NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE

9

PAVEMENT REHABILITATION

MATERIALS AND TECHNIQUES

REFER TO:	Action	Info	Int
Materials Engineer			
Assoc. Mat'ls Engr. I			
Assoc. Mat'ls Engr. II			
Soils Engineer			
Geologist			
Testing Engineer			
Office Manager			
Quality Control			
Project Development			
E.T.			
Chem.-Asphalt			
Mixes-Struc.			
Aggregate & Soils			

HIGHWAY RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE 9

PAVEMENT REHABILITATION

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION
OF STATE HIGHWAY OFFICIALS IN COOPERATION
WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:
PAVEMENT DESIGN
PAVEMENT PERFORMANCE
MAINTENANCE, GENERAL

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1972

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectvity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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PREFACE

There exists a vast storehouse of information relating to nearly every subject of concern to highway administrators and engineers. Much of it resulted from research and much from successful application of the engineering ideas of men faced with problems in their day-to-day work. Because there has been a lack of systematic means for bringing such useful information together and making it available to the entire highway fraternity, the American Association of State Highway Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Highway Research Board to undertake a continuing project to search out and synthesize the useful knowledge from all possible sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series attempts to report on the various practices without in fact making specific recommendations as would be found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available concerning those measures found to be the most successful in resolving specific problems. The extent to which they are utilized in this fashion will quite logically be tempered by the breadth of the user's knowledge in the particular problem area.

FOREWORD

By Staff
Highway Research Board

This report should be of special interest to highway engineers responsible for pavement design, including composite, flexible, and rigid; pavement performance, including dynamic loading effects and evaluation; and maintenance, including drainage, shoulders, and the pavement structure itself.

Administrators, engineers, and researchers are faced continually with many highway problems on which information already exists either in documented form or in terms of undocumented experience and practice. Unfortunately, this information is often fragmented, scattered, and unevaluated. As a consequence, full information on what has been learned about the problem is frequently overlooked in seeking a solution. Costly research findings and valuable experience may go unused and due consideration may not be given to recommended practices for

solving or alleviating the problem. In an effort to resolve this situation, a continuing NCHRP project, carried out by the Highway Research Board as the research agency, has the objective of synthesizing and reporting on common highway problems—a synthesis being defined as a composition or combination of separate parts or elements so as to form a whole greater than the sum of separate parts. Reports from this endeavor constitute an NCHRP Report series that collects and assembles the various forms of information into single concise documents pertaining to specific highway problems or sets of closely related problems. This is the ninth report in the series.

Prior to the initiation of construction of the Interstate Highway System in the United States, considerable attention was given to rehabilitation of pavements originally constructed in the 1930's and minimally maintained throughout the early 1940's so that they would be able to carry the increased traffic volumes and heavier wheel loads in the late 1940's and early 1950's. With initiation of the Interstate System in 1956, most highway agencies shifted emphasis from rehabilitation of existing highways to construction of new roads, often on new right-of-way. Both the Interstate and other new primary and secondary roads since the mid 1950's have generally been designed with pleasing horizontal and vertical alignments. Thus, it is expected that future emphasis will be on reconstruction or rehabilitation of existing highways maintaining the present alignment. Because traffic volumes have far exceeded projections, many miles of the Interstate and other major elements of the national highway network currently require, or in the near future will require, pavement rehabilitation much sooner than originally contemplated. Thus, the topic of pavement rehabilitation is one of immediate concern and no doubt will remain of serious concern over the next several years.

This report of the Highway Research Board provides information on the importance of maintaining existing pavement structures; the causes of pavement distress, and possible remedial treatments; overlay design procedures in common use that can be applied to both flexible and rigid overlays over either flexible or rigid pavement; design strategies for rehabilitation schemes to obtain a desired level of economy for a desired service life; example problems that demonstrate how to deal with present and future costs for rehabilitation schemes, including both user and maintenance costs; and current research efforts that bear on the topic, as well as future research needs.

To develop this synthesis in a comprehensive manner and to insure inclusion of significant knowledge, the Board analyzed available information (e.g., current practices, manuals, and research recommendations) assembled from many highway departments and agencies responsible for highway planning, design, construction, and maintenance. A topic advisory panel of experts from this subject area was established to guide the researchers in organizing and evaluating the collected data and for reviewing the final synthesis report.

As a follow-up, the Board will attempt to evaluate the effectiveness of the synthesis after it has been in the hands of its users for a period of time. Meanwhile, the search for better methods is a continuing activity and should not be diminished. An updating of this document is ultimately intended so as to reflect the improvements that may be discovered through research or practice.

CONTENTS

ACKNOWLEDGMENTS

This synthesis was completed by the Highway Research Board under the supervision of Paul E. Irick, Assistant Director for Special Projects. The Principal Investigators responsible for conduct of the synthesis were Thomas L. Copas and Herbert A. Pennock, Special Projects Engineers.

Special appreciation is expressed to Dr. B. Frank McCullough, Associate Professor, The University of Texas, who, as special consultant to the Advisory Panel, was responsible for the collection of data and the preparation of the report.

Valuable assistance in the preparation of this synthesis was provided by the Topic Advisory Panel, consisting of John W. Hewett, Highway Engineer, Pavement Design Branch, Highway Standards and Design Division, Office of Engineering and Operations, Federal Highway Administration; Roy W. Jump, Maintenance Engineer, Idaho Department of Highways; R. Ian Kingham, Staff Engineer, The Asphalt Institute; Walter B. McKendrick, Jr., Executive Director for Federal Agencies and National Organizations, Portland Cement Association; Thomas D. Moreland, State Highway Materials Engineer, Georgia State Highway Department; George W. Ring, Highway Engineer, Office of Research and Development, Federal Highway Administration; Donald R. Schwartz, Assistant Engineer of Research and Development, Bureau of Research and Development, Illinois Division of Highways; and George B. Sherman, Assistant Materials and Research Engineer, Materials and Research Department, California Division of Highways.

