity of selecting feasible alternatives.

A similar way of presenting the information in decision trees is with decision tables. The advantage of a decision table over a decision tree is that it has the potential to present information in a clearer manner. The example shown in table 4-15.3 is adapted from a PIARC report, *Evaluation and Maintenance of Concrete Pavements*.[3] This table relates an observed distress with associated distresses and appropriate strategies. The tables in the report actually have five column headings, which include specific details of the level of the observed distress, its most probable cause, and associated distresses, before concluding with a recommended rehabilitation. As can be seen from this table, there are even options within the final column to address short-term fixes and long-term fixes.



* Option to go to 1-1 provided

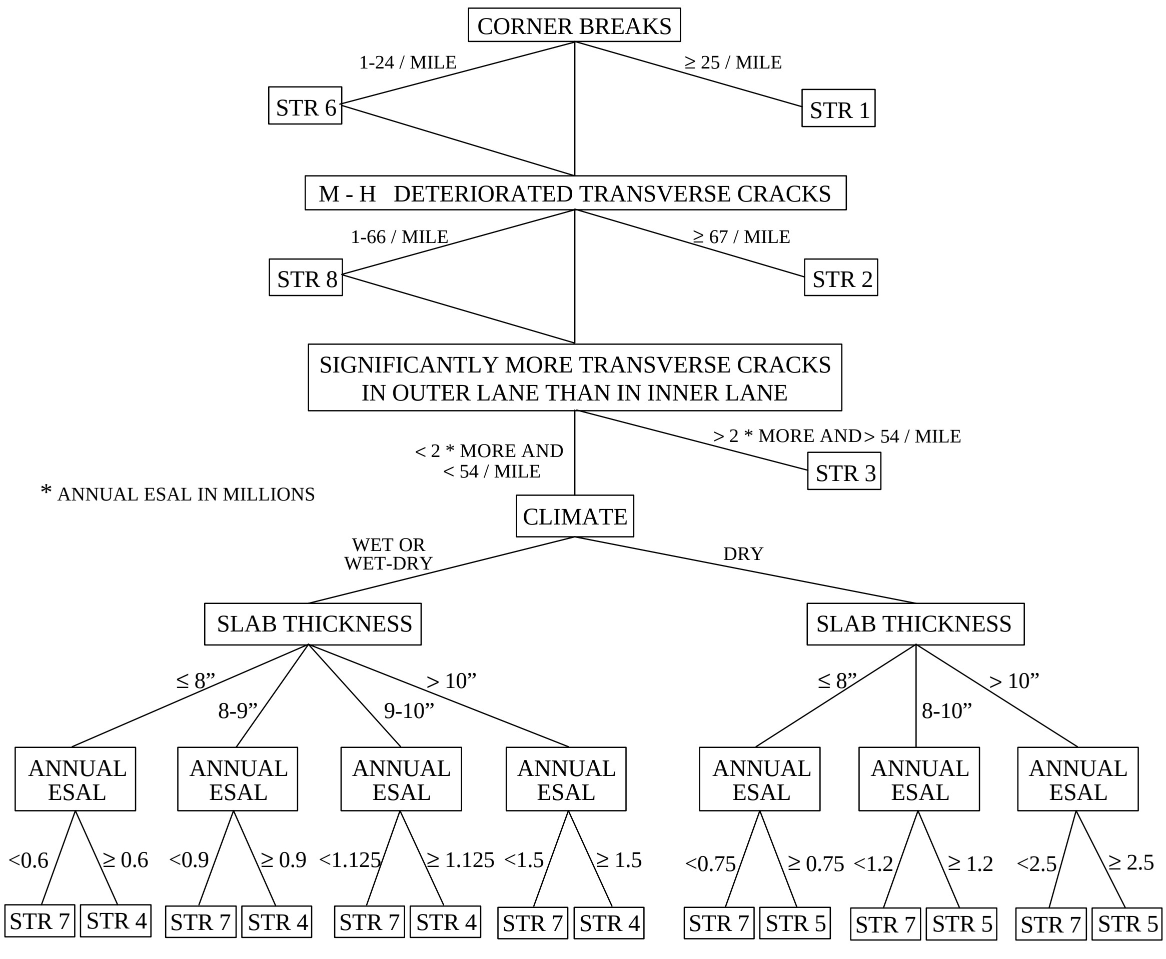
** Option to go to 1-1, 1-3, or 2-2 provided

1-1 Reconstruct Both Lanes
1-3 Reconstruct Outer, Restore Inner
3-1 Restore Outer, Reconstruct Inner
2-2 Overlay Both Lanes
3-3 Restore Both Lanes

Figure 4-15.2. Example of a decision tree to select a main rehabilitation approach for JPCP.[2]

Table 4-15.2. Relationship between decision tree codes, structural deterioration, and feasible repairs.[2]

Structural Code	Description	Feasible Repairs
STR 1	Structural deficiency of the pavement indicated by 15 or more corner breaks per km	a) full-depth repair of corner breaks, HMA overlay b) full-depth repair of corner breaks, crack and seat and HMA overlay c) full-depth repair of corner breaks, bonded overlay d) full-depth repair of corner breaks, unbonded overlay e) reconstruct
STR 2	150 or more meters of deteriorated transverse cracks per km	a) full-depth repair of cracks, HMA overlay b) full-depth repair of cracks, crack and seat and HMA overlay c) full-depth repair of cracks, bonded overlay d) full-depth repair of cracks, unbonded overlay e) reconstruct
STR 3	Significantly more transverse crack deterioration than in the next inner lane	a) full-depth repair of cracks, HMA overlay b) full-depth repair of cracks, crack and seat and HMA overlay c) full-depth repair of cracks, bonded overlay d) full-depth repair of cracks, unbonded overlay e) reconstruct
STR 4	Wet or wet-dry climate, specified slab thickness and ESAL levels	a) HMA overlay b) crack and seat and HMA overlay c) bonded overlay d) unbonded overlay
STR 5	Dry climate, specified slab thickness and ESAL levels	a) HMA overlay b) crack and seat and HMA overlay c) bonded overlay d) unbonded overlay
STR 6	Between 1 and 15 corner breaks per km	a) full-depth repair of corner breaks
STR 7	No indication of structural deficiency	a) do nothing
STR 8	Between 1 and 150 m of deteriorated transverse cracks per km	a) full-depth repair of cracks



1 in = 25.4 mm
1 mi = 1.609 km

Figure 4-15.3. Example decision tree to evaluate structural deficiency of JPCP.[2]

Table 4-15.3. Decision table for repair of concrete pavement distresses (adapted from reference 3).

Observed Distress	Associated Distresses	Appropriate Strategies
Faulting	Loss of sealant, subgrade erosion, shoulder deterioration along the slab	Temporary: diamond grinding, resurfacing with HMA overlay Durable: load transfer restoration, slab stabilization Both: reseal joints
	Cracked slabs, shoulder deterioration	Temporary: HMA overlay Durable: Reconstruction or overlay
	Cracked slabs	Temporary: HMA overlay Durable: Reconstruction or overlay
Joint Sealant Failure	Non-existent or poor slab-base drainage	Reseal joints and maintain or retrofit edge drains
	Same, with spalling	Spalls less than 50 mm: reseal Spalls greater than 50 mm: repair spall Both: maintain or retrofit edge drains
Surficial Cracking *(appearing soon after construction)*	Entire surface worn Sometimes minor rutting in wheelpath	General surface restoration techniques such as surface treatment or HMA overlay
	Roughness associated with concrete placement difficulties	Fill with resin
Transverse Cracking *(appearing soon after construction)*	Within 1.5 m from transverse joint	Full depth repair
	More than 1.5 m from transverse joint	Convert crack to joint, create reservoir, and seal
Transverse Cracking *(appearing during service)*	Associated with weak concrete, insufficient thickness: sealant failure, faulting	Reconstruction or overlay
	Associated with late joint sawing: sealant failure, faulting	Full-depth repair, reconstruction, or overlay of affected area
	Associated with poor support: sealant failure, faulting	Evaluate support conditions further before selecting strategy
	Associated with fatigue, end of life: sealant failure, faulting	Reconstruction or overlay
	Associated with fatigue, poor support: sealant failure, faulting	Slab reconstruction and restore support conditions
	Associated with fatigue, erosion of subgrade: sealant failure, faulting	Reconstruction or overlay

Table 4-15.3. Decision table for repair of concrete pavement distresses (adapted from reference 3) (continued).

Observed Distress	Associated Distresses	Appropriate Strategies
Longitudinal Cracking *(appearing during construction)*	Less than 1.5 m from longitudinal joint without transverse cracking: poor sealing	Convert to a joint, make a reservoir, and seal
	Greater than 1.5 m from longitudinal joint without transverse cracking: poor sealing	Convert to a joint, make a reservoir, and seal
	With transverse cracking: poor sealing	Full-depth repair
Longitudinal Cracking *(appearing during service)*	Associated with weak concrete, insufficient thickness: sealant failure, lane-lane faulting	Reconstruction or overlay
	Slabs too wide, late sawing: sealant failure, lane-lane faulting	Full-depth repair, slab replacement, reconstruction, or overlay
	Localized depression, settlement: sealant failure, lane-lane faulting	Define repair after additional investigation
	End of service life: sealant failure, lane-lane faulting	Full-depth repair, slab replacement, reconstruction, or overlay
Corner Break	Sealant failure, spalling	Full-depth repair, slab replacement, reconstruction, or overlay
Multiple Cracks *(shattered slab)*	Sealant failure, spalling	Reconstruction or overlay

5. EXAMPLE APPLICATION

To further explore the use of the decision-making tools presented in this module, consider the following example:

Location and Site Conditions

- Urban interstate highway located in the southwestern United States.
- Six traffic lanes (three in each direction); project is 11.7 km long.
- Current two-way traffic volume is 45,000 vehicles per day.
- Climate has occasional freezing; rainfall averages 75 mm per year (SHRP climatic zone III-A).

Pavement Construction History

- 254-mm JPCP, 14 years old.
- 100-mm dense-graded aggregate base.
- 4.6-m skewed transverse joint spacing, no dowels; joints sealed at construction with hot-poured bituminous material.
- AC shoulder.

Pavement Performance

The performance of this pavement is similar in both directions. The outer lane of this project averages 11 corner breaks per km. There are some transverse cracks as well, about 74 m/km of low severity. Faulting in this lane averages 2 mm, and there are some signs of pumping, consisting of slight deterioration of the asphalt shoulder adjacent to the transverse joints. The middle lane exhibits lower levels of deterioration, with 2 corner breaks per km and 35 m/km of low severity transverse cracks. Faulting in the middle lane averages 1.6 mm. The inner lane has no corner breaks, 16 m/km of low severity transverse cracking, and average faulting of 1.2 mm.