R. E. Bollen, Engineer of Materials and Construction; A. G. Clary, Engineer of Maintenance; J. W. Guinnee, Engineer of Soils, Geology, and Foundations; and L. F. Spaine, Engineer of Design; all of the Highway Research Board, assisted the Special Projects staff and the Advisory Panel.

Information on current practice and ongoing research was provided by many highway agencies. Their cooperation and assistance were most helpful.

PAVEMENT REHABILITATION

MATERIALS AND TECHNIQUES

SUMMARY Effective and economic use of pavements demands satisfactory service with a minimum of construction and maintenance for as long as possible. As a pavement gets older, it tends to become inadequate and will require rehabilitation to restore it to a satisfactory capacity and to extend its useful life.

The problems to which a pavement is subject are many and varied. Each problem has a number of possible causes, and for each cause there may be more than one method of rehabilitation.* Depending on the problem and its cause, the rehabilitation undertaken may be a relatively simple and inexpensive correction procedure, such as slabjacking or heater planing, or it may entail complete rehabilitation through the design and placement of an additional layer of material—an overlay.

Overlays are also subject to problems, and one of the most widespread of these is reflection cracking caused by expansion and contraction of the original pavement. Many methods have been tried in an attempt to minimize or eliminate reflection cracking. Where concrete is used to overlay concrete, some success has been obtained by a stress relief course between overlay and existing pavement, duplication of the existing joint pattern in the overlay, and additional reinforcement of the overlay. Numerous methods have been tried with asphalt overlays with varying degrees of success. Some of these are breaking of the existing concrete pavement prior to overlaying, steel reinforcement of the asphalt overlay, the use of additives (such as rubber) in the overlay mix, and the use of softer asphalts.

The proper structural design of an overlay is important. Overlays should be designed taking into account the existing support conditions, with the thickness of the overlay changing as support conditions change. Several design methods are available for the various overlay types. The underlying assumptions and limitations of the chosen design method should be understood before it is used. This is particularly true for the selection of the coefficients which represent the load-carrying capacity of the existing pavement.

Even though a decision has been made to overlay an existing pavement, economic considerations will often control the selection of a design scheme. A design scheme includes overlays, seal coats, and general maintenance. To obtain the most economic design, several schemes should be compared for a finite period of time (the cost analysis period). A design scheme may entail a single overlay designed to last through the cost-analysis period, or it may consist of a thinner overlay placed now, plus additional overlays placed at other times during the analysis period. All costs that will occur during this period should be considered. These include not only the cost of the overlay(s) and annual routine maintenance, but also the cost of maintaining traffic during placement of an overlay and user costs due to traffic delays. The comparison of the present values of all these costs will indicate which design scheme is the most economical.

A number of research projects concerning pavement rehabilitation are currently under way, but there are other areas where additional research may be beneficial. These include new methods and equipment for evaluating existing pavements and also ways to minimize reflection cracking.

* The problem of skid resistance is the subject of a future synthesis and is not covered herein.

CHAPTER ONE

INTRODUCTION

The primary purpose of a highway system is to provide a safe, comfortable, convenient, and economical method of transporting goods and people. The role of the engineer is to design, construct, and maintain the system in an efficient and economical manner. These constraints necessitate the rehabilitation and upgrading of existing pavement structures. How does the engineer evaluate the existing pavement and use it in the new design? How does he determine whether to continue maintenance, to reconstruct, or to rehabilitate? These are complex problems that must be solved by the engineer. This synthesis provides information concerning rehabilitation of an existing pavement structure to meet future demands.

BACKGROUND OF THE PROBLEM

During the late 1940's and early 1950's, considerable attention was given to rehabilitating pavements constructed in the 1930's so that they would be able to carry increased traffic and heavier wheel loads. With the initiation of the Interstate system in 1956, the emphasis of most highway agencies shifted from rehabilitation of existing highways to construction of new ones—in particular, the 42,500-mile Interstate system. This system and other new state, county, and city systems have been designed with good geometric alignment and thus future emphasis will be on reconstruction or rehabilitation of these highways on their existing alignment. Because some of these pavement structures were designed for service lives as low as 10 years, the development of criteria for pavement rehabilitation is a major problem requiring immediate consideration.

The engineer, after deciding to use an existing alignment, must next decide between reconstruction or rehabilitation of the existing pavement structure. Furthermore, a decision must be made as to what period of time the design will cover. The assumption herein is that these decisions have been made.

DEFINITION OF REHABILITATION

Rehabilitation means to restore to a satisfactory condition. Pavement rehabilitation must be considered in terms of the broader highway purpose of providing a safe, comfortable, convenient, and economical method of transporting goods and people. Thus, rehabilitation entails a variety of practices from treatment of a problem in an existing material to the addition of an overlay (i.e., a supplemental layer of material). An overlay may perform one or more of the following:

1. Restore the pavement riding quality.
2. Restore skid resistance.
3. Upgrade the load-carrying capability of the existing pavement structure.

Therefore, an overlay may be either a thin resurfacing to improve riding quality or skid resistance, or of sufficient thickness to add structural strength to the existing pavement. The problem of restoring skid resistance is recognized at this point, but, because it is the subject of a future synthesis, is not treated further herein.

On the basis of construction procedure, material use, and past precedence, overlays may be described as follows:

1. Bituminous concrete over an existing bituminous pavement.
2. Bituminous concrete over an existing portland cement concrete pavement.
3. Portland cement concrete over an existing bituminous pavement.
4. Portland cement concrete over an existing portland cement concrete pavement.
5. Bituminous concrete over an existing composite pavement.*

Another consideration is the use of a granular layer between existing pavement and overlay with the first two combinations.