Proposed Rehabilitation from Decision Trees

Using table 4-15.2, the project may fall into the STR 3, STR 6, and STR 8 categories. Appropriate rehabilitation strategies range from full-depth repairs to HMA overlays, to fractured slab techniques with an HMA overlay. The decision tree in figure 4-15.2 also suggests that an overlay is appropriate. Using figure 4-15.3 indicates that the most heavily trafficked lane is in the STR 3 category and the other two lanes are in the STR 8 category. This would still make full-depth repair the most likely rehabilitation recommendation.

Using Decision Tables

Applying decision tables to the same project gives slightly different results. Table 4-15.1 provides an indication of all of the treatments that might be appropriate. The distresses that are noted are pumping, faulting, and slab cracking, and the treatments cover a range of options. Table 4-15.3 suggests that for a pavement with some faulting, cracked slabs, and shoulder deterioration, a durable rehabilitation is reconstruction or an overlay. For the transverse cracking, the recommended strategy depends on the cause of the cracking: if the pavement is of insufficient thickness, poor support, or fatigue, reconstruction or overlays are the appropriate remedy. For the corner breaks these are also recommended, along with full-depth repair and slab replacement.

6. LIMITATIONS OF DECISION TREES

Module 3-11 covers some of the limitations associated with the use of decision trees or decision tables to select rehabilitation strategies and that information need not be repeated in detail here. Two key points of that discussion are that:

1. Decision trees need to be continually updated so that new strategies, materials, and agency constraints are reflected in the available options.
2. Decision trees should be considered as an aid in selecting treatments, not as strict rules.

As can be seen from an examination of the decision trees and decision tables presented in this module, and their application to the simple design example, the project planner or engineer will always be left with a large amount of engineering to perform. Pavements rarely exhibit only a single distress, so that the problem is not to find the repair that addresses a single problem, but to find the strategy that addresses all of the problems and is cost effective as well. Decision trees/tables do provide some recommended solutions, but their use still requires engineering judgment and identification of the underlying causes of the distress.

Few engineers should be comfortable following a decision tree or table from the first input to its final outcome and blithely accepting the outcome. In fact, in the trees and tables presented in this module there is no one single outcome that results from using these tools. It is still up to the engineer to select alternatives that fit actual rather than idealized conditions. Furthermore, as noted earlier, this approach offers no opportunity for innovation or identifying new techniques that may address old problems. Alternative approaches, such as value engineering, must be used if the objective is to identify new and better repair methods.

7. SUMMARY

There are a number of tools that may be used to identify feasible rehabilitation alternatives for rigid pavements. At a very simple level, a single distress is matched with one or more repairs that correct the distress. An example of this is shown in table 4-15.1. However, the range of possible treatments shown for each distress suggest that the problem is more difficult. What are the extent and severity of the distress? What is it caused by? How long does the rehabilitation need to last? The answer to these questions will help to determine what is actually done.

Other tools that may be used to identify feasible rehabilitation alternatives include decision trees and decision tables, such as those shown in figure 4-15.3 and table 4-15.3. These approaches consider more detailed information, and can also be developed to work in combinations of distresses or combinations of distresses and different severity levels.

Ultimately, however, the identification of feasible alternatives will likely involve a number of approaches. There is no substitute for engineering judgment, which works into the decision-making process experience from previous projects, information from a pavement management database, and a knowledge of current research that is not usually found in decision trees. A value engineering approach, in which teams are encouraged to consider new ideas and solutions to old problems, is also a useful tool. The process must also consider the constraints that affect the project, such as budget, available time, ability to handle traffic during construction, projected life of the repair, desired life of the pavement, and so on.

8. REFERENCES

1. American Concrete Pavement Association (ACPA). 1993, "Pavement Rehabilitation Strategy Selection," Technical Bulletin TB—015.P, American Concrete Pavement Association, Skokie, IL.

2. Hall, K. T., Connor, J. M., Darter, M. I., Carpenter, S. H. 1989, "Rehabilitation of Concrete Pavements Volume III — Concrete Pavement Evaluation and Rehabilitation System," FHWA-RD-88-073, Federal Highway Administration, Washington, DC.

3. Permanent International Association of Road Congresses (PIARC). 1992, "Evaluation and Maintenance of Concrete Pavements," Permanent International Association of Road Congresses, Paris, France.

BLOCK 5

SELECTION OF PREFERRED REHABILITATION ALTERNATIVES

Throughout this course, participants have become familiar with the various aspects of rehabilitation design. These include project survey and evaluation procedures (block 2) and the different rehabilitation alternatives that are available for a particular project (blocks 3 and 4). The next steps in the process are the identification of feasible rehabilitation alternatives and the selection of the preferred rehabilitation alternative, or the one that best addresses the needs and constraints of the project. This block provides additional information on the development and evaluation of various rehabilitation design alternatives that may be appropriate for a given project. Factors to be considered in the ultimate selection of the preferred rehabilitation alternative are discussed, including both monetary and nonmonetary issues.

MODULE 5-1

SELECTION OF THE PREFERRED REHABILITATION ALTERNATIVE

Throughout this course, participants have become familiar with the various aspects of rehabilitation design. These include project survey and evaluation procedures (block 2) and the different rehabilitation alternatives that are available for a particular project (block 3, block 4, and block 5). The next steps in the process are the identification of feasible rehabilitation alternatives and the selection of the preferred rehabilitation alternative, or the one that best addresses the needs and constraints of the project. This block provides additional information on the development and evaluation of various rehabilitation design alternatives that may be appropriate for a given project. Factors to be considered in the ultimate selection of the preferred rehabilitation alternative are discussed, including both monetary and non-monetary issues.

1. INSTRUCTIONAL OBJECTIVES

In the previous modules, considerations for the design and use of various rehabilitation alternatives are discussed. However, little regard is given to the selection of the most appropriate rehabilitation alternative when several alternatives may adequately address the problems of the pavement. The selection of the preferred rehabilitation alternative for a given pavement section requires a systematic, step-by-step approach that considers all relevant factors. This module outlines the major steps and procedures in this process. The participants will be able to accomplish the following upon successful completion of this module:

1. Describe the importance of developing alternative rehabilitation designs.
2. List the eight major steps required for rehabilitation selection and design.
3. Describe techniques that can be used to assist in the development of cost-effective alternative rehabilitation designs.
4. List and briefly describe the major factors that should be used in deciding among alternative rehabilitation designs.
5. List the benefits of conducting a life-cycle cost analysis to determine the total cost of various pavement rehabilitation alternatives.
6. List major cost items related to both the highway agency and the users (motorists) that should be considered in a life-cycle economic analysis.
7. Conduct a life-cycle cost analysis for a rehabilitation project alternative.
8. Describe systematic procedures for selecting the preferred alternative when there are several feasible alternatives available.
9. Conduct an evaluation of two or more rehabilitation alternatives for a project and select the preferred design alternative.

2. INTRODUCTION

In recent years, the emphasis of highway agencies has gradually shifted from new design and construction activities to maintenance and rehabilitation of the existing network.[1] The need to maintain the already constructed network is essential to the economical operation of the overall transportation system. Funding constraints, however, make cost-effective maintenance and rehabilitation strategies even more important.

Historically, overlays have been the most common rehabilitation technique utilized.[2] However, these overlays have sometimes been constructed without regard to their applicability or to their cost-effectiveness. In many cases, it may have been more practical or cost-effective to perform other types of rehabilitation or to have performed earlier routine maintenance on the pavement.

To assist the engineer in selecting both the type and timing of pavement maintenance or rehabilitation, pavement performance must be systematically measured on a continuing basis. In this performance evaluation, both the functional and the structural performance of the pavement system should be considered. Functional performance, as defined in module 2-2, describes the adequacy of the pavement to meet its basic purpose of providing a safe and smooth riding surface. Functional adequacy is usually measured in terms of roughness and surface friction. Structural performance is related to the ability of the pavement to sustain traffic loading, and is the primary input to all rational overlay design methods. Deflection testing is usually performed to assess structural adequacy.

Although functional and structural performance are intuitively related, there is currently no well-defined relationship between structural distress and functional performance. Thus, at present, engineering judgment must be used in deciding when structural deterioration will lead to a level of functional performance below that considered reasonable by the users of the facility which obviously will vary among vehicles, users, and types of facilities. In many cases structural distress (fatigue cracking) has progressed to almost complete failure before any functional deterioration (roughness) can be measured. In these cases it is often much more cost-effective to rehabilitate the pavement at early stages of structural deterioration. Waiting for the pavement to exhibit significant functional distress usually results in dealing with extensive structural failure and much more extensive and costly rehabilitation treatments.

3. REHABILITATION ALTERNATIVES

There are numerous pavement rehabilitation alternatives available to the design engineer. Among some of the major rehabilitation techniques are:

- Overlay (asphalt or concrete).
- Full depth repairs.
- Partial depth repairs.
- Joint and crack sealing.
- Slab stabilization.
- Diamond grinding and milling.
- Subdrainage.
- Surface treatment (chip seal, fog seal, thin overlay, and so on).
- Recycling.

Depending upon the timing of the rehabilitation and the condition of the pavement, the designer may also wish to consider preventive maintenance treatments as an alternative to full scale rehabilitation.

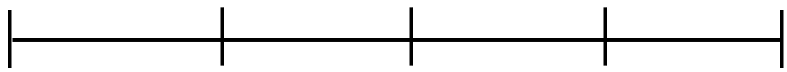
The “preferred” rehabilitation strategy for a given project 1) must be cost-effective, 2) must address the specific problems of the pavement, and 3) must meet any existing constraints of the project. The procedure for determining the preferred rehabilitation strategy is a complex one and will entail as much engineering judgment as engineering analysis. Compared to new pavement design, rehabilitation may require much more engineering judgment and analysis. The basic guidelines that should be followed in the determination of the appropriate rehabilitation strategy are discussed later.

4. TIMING OF REHABILITATION ALTERNATIVES

Any pavement section in a network has associated costs for maintenance and rehabilitation that occur at different points in time. Figure 5-1.1 illustrates the life-cycle costs (LCC, in terms of “present worth” [PW] costs or as an “equivalent uniform annual cost” [EUAC]) for two similar, adjacent pavement sections that are subject to the same traffic and climate, but have different rehabilitation strategies). There is a substantial difference in the costs to keep these pavements in service, which illustrates the fact that rehabilitation strategies can have a great effect on the LCC of pavements. If an agency manages each pavement section in its network using sound rehabilitation techniques and LCC, the overall condition of the network will be substantially improved for a given funding level.

Alternate A: Reconstruct until failure and then reconstruct again.