The basic reasons for rehabilitating an existing pavement structure, regardless of pavement type, are:

1. To correct existing conditions of distress in the pavement structure.
2. To prevent future pavement distress.

CRITERIA FOR REHABILITATION

A key item in enumerating criteria for rehabilitation is the definition of unsatisfactory service in a pavement structure. This is much more complex than the occurrence of cracks in the surface of the pavement. Nor does unsatisfactory service imply a catastrophic event. In fact, a pavement may be carrying high traffic volumes; but, if a high rate of maintenance expenditures is required, the pavement should be considered as one providing unsatisfactory service. Thus, a set of decision criteria such as, but not necessarily limited to, the following are required to evaluate the performance history of a pavement and ascertain whether it is unsatisfactory.

1. *Level of Service.* The controlling level of service will vary with the type of facility. For example, an Interstate highway will require a much higher level of service than is required on a secondary road with low traffic volumes. Therefore, the rehabilitation requirements in Chapters Two and Three will vary considerably depending on the facility type.
2. *Riding Quality.* The riding quality is a judgment item

* Composite pavement structure—a structure comprising multiple, structurally significant layers of different, sometimes heterogeneous composition. Two layers or more must employ dissimilar, manufactured binding agents.

that expresses a user's opinion as to how well the pavement is serving traffic. Generally, some minimum level can be specified for a given facility, to represent a minimum tolerable value accepted by the public. Usually, riding quality can be substantially improved by providing an additional layer of material. Riding quality can also be improved by such techniques as slabjacking and heater planing, described in Chapter Two.

3. *Safety.* Although safety may be evaluated to a degree by accident statistics, it is also a judgment value that encompasses a wide variety of conditions. For example, the pavement width of an existing facility may be increased to provide lane widths required for safe movement of increased traffic. Skid resistance and hydroplaning are also pertinent features of safety, although they are not considered in this report.

4. *Structural Adequacy.* Structural adequacy expresses a pavement's capability to carry the wheel loads of the present and future traffic. This factor may be evaluated using the procedures outlined in Chapter Three.

5. *Surface Condition.* The appearance of the pavement (for example, cracking, patching, raveling) is not necessarily related to the structural adequacy, although an inadequate load-carrying capacity will ultimately lead to a poor appearance. Many detrimental surface conditions may be rectified by the practices and procedures discussed in Chapter Two, but those due to structural inadequacies must be corrected by the procedures discussed in Chapter Three.

6. *Cost.* The total cost required to keep the pavement adequately serving traffic is a primary decision criterion. This involves not only construction costs, but also maintenance costs and user costs. User costs may be incurred due to accidents or traffic delays. These are not direct costs from the construction fund, but they represent a real cost to the traveling public. If a high-traffic-volume facility is subject to delays because of frequent maintenance, high user costs will be experienced; hence, a major rehabilitation will be required. User costs are described in Chapter Four.

Procedures are available to quantitatively consider each of these decision criteria, but, at present, there is no mathematical model that may be used to combine all these decision criteria to ascertain if rehabilitation is justified. Therefore, each engineer and/or administrator must apply his own weighting factors to each of the criteria.

PAVEMENT EVALUATION

In evaluating a pavement structure, the engineer may use surface condition ratings and/or structural adequacy ratings. These may be used as a planning guide for establishing rehabilitation priorities. Surface condition ratings can be used to quantify the decision criteria items of riding quality, safety, and structural adequacy. It is emphasized that these factors give only an indication of how well the facility is serving the traveling public at that time, although the data are very useful in providing a criterion for allocation of funds to those areas with the greatest need. For a long-term prediction, the structural adequacy of the pavement structure must be ascertained.

Surface Evaluation

Several agencies have developed a number of techniques for surface evaluation. Two of these are the present serviceability concept and the Roads and Transportation Association of Canada (RTAC, formerly Canadian Good Roads Association) method.

The present serviceability concept was developed in connection with the AASHO Road Tests (*1, 2*). A Present Serviceability value may be obtained by either subjectively rating the pavement through visual observations or by quantitative measurements of surface conditions. The subjective rating method is called the Present Serviceability Rating (PSR). It involves the use of a group of raters who ride over the pavement section, observe its rideability, assess its condition, and record their impressions on a form similar to Figure 1.

The Present Serviceability Index (PSI) is a statistical regression of mechanical measurements from the surface of the pavement with a PSR rating from a panel. The PSI may be computed using Eq. 1 for flexible pavements or Eq. 2 for rigid pavements.

$$P = 5.03 - 1.91 \log (1 + \overline{SV}) - 0.01 \sqrt{C + P} - 1.38 \overline{RD}^2 \quad (1)$$

$$P = 5.41 - 1.80 \log (1 + \overline{SV}) - 0.09 \sqrt{C + P} \quad (2)$$

in which

P = Present Serviceability Index;
$\overline{SV}$ = the mean of the slope variance in the two wheel paths;
$C + P$ = a measure of cracking and patching in the pavement surface; and
$\overline{RD}$ = a measure of rutting in the wheel path.