Expense	$850,000		$150,000		



Year	0	7.5	15	22.5	30

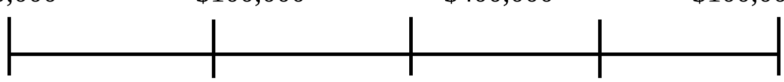
Total Additive Cost = 850,000 + 150,000 = **<u>$l,000,000</u>**

Present Worth = 850,000 + 150,000 (0.5555) = **<u>$933,250</u>**

Equivalent Uniform Annual Cost (EUAC) = 933,250 (0.0578) = **<u>$53,942 per year</u>**

Alternate B: Restore or resurface periodically before pavement reaches “failure” condition.

Expense	$400,000	$100,000	$400,000	$100,000



Year	0	7.5	15	22.5	30

Total Additive Cost = 410,000 + 100,000 + 400,000 + 100,000 = **<u>$1,000,000</u>**

Present Worth = 400,000 + 100,000 (0.676) + 400,000 (0.456) + 100,000 (0.414) = **<u>$691,400</u>**

Equivalent Uniform Annual Cost (EUAC) = 691,400 (0.0578) = **<u>$39,963 per year</u>**

Figure 5-1.1. Illustration of long-term costs for two different rehabilitation treatments for similar pavement sections (discount rate = 4 percent).

There is always more than one alternative rehabilitation design available for a given project. Each alternative has its own associated costs, constructability, performance life, reliability, maintainability, and other unique characteristics. It is desirable to select the preferred alternative, or the one that meets all of the engineering criteria (e.g., project constraints, such as traffic control and initial funding) and is cost-effective. The preferred alternative does not necessarily imply "optimal," since the various constraints (e.g., available funds) may not permit optimization in the literal sense.

The complex process involved in selecting the preferred alternative is perhaps the most difficult and controversial part of the entire process of developing rehabilitation alternatives. It is necessary, therefore, that a rational, systematic, and defensible approach be adopted. The common practice of selecting an alternative only because it has always been done, or it has the lowest initial rehabilitation construction cost, is poor engineering practice, and can lead to much higher future pavement costs and management problems for an agency.

In this process of determining the preferred rehabilitation alternative for a given project, a single project is only one among many, and there is a limited amount of funding available for the network. Thus, the timing, general scope and funding level for the project has usually been already set considering the network needs and therefore the preset funding and scope limit the alternatives that can be considered for a project because of overall network budget constraints. Even in the case of limited available funds, however, it is still important to consider alternative pavement treatments, and their potential life-cycle costs.

5. DEVELOPMENT OF REHABILITATION ALTERNATIVES

The recommended approach to the development and selection of the preferred rehabilitation design is shown below and consists of eight steps:

STEP 1. Obtain available project information. This information should include all design data, traffic data, soil and material reports, pavement management performance data, cost data, and climatic information that is relevant to the project.

STEP 2. Establish existing condition of pavement. This requires a detailed evaluation of the existing pavement, and should include visual distress, roughness, and surface friction measurements, a drainage survey and analysis, nondestructive and destructive testing to characterize materials and soils properties, and a traffic loading analysis. The extent of deterioration along the project must be determined.

STEP 3. Determine the causes of distress. This step requires a thorough evaluation of the data collected under steps 1 and 2 and provides and indication of the causes of the pavement deterioration. This aspect cannot be overemphasized. Successful pavement rehabilitation design requires an in-depth identification of the causes and extent of existing pavement so that appropriate rehabilitation alternatives can be considered.

STEP 4. Develop feasible alternatives. Based upon the results of step 2 and step 3, several preliminary alternative designs can be developed that address the causes of the existing deterioration and prevent their recurrence. For example, if pumping has significantly contributed to the damage, techniques to repair the damage it has created (e.g., slab stabilization) and techniques to prevent or reduce its future occurrence (e.g., joint resealing or subdrainage installation) must be included. Feasible alternatives must include both repair and preventive techniques. Examples of candidate repair and preventive methods for asphalt concrete pavements are provided in table 5-1.1. Candidate repair and preventive methods for portland cement concrete (PCC) pavements are given in table 5-1.2. Feasibility guidelines for the use of various rehabilitation alternatives for PCC pavements are presented in reference 3.

Another important concept is to apply rehabilitation work only to those areas that show signs of distress. For example, if only 4.8 km of a 9.7 km project are distressed, rehabilitation measures should be targeted for those 4.8 km only.

Table 5-1.1. Candidate repair and preventive methods for distresses in AC pavements.

DISTRESS	REPAIR METHODS	PREVENTIVE METHODS
Fatigue Cracking	1. Full depth repair 2. HMA overlay 3. Recycle and overlay	1. Crack sealing 2. Chip seal 3. HMA overlay
Bleeding	1. Apply hot sand 2. Open-graded overlay	
Block Cracking	1. Seal cracks 2. Double chip seal 3. HMA overlay	1. Chip seal
Depression	1. Level up overlay	
Polished Aggregate	1. Skid resistant surface treatment	
Potholes	1. Full or partial depth repair	1. Crack sealing 2. Surface treatment
Pumping Fines	1. Full depth repair	1. Crack sealing 2. Surface treatment 3. Subdrainage improvement
Raveling and Weathering	1. Seal coat 2. Thin HMA overlay	1. Rejuvenating seal
Rutting	1. Level up overlay 2. Cold milling with or without overlay	(Depends on cause)
Swell	1. Removal and replacement 2. Level up overlay	1. Paved shoulder encapsulation

Table 5-1.2. Candidate repair and preventive methods for distresses in PCC pavements.

DISTRESS	REPAIR METHODS	PREVENTIVE METHODS
Pumping	1. Slab stabilization	1. Reseal joints 2. Restore load transfer 3. Improve pavement drainage 4. Tied PCC shoulder
Faulting	1. Grind surface 2. Structural overlay	1. Slab stabilization 2. Reseal joints 3. Restore load transfer 4. Improve pavement drainage 5. Tied PCC shoulder
Slab Cracking	1. Full depth repair 2. Replace/recycle lane	1. Slab stabilization 2. Restore load transfer 3. Structural overlay
Joint or Crack Spalling	1. Full depth repair	1. Clean and seal joints/cracks 2. Restore Load transfer 3. Structural overlay
Blowup	1. Full depth repair treatment	1. Pressure relief joints 2. Clean and seal joints/cracks
Punchouts	1. Full depth repair	1. Tied PCC shoulder 2. Slab stabilization

There are four major types of alternatives that could be considered on any rehabilitation project provided there are sufficient funds:

a. *Restoration*. The work required to return the existing pavement structure to a suitable condition to perform satisfactorily, without the immediate placement of an overlay.
b. *Recycling*. The reuse of surfaces, bases, or subbases to improve structural and durability integrity. New material is also commonly added to existing materials to improve their strength and durability performance.
c. *Resurfacing*. The addition of paving layers to provide additional structure or improved serviceability.
d. *Reconstruction*. The complete removal and replacement of either the entire pavement section, or a major portion of it with a new design.

There are, of course, many design variations and combinations within each of these four main types that could be applicable within any given project.

STEP 5. Conduct engineering and economic analyses. All decision criteria, both monetary and non monetary, that will be used in selecting the preferred alternative must be identified. Engineering factors include such items as traffic control options, time of lane closures, material and equipment availability, and prevailing climatic conditions. Monetary factors include direct costs to the agency and to the user. A life-cycle cost analysis of each alternative then should be conducted that considers all relevant costs.

STEP 6. Select the preferred rehabilitation alternative. Each rehabilitation alternative must be evaluated with respect to the selected decision criteria, and the preferred alternative must be selected considering all important decision factors before being recommended to management.

STEP 7. Design the preferred rehabilitation alternative. A detailed design must be developed for the selected rehabilitation alternative, including all required plans, specifications, and estimates.

STEP 8. Make follow up reviews of pavement performance. Follow up performance reviews are essential to the process so that deficiencies can be identified and corrected in the next rehabilitation project.

The concepts and methodology used in value engineering (VE) are very applicable to the development of cost-effective alternatives in the rehabilitation pavement program. Value engineering may be defined as the systematic application of recognized techniques that identify the function of a product or service, establish a value for that function, and provide the necessary function reliably at the least overall cost.(4) It is pointed out that value engineering is not typical "cost reduction," in that it does not "cheapen" the product or service, nor does it "cut corners."(4) Reference 4 states that "value engineering is an important management tool to optimize expenditures for highway and transportation facilities."

For example, various phases of the value engineering approach recommend the generation of numerous alternative means for accomplishing the project, selecting the most promising alternatives, developing the selected alternatives, and conducting a life-cycle cost analysis of the alternatives. The FHWA course notebook on value engineering presents many good ideas on how to develop alternatives, and it should be consulted for additional information (see reference 5).

Some recommended VE techniques for identifying and developing rehabilitation alternatives are:(5)

1. Appointment of a design review committee made up of key people from planning, design, construction, traffic operations, standards, maintenance, and purchasing to assure all aspects of the problem are considered.
2. Solicitations for intradepartment or interdepartment suggestions.
3. Solicitations for ideas from contractors, material suppliers, and others involved in construction.
4. Brainstorming by either individuals or committees.
5. Review of previous studies conducted by the agency and by other agencies.

Perhaps the greatest obstacle to the development of cost-effective alternative rehabilitation designs is the process of "habitual thinking," as illustrated in the following quote.(5)

> *Thinking and doing things in the same way each time a problem occurs is a frequent cause of poor value. Most people have a natural tendency to reuse what worked the last time or copy the standard set by others. This is a defensive measure designed to minimize risk to security. The habit-forming process is promoted by management through rigid use of standard designs, procedures, custom and tradition without consideration of changing function, technology or value. It is said that habits take us where we were yesterday, but attitudes keep us there.*

> *Keeping up the state-of-the-art is essential in today's complex era. Reluctance to change because of yesterday's good experience is an attitude which inhibits improvement and discourages progress. Minds must be opened and fresh ideas incorporated.*

Another relevant quote comes from Dr. E. de Bono's renowned book on thinking:[6]

> *Contentment with an "adequate" solution or approach is the biggest block there is to any search for a better alternative. We are very happy with what we have because we cannot conceive of anything better and until we can conceive of something better we are not motivated to look for it.*

The design engineer should make strong efforts to identify all feasible alternatives no matter how unlikely one of them may appear. A feasible alternative is one that addresses the problems of the pavement and that fits within the identified constraints (e.g., geometrics, construction time, traffic flow conditions, clearances, right-of-way, and funding). The more constraints there are, however, the fewer the alternatives that can be considered.

The practice of applying "standard designs" to all projects (e.g., use of 51 mm overlays) often results in an inefficient design. Customized engineering design and consideration of various alternatives for individual projects are needed to ensure the best use of limited funds.

6. SELECTION OF THE PREFERRED ALTERNATIVE DESIGN

Once several distinct feasible alternatives have been developed, they must be evaluated and the preferred rehabilitation alternative selected. There is no absolute and indisputable method for selecting the preferred rehabilitation alternative for a given project. A considerable amount of professional engineering judgment must be applied to each project.