All the foregoing parameters can be obtained readily, except the slope variance value; this requires the use of the AASHO Road Test longitudinal profilometer or the CHLOE profilometer. Other equipment (such as the BPR roughometer, the PCA road meter, and the GMR profi-

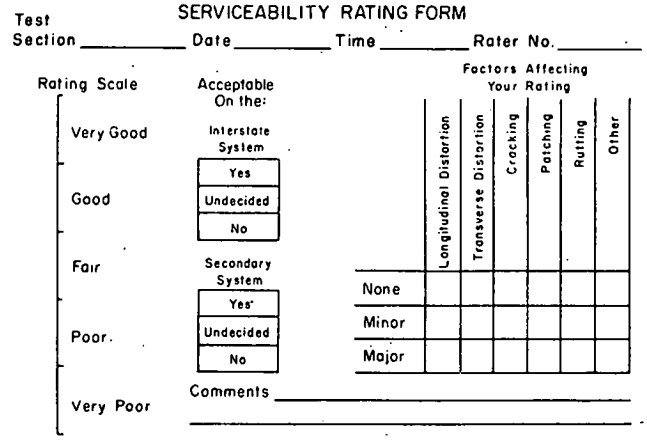


Figure 1. Present serviceability rating form for use by a rating panel.

lometer) requires correlation and modification to the equations (*3, 4*).

The Present Serviceability values may be used to measure a trend of pavement performance and to decide whether to proceed with a more detailed investigation. Experience suggests that Present Serviceability values in the range of 2.0 to 2.5 are considered as minimum values. In fact, some recent information from various state highway agencies indicates a value of 3.0 may be the point at which pavement rehabilitation should be considered and programmed (*5*). It is emphasized that these levels give guidance for planning purposes; they are not absolute minimums.

Figure 2 shows a typical rating form used in the RTAC method for rating the surface condition of flexible pavement (*6, 7*). Various pavement defects, as well as the severity in distribution, are noted on the form. With this method, the defects are related to the components of the pavement structure where the defect is considered to originate.

Structural Evaluation

The primary technique for structural evaluation is deflection measuring using the Benkelman beam (*1*), the Dynaflect (*8*), the Road Rater, etc. (Fig. 3). The selection of equipment will depend on availability and the rate at which measurements are to be made. The Benkelman beam requires considerable time, but its first-cost expense is low, whereas the others are much more rapid, but the initial expense is considerably higher. The procedures for structural evaluation are covered in detail in Chapter Three.

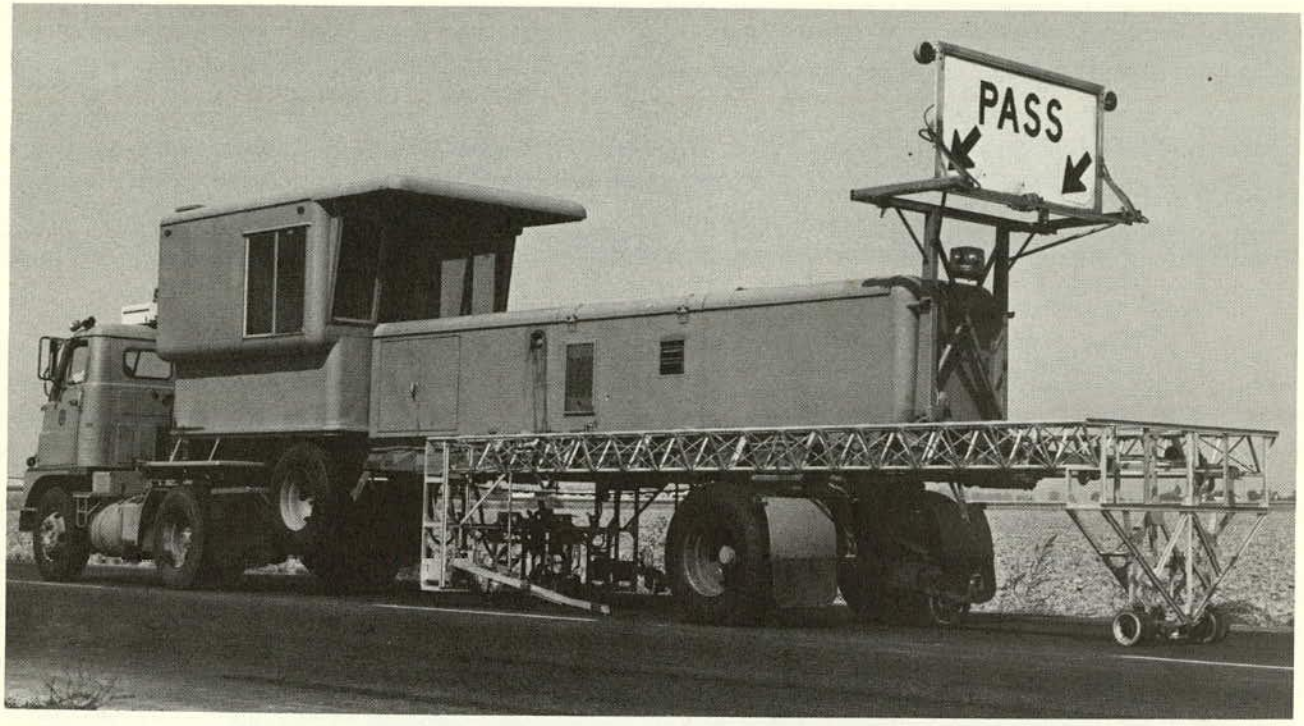
PAVEMENT DISTRESS AND BEHAVIOR

The decision criteria must consider riding quality, structural adequacy, and surface condition which may be directly or indirectly related to one or more manifestations of pavement distress (i.e., cracking, spalling, etc.). The individual manifestations, as seen by the engineer, cause him to start action toward some phase of rehabilitation.

Over the years various groups and individuals have

CONDITION SURVEY Date _____ Observers _____		Estimating Unit (by Observation)	Severity					Distribution of Condition		Remarks	Condition Diagrams (if required)
			None	Minor	Moderate	Major	Severe	Local	Distr.		
Unsatisfactory Surface	Hair Cracking (roller)	Area									
	Map Cracking	Area									
	Stripping	Length									
	Revelling	Length									
	Shoving	Length									
	Bleeding	Length									
Unsatisfactory Base or Subbase	Compaction Rutting O.W.P.	Depth									
	Compaction Rutting I.W.P.	Depth									
	Chicken Wire Cracking O.W.P.	Length									
	Chicken Wire Cracking I.W.P.	Length									
Inadequate Pavement Thickness or Strength	Shear Failure Rutting	Depth									
	Alligator Cracking O.W.P.	Length									
	Alligator Cracking I.W.P.	Length									
	Longitudinal Cracking O.W.P.	Length									
	Longitudinal Cracking I.W.P.	Length									
	Potholes	Length									
	Patches in O.W.P.	Length									
	Patches in I.W.P.	Length									
Age and Environment	Random Longitudinal Cracks	Length									
	Centreline Longitudinal Cracks	Length									
	Transverse Cracking	Frequency									
	Internal Corner Cracks	Length									
	External Corner Cracks	Length									
	Edge Breaking	Length									
	Differential Frost Heaving	Length									
	Surface Roughness	Riding									
	Surface Drainage (Ponding)	Length									
Quality, Extent and Significance of Const. and Maintenance	Differential Fill Settlements										
	Poor Crack Filling										
	Seal Coat or Surface Treatment										
	Patching										
	Other :										
	General Appraisal	Unit	Very Good	Good	Fair	Poor	Very Poor	Local	Distr.	Qualifications	
	Original Construction	Length									
	Maintenance	Length									

Figure 2. Pavement surface condition rating form. Source: Roads and Transportation Association of Canada.



California Traveling Deflectometer

Dynaflect

Road Rater

Figure 3. Deflection measurement equipment.

characterized various modes of distress (*2, 9-16*). The format of NCHRP Project 1-10 (*17*) has been used to divide distress modes into three categories: fracture, distortion, and disintegration (Fig. 4). A distress mechanism (e.g., loading) leads to a distress manifestation (e.g., cracking), which may be categorized in one of the distress modes (e.g., fracture).

A problem cannot be effectively corrected in the field unless the distress mechanism (cause) can be identified. Figure 4 can be used as a guide by first identifying the distress manifestation, followed by a careful on-site investigation to establish the distress mechanism. The techniques described in Chapters Two and Three may be used to correct the problem.

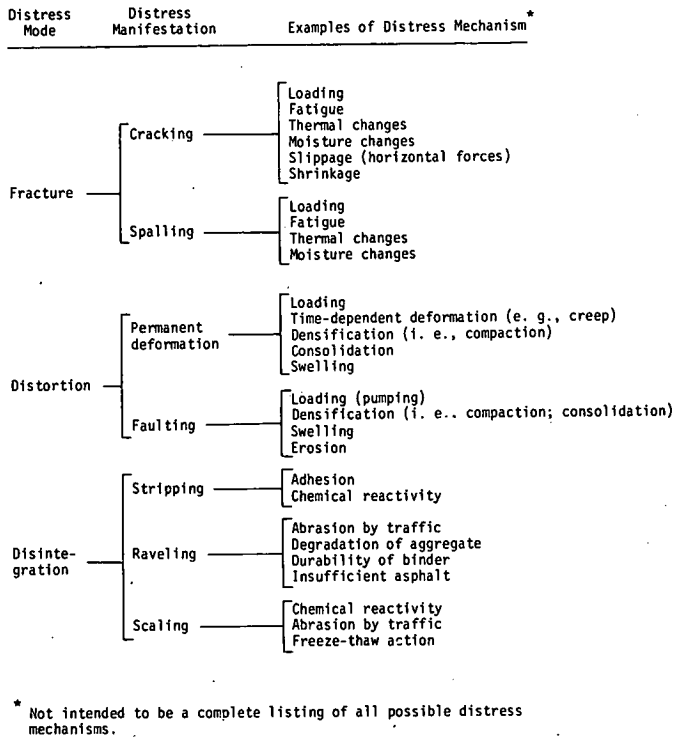


Figure 4. Categories of pavement distress.

CHAPTER TWO

PRESENT PRACTICES AND TECHNIQUES USED IN PAVEMENT REHABILITATION

To decide which procedure or technique to use in rehabilitating a pavement, it is first necessary to identify the type of distress manifestation (problem) affecting the existing pavement structure. The next step is to determine the distress mechanism (cause) producing the distress manifestation. After identifying the cause, the proper steps for improving the serviceability level may be taken (solution). Typical pavement deficiencies are illustrated in *HRB Special Report 113 (68)*.

The discussions for each problem are generally separated on the basis of the existing pavement type. The procedures are usually corrective, but in some cases preventive measures are presented that should ensure that the distress does not occur in an overlay.