Also, the preferred alternative must fit in with the overall management of the pavement network. Funding for any given project is normally set a year or more in advance and usually cannot be significantly increased. The rehabilitation alternative that has the lowest total life-cycle cost for a given project may not coincide with the best interests of the entire pavement network, considering the limited availability of funds.

Overriding Factors

The preferred alternative does not necessarily imply "optimal," since the various constraints (e.g., available funds) may limit optimization of each project in favor of optimization at the network level. Rather, the preferred alternative will be the one that best addresses the needs of the pavement while meeting all functional and monitory constraints that exist. Examples of the constraints that may affect the selection of alternatives are listed below:[1]

- Limited project funding and scope.
- Provisional (staged) construction.
- Traffic control requirements.
- Minimum desirable life of rehabilitation.
- Future maintenance requirements.
- Geometric design problems and constraints.
- Present and future utilities.
- Right-of-way restrictions.
- Regulatory restrictions.

- Available materials and equipment.
- Contractor expertise and manpower.
- Agency policies.

These factors must be considered in the selection of the preferred rehabilitation strategy. In addition, consideration should be given to how a rehabilitation strategy may affect the network as a whole. It may occasionally be necessary to select an alternative that is not optimal for a project because of overall network constraints.

Life-Cycle Cost Analysis

While many agencies base their rehabilitation selection on the initial cost of the various alternatives, life-cycle costs should be one of the key considerations in the selection process. Life-cycle costs ideally incorporate all costs that an alternative will accumulate over its performance period, and can be categorized as:

1. Costs to the highway agency.
   - Initial rehabilitation construction.
   - Future maintenance and rehabilitation.
   - Future salvage value.

2. Costs to the highway user.
   - Traffic delay costs.
   - Vehicle operation costs.
   - Accident costs.
   - Discomfort costs.

All of these costs are difficult to estimate for a given rehabilitation alternative. The highway user costs are perhaps the most difficult to estimate at the present time. There are some research data and analysis procedures available that can be utilized to estimate several of these user costs.[7,10]

Life-cycle costs are usually expressed in terms of a PW cost or an EUAC.[11,12] 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. The EUAC method expresses present and future costs in terms of an equalized, annual payment using a selected discount rate. The equations for the determination of these costs are given below:

$$PW = C \times \frac{1}{(1+i)^n} \tag{5 1.1}$$

$$EUAC = PW \times \frac{i}{[1-(1+i)^n]} \tag{5 1.2}$$

where:

PW	=	Present worth of future costs, $
C	=	Future cost at time t = n, $
i	=	discount rate, expressed as a decimal
n	=	time at which future cost incurred; also analysis period, years
EUAC	=	Equivalent uniform annual cost

An example of the PW and EUAC cost calculations is provided in equation 5-1.2. It is important to be aware of the fact that the PW method requires that all alternatives be considered over the same analysis period. The EUAC method may be used when the analysis periods of the alternatives differ.

Additional information on conducting life-cycle cost analyses is found in reference 8 and in reference 13, reference 14, reference 15, and reference 16. The NCHRP synthesis, *Life-Cycle Cost Analysis of Pavements*, is probably the most comprehensive document available and is recommended for further information.[14] The FHWA also conducted a comprehensive Life-Cycle Cost Symposium and the primary document from that symposium "Searching for Solutions A Policy Discussion Series Number 12 Life-Cycle Cost Analysis" is also very informative.[24]

An excellent example of life-cycle cost analysis is also given in reference 16. Figure 5-1.2 and figure 5-1.3 are from that reference and illustrate a life-cycle cost analysis for a specific project.

Items Considered in a Life-Cycle Cost Analysis

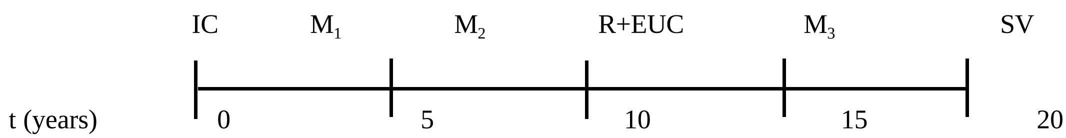
<u>Analysis Period</u>

The analysis period refers to the time over which the economic analysis is to be conducted. 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.[1] For rehabilitation work, the analysis period will usually be shorter, such as 10 to 20 or more years, depending on the future use of the facility, the need for geometric improvements, and other factors. An analysis period of at least 10 years is recommended so that future costs of competing strategies can be reasonably considered.

Overall, it is best to compare all alternatives over the same analysis period. If the analysis period is set to 15 years, for example, and the maximum initial life that one rehabilitation alternative can provide is 10 years, another rehabilitation project would have to be applied at 10 years into the future, so that the costs at 15 years can be calculated. If an alternative life exceeds the analysis period, then a salvage value can be considered so that a fair comparison can be made between alternatives.

LCCA COMPUTATION EXAMPLE

Given: Pavement project with the following cash flows.



IC = Initial Construction Cost = $120,000 (t=0)
M_i = Maintenance costs at different years
M_1 = $10,000 (t=4) M_2 = $12,000 (t=7)
M_3 = $25,000 (t=14) M_4 = $20,000 (t=17)
R = Rehabilitation Cost = $90,000 (t=10)
EUC = Extra Users Cost = $30,000 (t=10)
SV = Salvage Value = $50,000 (t=20)

Determine: PW and EUAC.

Assume: Analysis Period = 20 years (the life span over which this project is evaluated, is 20 years; note that the analysis period must be the same for each alternative).

Average Annual Discount Rate = 5 percent (assumed to be the difference between the market interest rate and construction inflation rate).

Analysis:

Summary information regarding the costs is presented in the following table. The PW of each cost term is calculated using equation 5 1.1 (multiplier term is shown in the table). Application of this multiplier brings the amount of each cost term back to time t=0, or its present worth.

Cash Flow Symbol	**Symbol Definition**	**Cash Flow Amount, $**	**n**	$\frac{1}{(1+i)^n}$	**PW of Cash Flow,$**
IC	Initial Construction Cost	120,000	0	1	120,000
M_1	Maintenance Costs	10,000	4	0.8227	8,227
M_2	Maintenance Costs	12,000	7	0.7107	8,528
R	Rehabilitation Costs	90,000	10	0.6139	55,252
EUC	Extra User Costs due to Rehab	30,000	10	0.6139	18,417
M_3	Maintenance Costs	25,000	14	0.5051	12,627
M_4	Maintenance Costs	20,000	17	0.4363	8,726
SV	Salvage Value	50,000	20	0.3769	18,844
				TOTAL PW	**$212,933**

Therefore, the total present worth cost is $212,933. The EUAC is determined by applying equation 5 1.2:

$$\text{EUAC} = 212{,}933 \ \frac{0.05}{1\ \ (1.05)^{20}} = \$17{,}086/\text{year}$$

Figure 5-1.2. LCCA Computation Table.

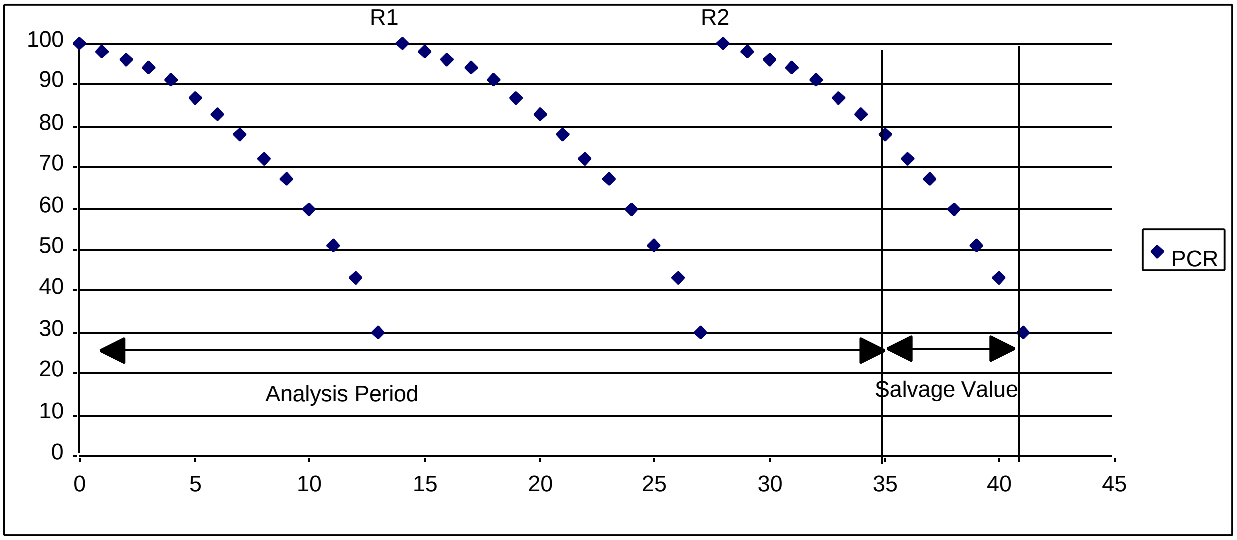


Figure 5-1.3. Pavement performance graph showing salvage value as remaining life of last treatment.

<u>Rehabilitation Alternative Performance Period</u>

The performance period of a rehabilitation alternative refers to the time between rehabilitation construction and the next major rehabilitation work. For example, hot-mix asphalt (HMA) overlays are usually designed for a 8- to 15-year period, and this becomes the rehabilitation performance period until another rehabilitation project is needed. Information on the expected performance periods of other pavement rehabilitation techniques is given in references 8, 13, 17, and 18.

Several States already have performance databases that are providing some information, allowing for more informed design and rehabilitation decisions to be made. Presently, professional engineering judgment is required, along with analysis of the effects of traffic loadings, to obtain an estimate of the life of various rehabilitation alternatives.

<u>Initial Costs</u>

Initial costs are the total costs to construct the alternative being considered. Initial costs may include design costs as well as all construction costs. If the design costs of each alternative are approximately the same, then those costs can be ignored and only construction costs should be considered. Sources of initial costs include previous projects, previous bids, and other historical data, but should reflect the most current data available.[14] When new materials or techniques are being considered where historical information is not available, great care should be taken in generating the costs for those items.[14]

<u>Future Maintenance and Rehabilitation Costs</u>

All future maintenance and rehabilitation actions and costs to maintain the pavement in serviceable condition throughout the analysis period must be estimated for each design alternative. The proper selection of future treatments and maintenance actions should be based on sound engineering practice backed up with information from agency records. Previous experience, if available from similar projects, can be used as a guideline. However, the estimates need to be modified for any differences (e.g., highway class, traffic levels) that may exist between the proposed rehabilitation alternative and the projects from which the experience was derived. For example, an experienced engineer may be fairly certain that a typical AC pavement design will require maintenance starting 5 years after construction, followed by a nominal 50 mm overlay in 10 years and each succeeding 10 years. Such information could be used in estimating LCC for a similar, new project, with corrections perhaps being made to account for future increases in heavy traffic.

Both maintenance and rehabilitation costs are directly influenced by how well the pavement performs, which, in itself, is sometimes difficult to estimate. Furthermore, the timing of the future maintenance or rehabilitation activity can have a large impact on the magnitude of its cost, since maintenance and rehabilitation costs increase with decreasing pavement condition.[14] Also, the pavement type, the type and magnitude of the pavement loads, and the specific environment play a large part in determining the future maintenance and rehabilitation needs of a specific project. Data from pavement management data on pavement performance and anticipated loads help support this information.

<u>User Costs</u>

User costs are costs imparted to the user due to the condition of the roadway between rehabilitation treatments and during construction of the roadway. Items generally considered as part of user costs include:

- Traffic delay costs (costs and inconvenience of traffic rerouting, traffic control, and delays).
- Vehicle operating costs (a vehicle traveling on a rough road will suffer more wear and will consume more fuel).
- Accident costs (construction zones and rough roads increase the potential for accidents).
- Discomfort and medical costs (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 impart a cost to the user, the actual cost has been difficult to quantify particularly for road roughness. Traffic delay and user costs are sometimes considered as one of the engineering criteria, instead of being included in the cost analysis for the comparison of alternatives and selection of the preferred alternative. Traffic delay costs are sometimes determined and included in the life-cycle cost analysis (LCCA) as an additional computation which provides an indication of the effect the construction of each alternate has on the traveling public.[25]

One example of computing traffic delay costs comes from California. In an unpublished study in 1986, the California Department of Transportation determined that the average value (in 1986) of time for general freeway traffic was $6.25 per vehicle-hour of delay. Vehicle delay costs are then determined by computing the amount of time lost due to speed reductions in a construction. The user delay costs were computed as follows:

$$UC = (\$6.25)(L/RS - L/NS)(ADT)(PT)(CP)$$

Where UC = User Costs.
L = Project Length.
RS = Reduced Construction Speed.
NS = Normal Speed Before Construction.
PT = Ratio of Traffic Effected to Total Traffic During Construction Period.
CP = Construction Period.

As an example consider a 5 km project where the traveling speed has been reduced from 100 km per hour to 50 km per hour. The average daily traffic is 40,000 vehicles per day and the project lasts 100 days with both day and night traffic effected. The user cost is:

UC = ($6.25)(5/50-5/100)(50,000)(1.00)(100) = $1,562,500

Note the units cancel out to leave $.

Information on the estimation of user costs is provided in references 7, 8, 9, 10, 23, 24, and 25.

Salvage Value

Salvage value represents the worth of the pavement for each alternative at the end of the analysis period. Salvage value may be either a positive or negative value depending upon whether the pavement has some remaining service value or have become a liability that will add additional costs to the alternative at the end of the analysis period to make the pavement serviceable. 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 salvage value.[14]

Salvage value can be estimated as the value of the existing pavement materials at the end of the analysis period or as a percentage of the cost of the most recent rehabilitation treatment. It may be based on the percent of the pavement life remaining since the last rehabilitation treatment compared to the remaining life or on past experience or historical data. If it can be assumed that each alternative will have equal salvage value, then this cost factor can be neglected and need not be estimated. Salvage value differences between rehabilitation alternatives may not be substantial and the net difference after being discounted over 10 years to 20 years will further reduce the difference.

The concept of salvage value can be used to great advantage when a rehabilitation alternative has life that extends beyond the analysis period. Such an alternative may then have a higher salvage value than one that was badly deteriorated (little remaining life) at the end of the analysis period. What salvage value should be assigned depends on a realistic expectation of how the pavement will, in fact, be "salvaged" at the end of the analysis period: by recycling, rehabilitation, or through continued service.

Salvage value is commonly estimated by determining the ratio of the remaining life of the last treatment to the total life of that treatment and multiplying that ratio by the cost of the last treatment which is then considered the salvage value. In the example shown in figure 5-1.3, the salvage value would be 7/13 of the last treatment cost.

Discount Rate

The discount rate (which is the difference between commercial interest rate and inflation rate for business investments) represents the time value of money. It is usually expressed as an annual compounded rate that represents the rate of money interest will earn over a future period discounting inflation. The selection of the appropriate discount rate is critical to the selection of the preferred rehabilitation alternative since it can affect the outcome of a life-cycle cost analysis. The use of a low discount rate (2 percent to 3 percent) favors projects with large initial costs, whereas the use of a high discount rate (6 percent to 8 percent) favors projects that have lower initial costs but more future (maintenance or rehabilitation) costs.

One reasonable characterization of the discount rate defines it as the difference between the market rate of interest and the rate of inflation.[1,14,19] Future costs should be estimated in present dollars and kept constant in the future.

Using the difference between the rate of interest and the rate of inflation as the discount rate, Epps and Wootan recommend a real discount rate of 4 percent based on their determination that the real long-term rate of return on capital had been between 3.7 percent and 4.4 percent since 1966.[20] Oglesby and Hicks state that "the minimum rate for governmental investment should reflect the real cost of capital, which some have estimated as being in the range of 4 percent."[19]

It should be noted that the difference between interest rates and inflation rates does not remain constant over time and for that reason no specific discount rate that will always be correct. However, the selection of the appropriate rate should not be based on short-term economic conditions but should be based on the longer term average condition.

FHWA Policy

The Federal Highway Administration has developed a clear policy on the use and need for life-cycle cost analysis for projects on the National Highway System. The National Highway System Designation Act of 1995 directs the FHWA to establish a program that requires States to conduct life-cycle cost analysis of each high cost NHS project that exceeds $25 million. In response to that direction the FHWA has established the following policy.[23] For projects less than $25 million, the FHWA strongly encourages the States to use similar life -cycle cost analysis procedures.

> *This policy statement sets forth principles of good practice for the application of life-cycle cost analysis to highway and related infrastructure investment decisions. The FHWA fully supports and promotes sound economic analyses of highway investment alternatives that consider relevant coasts and benefits over the full life of the facility. States and local agencies are encouraged to follow these principles in evaluating highway investment alternatives. Alternative forms of LCCA are acceptable if they are consistent with principles of good practice contained in this statement.*
>
> *1. Life-cycle costs are important considerations along with budgetary, environmental, safety, and other factors in highway investment decisions. Investment alternatives having the least net cost (or the greatest net benefit) cannot be identified without considering streams of discounted benefits and costs over the entire life of the investment. Especially in periods of tight budgets, it is important to use life-cycle cost analysis, value engineering, and other appropriate techniques to maximize the return from investments of scarce highway resources. The importance of considering life-cycle costs in infrastructure investment decisions was emphasized in the President's Executive Order 12893, "Principles for Federal Infrastructure Investments."*
>
> *2. Life-cycle cost analysis principles involving the systematic evaluation of costs and benefits over the life of highway improvements have been utilized in benefit cost analysis, cost-effectiveness analysis, and other economic analysis techniques for many years. Continued use of these principles can help reduce costs of providing essential highway services that stimulate our economy and enhance our quality of life.*
>
> *3. Life-cycle costs should be considered in all phases of construction, maintenance, and operation. A project's design will affect its initial construction cost as well as future maintenance and rehabilitation costs. The initial design can affect not only the frequency of required maintenance, but costs of performing maintenance as well. Whether as the result of formal value engineering studies or less formal evaluation of design alternatives, small changes in design that facilitate maintenance and operations may pay for themselves in long-term cost savings.*

4. Analysis periods used in LCCAs should be long enough to capture long-term differences in discounted life-cycle costs among competing alternatives and rehabilitation strategies. The analysis periods should cover several maintenance and rehabilitation cycles and, depending on the condition and age of the facility, may cover reconstruction of the facility as well. Analysis periods for improvements on Interstate and other NHS highways generally should be longer than for improvements on lower order roads, reflecting the NHS's greater importance.

5. All significant differences in agency and user costs anticipated during the analysis period should be considered in the analysis. Agency costs should consist of initial construction costs, future maintenance and rehabilitation costs including traffic control costs and costs of special construction procedures to maintain traffic, and agency operating costs for such things as tunnel lighting and ventilation. Where the agency operating a facility is not the one making the investment decision, it is important for the funding agency to include operating costs borne by all organizations responsible for operating the facilities. User costs to be considered in an LCCA generally include vehicle operation costs, accident costs, and delay-related costs incurred throughout the analysis period. Increased costs due to deteriorated riding surfaces, circuitous routings, and accidents and delays around and through work zones are important cost considerations.

6. While there may be considerable uncertainty about the life an improvement, future traffic using the facility, future maintenance and rehabilitation costs, user-operating delay costs, the appropriate discount rate to use, and other elements of LCCA, these factors should all be considered in the analysis. Regarding uncertainty, Executive Order 12893 indicates that "when the amount and timing of important benefits and costs are uncertain, analyses shall recognize the uncertainty and address it through appropriate quantitative and qualitative assessments." These assessments may include sensitivity analysis, probabilistic or risk analysis techniques, expert panels, or other methods for estimating the degree of uncertainty underlying key LCCA factors and the influence of that uncertainty on the choice of investment alternatives. Even if there is a relatively high degree of uncertainty about key LCCA factors, it is better to try to evaluate that uncertainty than to ignore it.

7. Future agency and user costs should be discounted to net present value or converted to equivalent uniform annual costs using appropriate discount rates. Discount rates selected should be consistent with guidance provided in OMB Circular A 94. Technical advisories on these and other technical issues in the application of LCCA will be issued by FHWA in the future.

Life-Cycle Cost Example

A detailed example of a life-cycle cost analysis for a pavement project is provided in figure 5-1.4. It should be recognized that such procedures are not precise, since reliable data for maintenance, subsequent stages of construction, salvage value, and pavement life are not always available and it is usually necessary to apply engineering judgment. Even though these difficulties exist, this approach is believed to provide the best potential to achieve the lowest annual pavement cost.

This life-cycle cost analysis is an example of the procedures and concepts only, and nothing should be concluded as to the economic benefits of one alternative over the others. The costs of construction and pavement conditions vary widely, and thus completely different results may be obtained in another location and for different pavement condition and costs.

Step 1. Existing Pavement Design:

9 in (229 mm) jointed plain concrete pavements (JPCP)	Skewed, random joint spacing, 3.7 to 5.9 m
4 in (102 mm) cement treated base	No dowel bars at transverse joints
Silty clay subgrade	15 years old
Rural, four lane divided highway (controlled access)	

Step 2. Existing Pavement Condition:

a. 7 percent cracked slabs in outer lane.
b. 50 yd^2 (45.7 m^2) of partial depth repair required for joint spalls.
c. Pumping along approximately three quarters of the project (outer lane only).
d. Faulting in outer lane is serious, averaging 0.15 in (3.8 mm). Inner lane is only 0.05 in (1.3 mm).
e. Present Serviceability Index averages 2.8 in the outer lane and 3.8 in the inner lane.
f. AC outer shoulder in fair condition.