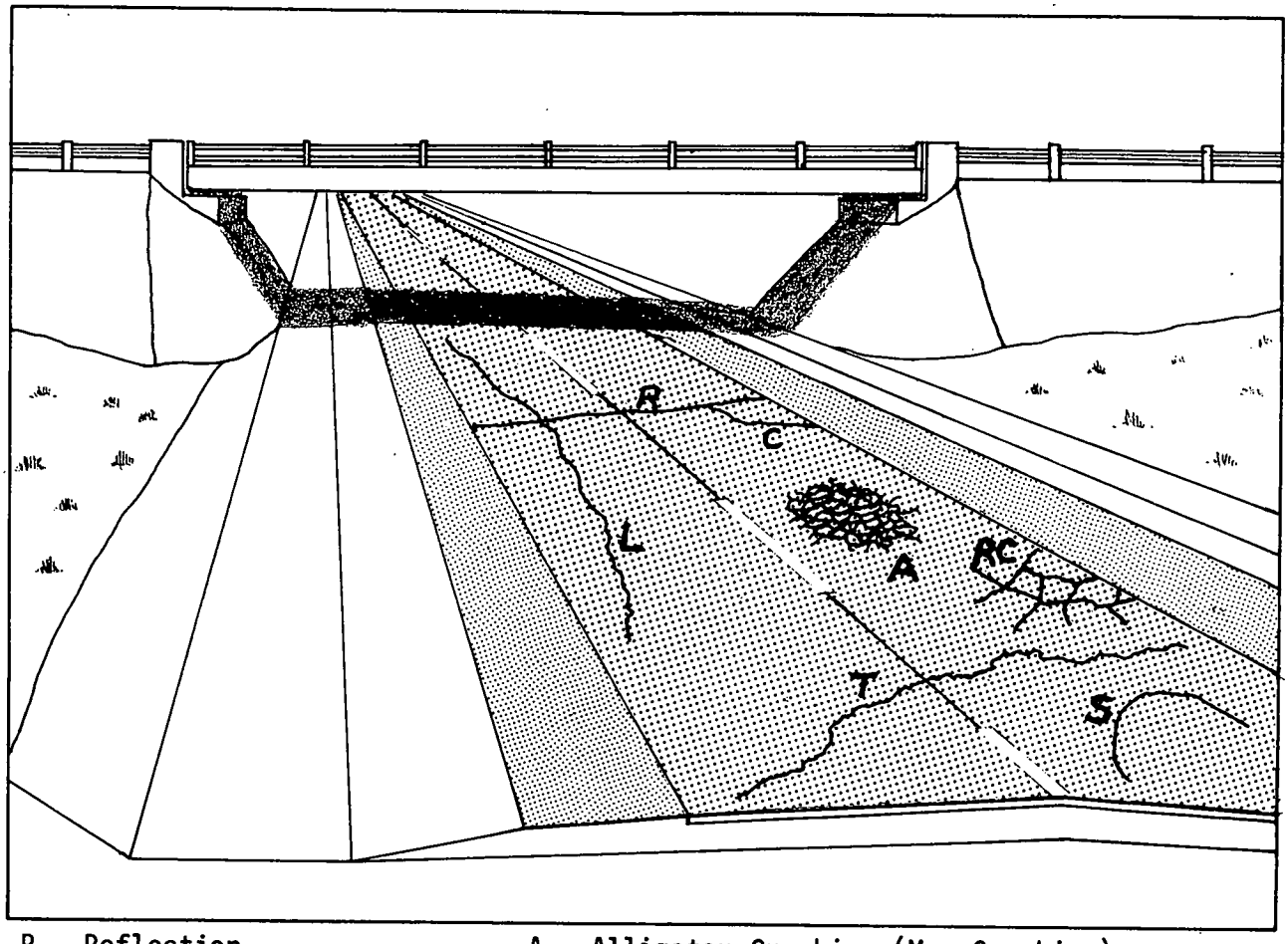
PROBLEM: CRACKING

Cracking may take many forms, as shown in Figure 5. Any increase of cracking lowers the PSI as may be seen by examining Eqs. 1 and 2. Although from a sensitivity standpoint an increase in cracking has only a small effect on PSI relative to other parameters, its importance cannot be downgraded. The occurrence of cracking can lead to other problems (e.g., spalling, permanent deformation, or faulting) and/or increase their rate of development.

To properly treat and correct cracking, the cause must be identified. In the following subsections, possible correction techniques are discussed for each cause. Figure 6 is a summary table of the information presented in this section.

Opinion varies as to value of crack sealing and slurry seal, but for those desiring to seal cracks the following concepts are suggested.

Regardless of the cause of cracking, a program should be considered for sealing the cracks to prevent the entrance of water where water may prove detrimental to lower layers (*18, 19*). Large cracks (⅛ in. or greater) should be routed to a minimum width of between ⅜ in. and ½ in. and to a depth equal to the surface width. Both theory and tests indicate that a square-shaped sealant reservoir is required,



R - Reflection
C - Corner
L - Longitudinal
A - Alligator Cracking (Map Cracking)
T - Transverse
S - Slippage
RC - Random Cracking

Figure 5. Cracking patterns.

and that simply filling the crack from top to bottom will almost certainly result in a new crack (*20*). The routed crack should be cleaned by rotary brushes and/or compressed air and then filled. Crack sealers for existing portland cement concrete pavements are given in Table 1. Crack sealing of existing asphalt concrete is contained in a manual of The Asphalt Institute (*18*). The surface of the sealant should be sprinkled with a light sand to prevent traffic pickup (*18*). Over-filling of the cracks should be avoided because it will produce pavement roughness and an unsightly appearance. In areas where extensive small cracks are present in existing asphalt concrete pavements, a slurry seal or seal coat may be applied to the surface (*21*).

Cause: Excessive and Repeated Loadings

Excessive and repeated loadings can produce most of the cracking patterns shown in Figure 5. This cracking may be indicative of inadequate structural design, in which case an overlay designed in accordance with the concepts in Chapter Three may be necessary.

Solutions for Existing Portland Cement Concrete

Many times adverse groundwater conditions provide excessive water to the foundation soils, thereby reducing their load-carrying capability. Elimination of this water by a subgrade drainage system or by lowering the drainage ditch below the base course (Fig. 7) may increase the support capability of the the soil (*15, 22*). Details for drainage layout, filter material, etc., appear in Yoder (*15*) and the Armco "Handbook" (*22*).

Corner or longitudinal cracking may be caused by the loss of support resulting from subbase or subgrade pumping. Subgrade drains should be installed parallel to the edge of the pavement (Fig. 8) to remove the water (*15, 23*).