Step 3. Select Feasible Alternatives (that both repair and prevent distress):

		ALTERNATIVES	
WORK ITEMS	**No. 1**	**No. 2**	**No. 3**
Replace Shattered Slabs	X	X	
Patch Spalls	X		
Stabilize Slabs (75% Panels)	X	X	
Subdrains	X	X	X
Grind Surface Outer Lane	X		
AC Overlay		X	
Replace Outer Lane (recycle existing concrete)			X
Place Tied PCC Shoulder		X	
Initial Service Life (years)	**7**	**12**	**20**

Step 4. Select Analysis Period:

The three alternatives have different initial service lives. All alternatives must be analyzed over the same analysis period (e.g., 20 years) to conduct a life-cycle cost analysis.

Step 5. Select Discount Rate:

Based on historical data, a discount rate of 4 percent is selected.

Figure 5-1.4. Example of preliminary life-cycle cost analysis of pavement rehabilitation alternatives.

Step 6. Estimate Life-Cycle Costs and Computed Present Worth:

The unit costs to perform each of the work tasks are estimated using previous bid estimates and other information. The unit costs are estimated for the initial year, which will be the first year of the analysis period. For each alternative, maintenance is planned to be applied about once every 5 years to 7 years. This includes joint resealing, crack sealing, and slab stabilization. The salvage value represents the expected worth of the existing pavement at the end of its service life. It was estimated as a percentage of the original construction and subsequent rehabilitation costs. All costs, including shoulder costs, are computed per two lane mile.

No. 1 Restoration

Year	Work Type	Cost, $	Present Worth, $
0	Restoration	90,580	90,580
7	Restoration	45,760	34,774
14	Resurface	144,322	83,342
20	End of Service Life		
	Salvage Value	67,500	30,000

Life of alternative = 20 years Total Present Worth = $177,890

No. 2 Resurfacing

Year	Work Type	Cost, $	Present Worth, $
0	Resurfacing	180,162	180,162
6	Maintenance	10,000	7,903
12	Resurfacing	96,215	60,096
20	End of Service Life		
	Salvage Value	85,000	38,793

Life of alternative = 20 years Total Present Worth = $209,368

No. 3 Reconstruct Outer Lane and Shoulder

Year	Work Type	Cost, $	Present Worth, $
0	Replace Lane/ Shoulder	255,700	255,700
10	Maintenance	10,000	6,756
15	Maintenance	10,000	5,553
20	End of Service Life		
	Salvage Value	60,000	27,383

Life of alternative = 20 years Total Present Worth = $240,626

Present worth calculations are performed using the following equation:

Present Worth = Cost $[1/(1+i)^n]$

where: i = Discount rate (4 percent in this example)
n = Time to incurred cost (in years)
Cost = Individual cost in current dollars

Figure 5-1.4. Example of preliminary life-cycle cost analysis of pavement rehabilitation alternatives. (continued)

Step 7. Compute Equivalent Uniform Annual Costs:

Using the PW, the EUAC is determined using the following equation:

$$\text{EUAC} = (\text{Present Worth}) \; x \; \text{CRF}$$

where CRF = capital recovery factor = $i/[1\ 1/(1+i)^n]$.

The PW and EUAC are summarized below for each alternative:

Alternative	Analysis Period (yrs)	Present Worth, $	CRF	EUAC, $
No. 1 Restoration	20	177,890	0.0736	$13,093
No. 2 Resurfacing	20	209,368	0.0736	$15,409
No. 3 Replace Lane and Shoulder	20	240,626	0.0736	$17,710

Step 8. Summary of Results:

The life-cycle cost analysis shows that Alternative No. 1, Restoration, has the least equivalent uniform annual cost. This alternative would cost the agency the equivalent of a yearly payment of $13,093 over a 20-year period at the assumed discount rate. Alternative No. 2 is 18 percent more costly than No. 1, and No. 3 is 35 percent more costly than No. 1.

User costs due to lane closures or extra user costs due to increased roughness have not been included. If traffic volume was very high, the alternative that had the most lane closure would have a much higher user cost and the consideration of user costs could possibly change the cost analysis significantly.

Step 9. Effect of Error in Life or Cost Estimates:

Given the preceding results, an important question that could be asked is: if the estimate of pavement life was in error, how much would this affect the resulting life-cycle costs (or EUAC)? To answer this question, assume that the restoration alternative lasts only 5 years instead of 7. This is an error of 40 percent. (The following average annualized costs result for this service life using the same costs as above and placing an overlay at 10 years instead of 14 years).

$$\text{EUAC} = \$14{,}343$$

This value is 9.5 percent higher than the cost for a restoration life of 7 years. Thus, a fairly large error in life prediction will result in a much smaller error in the annual cost. This occurs because of the effect of discounted future costs.

Figure 5-1.4. Example of preliminary life-cycle cost analysis of pavement rehabilitation alternatives. (continued)

Software has also been developed that advises the design engineer in pavement evaluation, projection of future determination, selection of feasible rehabilitation alternatives, and life-cycle costing for PCC pavements.[21] A recent study conducted using case studies of PCC pavement rehabilitation showed that when the existing pavement was in relatively “good” condition, restoration was the most cost-effective strategy. For those pavements rated “fair,” restoration was the most cost-effective strategy in four out of five cases, and for those rated “poor,” overlay or reconstruction were the most cost-effective in all cases.[22]

Evaluate Other Important Decision Factors

The life-cycle cost of a rehabilitation alternative is only one of several factors that should be considered in the overall evaluation of the different design alternatives. Other factors, which are difficult to quantify in monetary terms, should also be considered in the evaluation process. Management and policies within a highway agency should generally establish selection criteria and weighting. Normally, the decision factors will include:

1. Overall pavement management of network (policies).
2. Future rehabilitation options and needs.
3. Auto and truck traffic volume.
4. Initial costs.
5. Future maintenance requirements.
6. Traffic control during construction (safety and congestion).
7. Construction considerations (duration of construction).
8. Conservation of materials and energy.
9. Potential foundation problems.
10. Potential climatic problems.
11. Performance of similar pavements in the area.
12. Availability of local materials and contractor capabilities.
13. Worker safety during construction.
14. Incorporation of experimental features.
15. Stimulation of competition.
16. Municipal preference, local government preference, and recognition of local industry.

These may be considered either in a general way, or as tie breakers when they produce equal costs for the alternatives or in a more structured decision matrix.

Detailed Design for Selected Alternative

After the preferred alternative has been selected, a more detailed design and cost estimate should be made. This may require further field and laboratory testing and evaluation. The final design should be reasonably close to the design used in the preliminary analysis.

The plans and specifications must be carefully prepared to reflect the final design. A number of rehabilitation projects have not been constructed properly due to deficiencies in the plans and specifications. Many rehabilitation projects require continuous inspection to ensure a quality project.

7. OTHER CONSIDERATIONS

The degree of importance of a specific factor varies from project to project. Some factors are mandated by law and must be observed (environmental concerns, for example). Consideration of other factors is just good engineering practice. At times, a factor may have little influence on the cost of a project, while at other times it may dictate a significant percentage of the total cost (for example, lengthening of vertical curves to obtain greater sight distance). In some instances, one of the factors may even dictate major design decisions, such as when right of way limitations restrict widening of streets.

8. SUMMARY

This module describes the steps required in the design and selection of rehabilitation alternatives. The eight steps shown in the beginning of this module are believed to be extremely important in determining the most cost-effective rehabilitation alternative. Key items include a careful evaluation of the existing project, being creative in developing feasible alternatives that repair existing distress and prevent (or minimize) further distress from occurring while meeting the constraints of the project, and selecting the preferred design based on life-cycle costs and other important factors. These principles applied over many rehabilitation projects will certainly provide the best overall pavement condition for the limited available funding.

9. NETWORK CONDITION AND REHABILITATION TIMING (LIFE-CYCLE COSTS)

The same basic analysis that is used to analyze the life-cycle costs also applies to networks. One of the most basic considerations regarding pavement performance that applies at both the project and network level is the high cost of delaying rehabilitation treatments. After pavements are initially constructed, they generally perform in a satisfactory manner with a slow rate of deterioration for some period of time. At some point, however the pavement begins to deteriorate more quickly. As time progresses the pavement continues to deteriorate but at an ever increasing rate. Some knowledge of the typical deterioration curve for a particular pavement type and treatment is necessary in determining the optimum time at which an overlay is most cost-effective. User costs, the overlay restoration and overlay thickness required, the overlay construction costs, and overlay life are all affected by the rate of deterioration.

For any given pavement, as the pavement serviceability decreases below a critical level, user costs generally increase rapidly. These user costs include vehicle maintenance and operation costs, lost travel time due to delays from lane closures, and accidents. At the same time, as the pavement condition deteriorates, additional distresses develop. These additional distresses will invariably require additional repair, in terms of patching, subsealing, crack sealing full-depth replacement, joint repair, etc., prior to placement of an overlay. Even with extensive repairs, not all of the pavement deterioration can be repaired prior to the placement of the overlay which results in decreased service of the overlay.

The conceptual relationship between the drop in serviceability and the timing of rehabilitation is shown in figure 5-1.5. This graph illustrates that if $1 is spent for rehabilitation after 75 percent of the pavement life is consumed (here 75 percent of the life of a pavement produces a 40 percent drop in the serviceability of the pavement), $4 to $5 will be spent if the rehabilitation is delayed until 87 percent of the life is consumed. In this example 75 percent of the life of the pavement consumed 49 percent of the serviceability of the pavement and an additional 12 percent delay in treatment timing caused an additional 40 percent loss of serviceability. While the actual numbers are variable and highly dependent on many factors unique to each agency, the concept is quite valid and emphasizes the importance of the timely application of pavement rehabilitation.

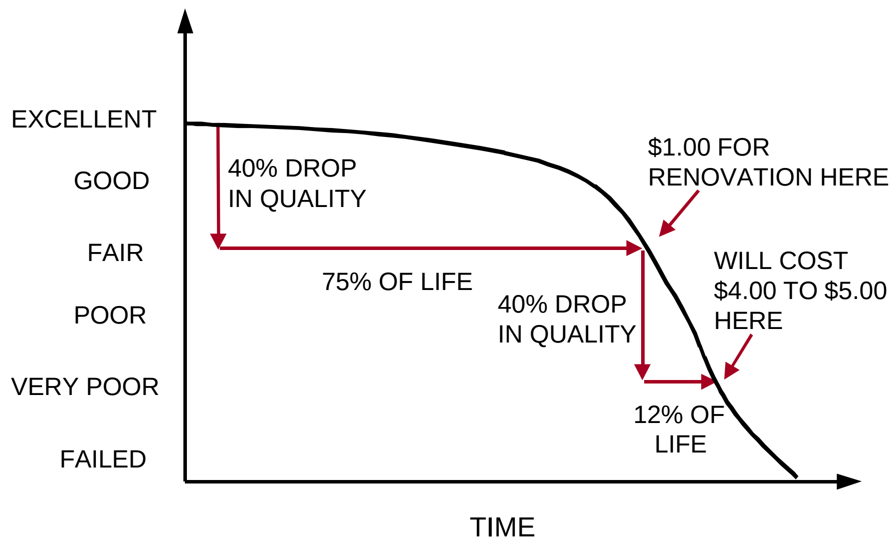


Figure 5-1.5. Typical pavement condition life-cycle.

Pavement deterioration and the high cost of scheduling rehabilitation at lower serviceability levels have been analyzed by several States. One such study was conducted by the Washington State Department of Transportation.[26] In this study, Pierce used the average pavement performance curves that came from performance data contained in their pavement management system (PMS) for a basic pavement type and treatments. Specific overlay thickness were determined using American Association of State Highway and Transportation Officials (AASHTO) overlay design procedures from the 1986 AASHTO Pavement Design Guide, using component analysis at different pavement condition levels. Costs were determined for each series of strategies based on the programmed condition level, and the overlay thickness and preparation costs needed at the various levels. The following three figures show the performance curves for treatments at three different program service levels.

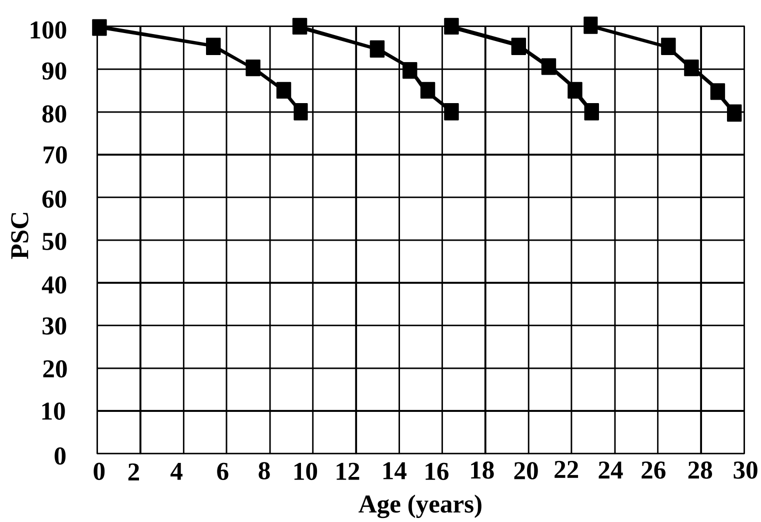


Figure 5-1.6 Example of strategy where the pavement is resurfaced when it reaches a pavement condition rating of 80.

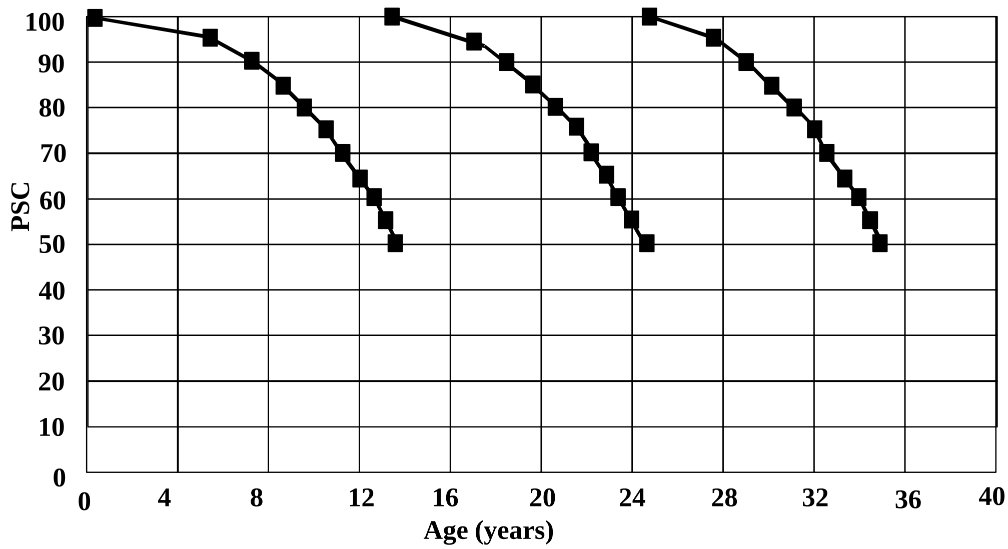


Figure 5-1.7. Example of strategy where the pavement is resurfaced when it reaches a pavement condition rating of 50.

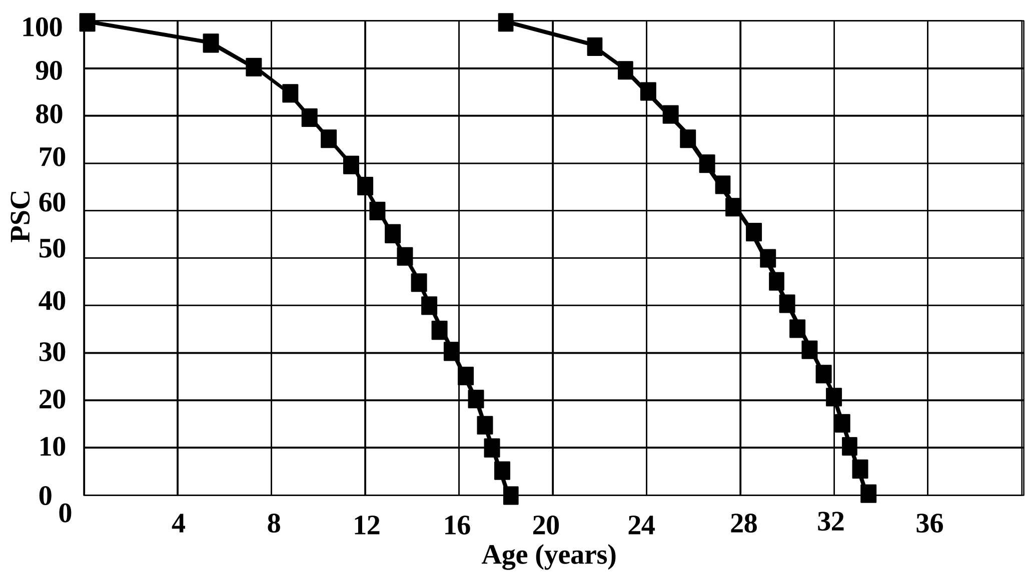


Figure 5-1.8. Example of strategy where the pavement is resurfaced when it reaches a pavement condition rating of 0.

The present worth was determined for each of the different strategies and also the uniform annual cost. The uniform annual cost for each strategy can be seen in figure 5-1.9. The resulting annualized costs are shown for strategies that resurface the roadway at pavement condition levels of 80, 70, 60, 50, etc.

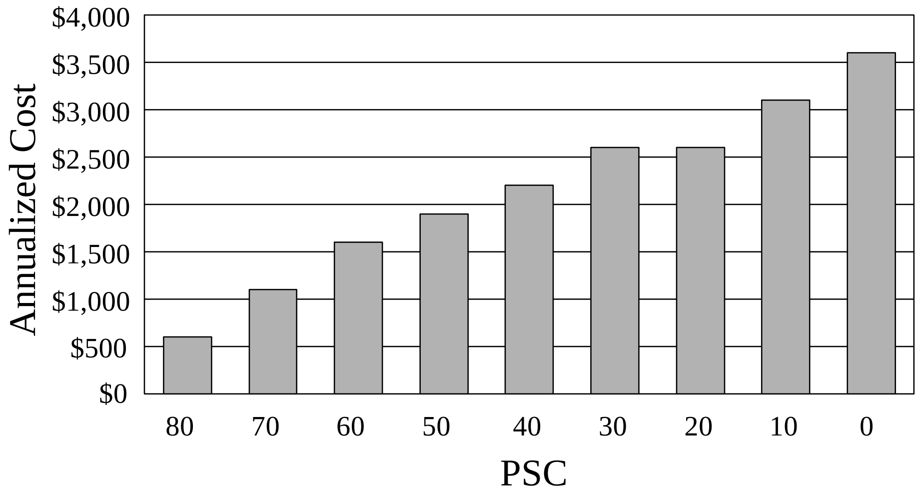


Figure 5-1.9. Annualized costs per lane mile for treatments applied at nine different pavement condition levels.

Referring back to the set of three figures that represent the performance curves for treatments applied at 80, 50 and 0 condition, the level at which a strategy is programmed correlates directly with the time between rehabilitation treatments. From the example, curves the time associated with applying a treatment at 80 was 7 years between treatments. The time between applying treatments at level 50 was about 11 years. When you consider this timing between treatments at the network level determines the amount of the network that has to be resurfaced every year. Thus, for a pavement repair strategy that programs projects at a pavement condition of 80, or every 7 years, then 1/7 of the network has to be paved every year. For a pavement repair strategy that programs projects at a pavement condition level of 50 then 1/11 of the network has to be paved every year. With this information, Washington Department of Transportation (WSDOT) combined the annualized cost for each strategy with the corresponding amount of the network that had to be resurfaced each year for each strategy which then produced an annualized cost for the network for each strategy.

The annualized cost for each strategy or condition level is shown in the following figure 5-1.10.

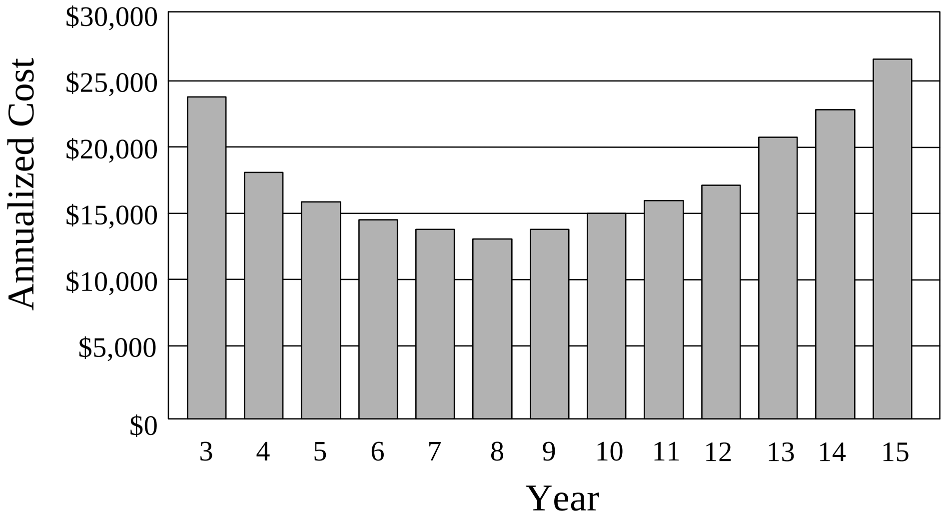


Figure 5-1.10. Annualized cost for the network per centerline mile for rehabilitation at different pavement condition levels and cycles.