Another corrective measure is to increase the shoulder slope if surface drainage is being impeded. This will eliminate or reduce the amount of water infiltration into the longitudinal joint between shoulder and pavement (*22*). If this joint is open, it should be routed and filled with an appropriate sealant (*18*).

TABLE 1

CRACK SEALERS FOR PORTLAND CEMENT CONCRETE PAVEMENTS

MATERIAL	TYPE	SPECIFICATION		
		FEDERAL	ASTM	AASHO
Rubber-asphalt	Cold-applied	SS-S-156	D 1850	—
	Cold-applied mastic	SS-S-159b	D 1850	—
	Hot-applied	SS-S-001401	D 1190	M 173
Tar	Hot-applied	—	—	M 52–42

Another method of increasing the support capability of silty and clayey subgrades is the use of lime injection. With this method, holes are drilled at approximately 8-ft centers through the pavement structure and lime slurry is pressure-injected into the subgrade. The lime moves horizontally in the soil, stabilizing it and increasing its load-carrying capability. (This method should be distinguished from slab-jacking, which is described later.)

Solutions for Existing Asphaltic Concrete

The improvement of the subgrade drainage and the use of lime injection as described in the previous section are also applicable to existing asphaltic concrete pavements. In some cases it may be desirable to provide additional edge support.

Cause: Thermal Changes

Thermal changes are usually the cause of transverse reflection cracks in overlays. These cracks are unsightly, eventually produce roughness in the pavement, and allow moisture into the lower layers, possibly reducing the pavement's supporting power.

In most cases, existing portland cement concrete pavements have joints that can produce reflection cracking in the overlay material due to daily and seasonal temperature fluctuations (see Fig. 9). Asphaltic concrete pavements also develop transverse cracking caused by thermal changes (*24*); consequently, an asphaltic concrete overlay may reflect these cracks.

It should be recognized that an overlay will also be influenced by the established movement pattern of the existing pavement.

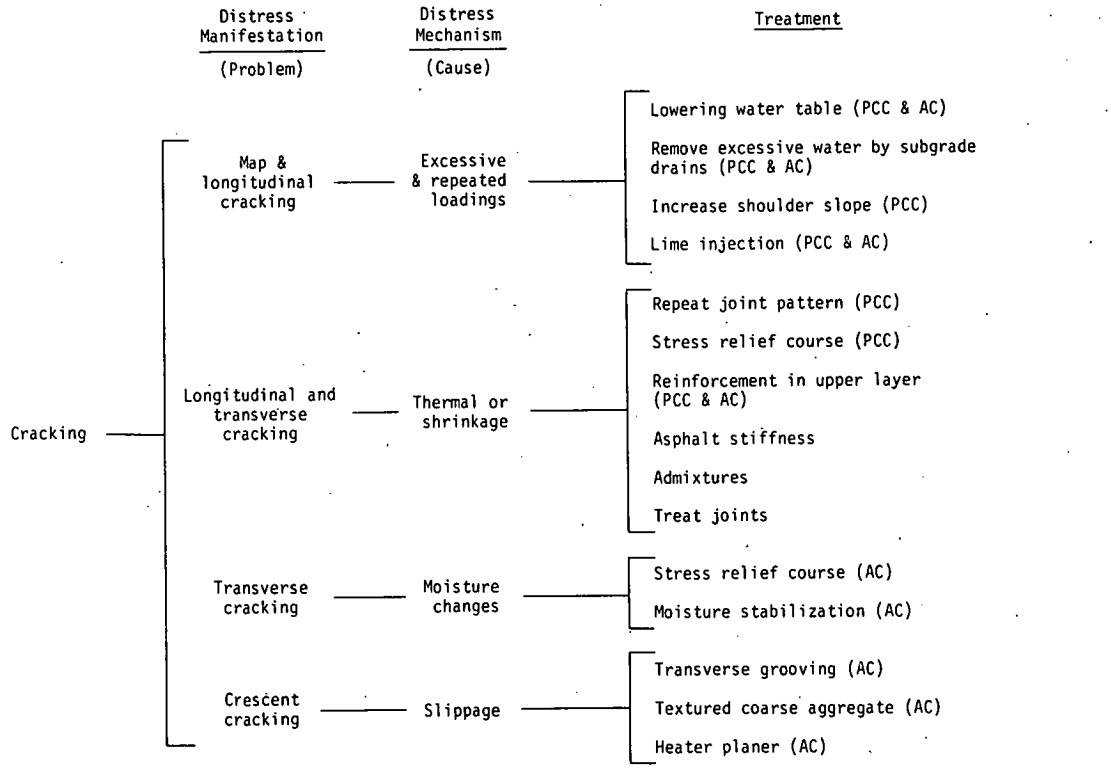


Figure 6. Treatments for various causes of cracking.

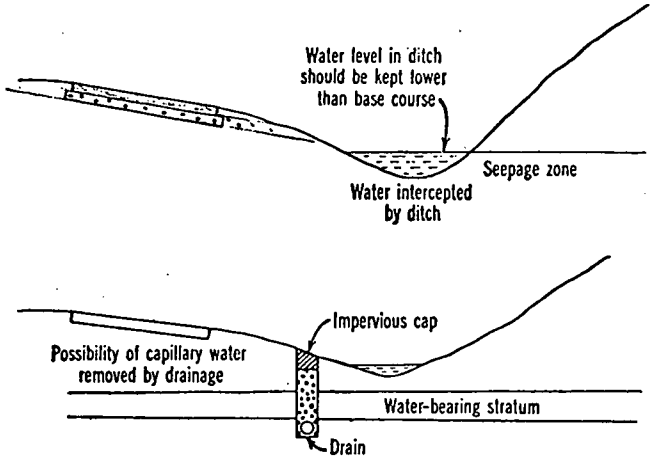


Figure 7. Examples of subgrade drainage to increase load-carrying capacity.