The figure shows that the lowest annualized network costs were achieved when the pavement was resurfaced every 7 years to 9 years which corresponds to resurfacing at a pavement condition level of about 60. This study showed clearly that it was actually more cost-effective to maintain the pavements in good condition than it was to let them deteriorate to poor condition. Good roads do cost less. A similar study conducted in Utah showed the same results.

10. REFERENCES

1. "Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials, Washington, DC, 1986.

2. Smith, R.E., R.P. Palmieri, M.I. Darter, and R.L. Lytton, "Pavement Overlay Design Procedures and Assumptions, Volumes I, II, and III," FHWA/RD 85/006, FHWA/RD 85/007, FHWA/RD 85/008, Federal Highway Administration, 1985.

3. Darter, M.I. and K.T. Hall, "Structural Overlay Strategies for Jointed Concrete Overlays, Volume IVC Guidelines for the Selection of Rehabilitation Alternatives," FHWA RD 89 145, Federal Highway Administration, June 1990.

4. Turner, O.D. and R.T. Reark, "Value Engineering in Preconstruction and Construction," NCHRP Synthesis of Highway Practice 78, Transportation Research Board, 1981.

5. "Value Engineering for Highways," Course Notebook, Federal Highway Administration/National Highway Institute, 1986.

6. De Bono, E., "De Bono's Thinking Course," BBC Books, London, England, 1988.

7. Van Wijk, A.J., “Purdue Economic Analysis of Rehabilitation and Design Alternatives in Rigid Pavements, CA User’s Manual for PEARDARP,” Technical Report, Federal Highway Administration, September 1985.

8. Uddin, W., R.F. Carmichael III, and W.R. Hudson, “Life-Cycle Analysis for Pavement Management Decision Making Final Report,” FHWA PA 85 028, Federal Highway Administration, 1986.

9. “Vehicle Operating Costs, Fuel Consumption, and Pavement Type and Condition Factors,” Final Report, Appendix A, Federal Highway Administration, Office of Highway Planning, June 1982.

10. McFarland, W.F., “Benefit Analysis for Pavement Design Systems,” Research Report 123 13, Texas Transportation Institute, 1972.

11. Thuesen, G.J. and W.J. Fabrycky, “Engineering Economy,” Prentice Hall, Inc., Englewood Cliffs, NJ, 1984.

12. Brigham, E.F., “Financial Management, Theory and Practice,” Third Edition, Dryden Press, 1982.

13. Browning, G.S., “Pavement Selection Based on Life-Cycle Cost,” Final Report, State Study No. 82, Mississippi State Highway Department, July 1985.

14. Peterson, D.E., “Life-Cycle Cost Analysis of Pavements,” NCHRP Synthesis of Highway Practice 122, Transportation Research Board, 1985.

15. Campbell, B. and T.F. Humphrey, “Methods of Cost-Effectiveness Analysis for Highway Projects,” NCHRP Synthesis of Highway Practice 142, Transportation Research Board, December 1988.

16. Kher, R.K., W.A. Phang, and R.C.G. Haas, “Economic Analysis Elements in Pavement Design,” Transportation Research Record 572, Transportation Research Board, 1976.

17. Darter, M.I., E.J. Barenberg, and W.A. Yrjanson, “Repair of Jointed Concrete Pavement,” NCHRP Report 281, Transportation Research Board, 1985.

18. Snyder, M.B., M.J. Reiter, K.T. Hall, and M.I. Darter, “Rehabilitation of Concrete Pavements, Volume I Repair Rehabilitation Techniques,” FHWA RD 88 071, Federal Highway Administration, July 1989.

19. Oglesby, C.H. and R.G. Hicks, “Highway Engineering,” 4th Edition, John Wiley and Sons, Inc., New York, 1982.

20. Epps, J.A. and C.V. Wootan, “Economic Analysis of Airport Pavement Rehabilitation Alternatives - An Engineering Manual,” DOT FAA FD 81 78, Federal Aviation Administration, 1981.

21. Hall, K.T., J.M. Connor, M.I. Darter, and S.H. Carpenter, “An Expert System for Concrete Pavement Evaluation and Rehabilitation,” Transportation Research Record 1207, Transportation Research Record, 1988.

22. Darter, M.I. and K.T. Hall, “Case Studies in Rehabilitation Strategy Development,” Transportation Research Record 1307, Transportation Research Board, 1991.

23. Federal Register: September 18, 1996 (Volume 61, Number 182).

24. "Life-Cycle Cost Analysis," Searching for Solutions A Policy Discussion Series Number 12 Summary of Proceedings FHWA Life-Cycle Cost Symposium, December 15, 16, 1993. Washington, DC.

25. Washington State Pavement Design Guide Volume 2 Olympia, Washington 1996.

26. Pierce, L.M., "Determination of the Effective Pavement Structural Condition for Pavement Rehabilitation" unpublished report WSDOT, Olympia, June 1992.

SOURCES OF ADDITIONAL INFORMATION

In the development of this course, extensive use was made of published literature so that the most current and up-to-date information is provided to the participants. Each module contains a comprehensive listing of pertinent reports, papers, and publications that were used in its development, and the participant is encouraged to refer to those for more detailed or additional information on a specific topic.

To assist interested participants in obtaining specific reports or in contacting professional and industry associations for more information on a particular topic, the following addresses are provided:

STATE AND FEDERAL ORGANIZATIONS

AASHTO Publications

American Association of State Highway and Transportation Officials (AASHTO)
444 N. Capitol Street, N.W., Suite 225
Washington, D.C. 20001
(202) 624-5800

FHWA Publications

National Technical Information Service (NTIS)
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650
(703) 321-8547 (*fax*)

TRB Publications (including NCHRP)

Transportation Research Board
P.O. Box 289
Washington, D.C. 20055
(202) 334-3214
(202) 334-2519 (*fax*)

PROFESSIONAL AND INDUSTRY ASSOCIATIONS

American Concrete Institute (ACI)
P.O. Box 19150
Detroit, MI 48219-0150
(313) 532-2600
(313) 533-4747 (*fax*)

American Concrete Pavement Association (ACPA)
3800 N. Wilke Road, Suite 490
Arlington Heights, IL 60004
(708) 394-5577
(708) 394-5610 (*fax*)

American Society for Testing and Materials (ASTM)
1916 Race Street
Philadelphia, PA 19103-1187
(215) 299-5585
(215) 977-9679 (*fax*)

American Coal Ash Association (ACAA)
1913 I. St., N.W., Sixth Floor
Washington, D.C. 20006
(202) 659-2303
(202) 223-4984 (*fax*)

American Public Works Association (APWA)
Research Foundation
106 West 11th Street, Suite 800
Kansas City, MO 64105-1806
(816) 472-6100
(816) 472-1610 (*fax*)

National Center for Asphalt Technology
(NCAT)
211 Ramsay Hall
Auburn, AL 36849-5354
(205) 844-6228
(205) 844-6248 (*fax*)

American Society of Civil Engineers, (ASCE)
Book Orders
P.O. Box 831
Somerset, N.J. 08875-0831
(212) 705-7288
(212) 980-4681 (*fax*)

National Stone Association (NSA)
1415 Elliot Place, N.W.
Washington, D.C. 20007
(202) 342-1100
(202) 342-0702 (*fax*)

American Road & Transportation Builders
Association (ARTBA)
501 School Street, S.W.
Washington, D.C. 20024
(202) 488-2722
(202) 488-3631 (*fax*)

Asphalt Emulsion Manufacturers Association
(AEMA)
#3 Church Circle, Suite 250
Annapolis, MN 21401
(410) 267-0023
(410) 267-7546 (*fax*)

The Asphalt Institute (AI)
Executive Offices and Research Center
Research Park Drive
P.O. Box 14052
Lexington, KY 40512-4052
(606) 288-4960
(606) 288-4999 (*fax*)

Asphalt Recycling and Reclaiming Association
(ARRA)
#3 Church Circle, Suite 250
Annapolis, MN 21401
(410) 267-0023
(410) 267-7546 (*fax*)

Asphalt Rubber Producers Group (ARPG)
3336 North 32nd Street, Suite 106
Phoenix, AZ 85018-6241
(602) 955-1141
(602) 956-3506 (*fax*)

Concrete Sawing and Drilling Association
(CSDA)
6077 Roswell Road, Suite 205
Atlanta, GA 30328
(404) 257-1177
(404) 843-1372 (*fax*)

Concrete Reinforcing Steel Institute (CRSI)
CRSI Distribution Center
P.O. Box 100125, Dept. No. B29
Roswell, GA 30075
(404) 442-8631
(404) 442-9742 (*fax*)

Institute of Transportation Engineers (ITE)
525 School St., S.W., Suite 410
Washington, D.C. 20024-2729
(202) 554-8050
(202) 863-5486 (*fax*)

International Slurry Seal Association
1101 Connecticut Avenue, N.W.
Suite 700
Washington, D.C. 20036
(202) 857-1160
(202) 429-5108 (*fax*)

National Ready Mix Concrete Association
(NRMCA)
900 Spring St.
Silver Spring, MD 20910
(301) 587-1400
(301) 585-4219 (*fax*)

International Grooving and Grinding Association (IGGA)
Nine Village Circle
Suite 450
Westlake, TX 76262
(817) 491-9585
(817) 430-5801 (*fax*)

National Lime Association (NLA)
3601 North Fairfax Drive
Arlington, VA 22201
(703) 243-5463

National Asphalt Pavement Association (NAPA)
NAPA Building
5100 Forbes Boulevard
Lanham, MD 20706-4413
(301) 731-4748
(301) 731-4621 (*fax*)

Portland Cement Association (PCA)
Order Processing
5420 Old Orchard Road
Skokie, IL 60077-9823
(708) 966-6200 (ext. 564)
(708) 966-9666 (*fax*)

PAVEMENT COMPUTER PROGRAMS AND SOFTWARE

McTrans
The Center for Microcomputers in Transportation
University of Florida
512 Weil Hall
Gainesville, FL 32611
(904) 392-0378
(904) 392-3224 (*fax*)

PC-TRANS
University of Kansas
Transportation Center
2011 Learned Hall
Lawrence, KS 66045
(919) 864-5655
(919) 864-3199 (*fax*)