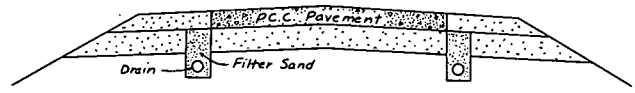


Figure 8. Example of subgrade drainage to prevent pumping.

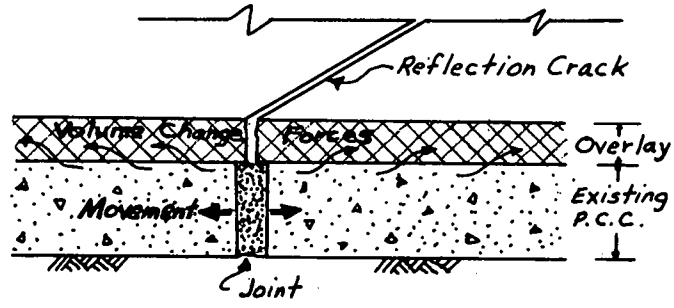


Figure 9. Phenomenon producing reflection cracking in overlays.

Solutions for Portland Cement Concrete Overlays

The detrimental effects of reflection cracking in portland cement concrete overlays may be reduced or eliminated by any one of the schemes shown in Figure 10. For the first case (Fig. 10a), the same joint spacing pattern may be used for the overlay; hence, no differential condition is created.

For the second case (Fig. 10b), where a different joint pattern scheme is used, a stress-relieving course may be used to absorb the volume change forces and prevent their transfer into the portland cement concrete overlay. The stress-relieving layer should be a material with a stiffness much less than the portland cement concrete and from 1 to 4 in. thick. Bituminous concretes with a mix design that provides a flexible material have performed satisfactorily in this capacity. For asphaltic concrete, the softer penetration asphalt cements (i.e., 85-100 or 120-200) are recommended.

In Figure 10c, a continuously reinforced concrete pavement (CRCP) has been placed directly on top of the existing portland cement concrete pavement. Sufficient additional longitudinal reinforcement should be placed in the CRCP overlay to carry the volume change forces developed by the existing portland cement concrete pavement (*25*). A thin asphalt leveling course can be placed over the existing pavement to correct grade and cross section and to serve as a bond breaker prior to constructing the CRCP overlay.

Solutions for Asphaltic Concrete Overlays

Many highway agencies have experimented with numerous methods of asphaltic concrete overlay construction. Table 2 is a list of these methods that was compiled from various references (*26-36*). Basically, all the methods may be categorized into one of four classifications, as given in the table. Classifications I and II are the ones generally used by most highway departments. The other two classifications are experimental and are not widely used.

Regardless of the method, there are a number of prerequisites that should be satisfied. Among the features to be considered in evaluating any method of controlling reflection cracking are the following (*32*):

1. Should eliminate or reduce reflection cracking.
2. Should not lessen strength, durability, etc., of the asphaltic concrete layer.
3. Should not add excessive cost.
4. Should not delay the construction process.
5. Should not require special skills.

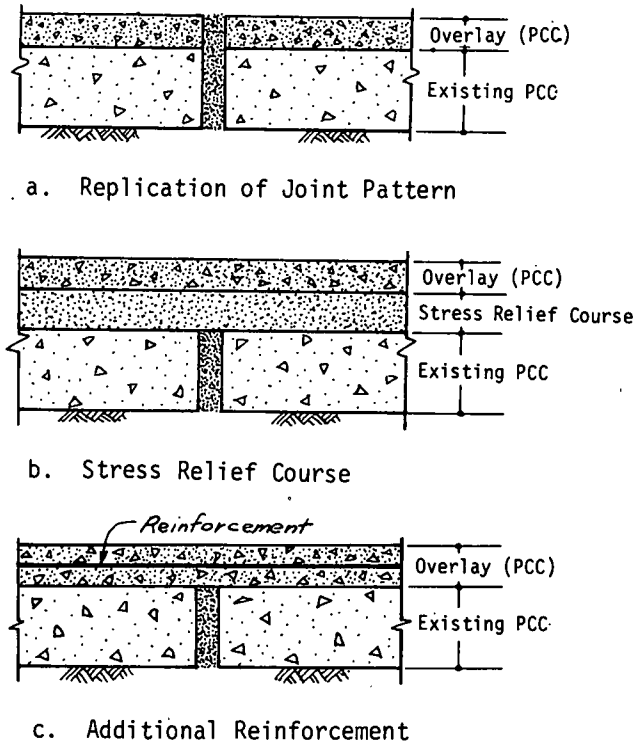


Figure 10. Methods of accommodating reflection cracking in portland cement concrete overlays.

TABLE 2

METHODS ATTEMPTED TO CONTROL REFLECTION CRACKING IN ASPHALTIC CONCRETE OVERLAYS

I. Increased Thickness of Asphaltic Concrete Overlay.
II. Special Treatment of Existing Portland Cement Concrete Pavement:
  a. Intermediate layer of granular base.
  b. Reseating the slab by rolling.
  c. Breaking slab into small segments.
III. Special Consideration in Asphaltic Concrete Overlay Design:
  a. Reinforcing.
  b. Asphalt hardness.
  c. Admixtures.
IV. Treatment of Joints and Cracks:
  a. Fill existing portland cement concrete joint with incompressibles.
  b. Break bond between asphaltic concrete and portland cement concrete.
  c. Replicate portland cement concrete joints by sawing the asphaltic concrete layer.

In the following paragraphs, the performance data available on these various methods are discussed. Unfortunately, there has not been any controlled experimental project where all these methods have been used in a factorial experiment design. Therefore, any comparisons across project lines are relative and are of limited value. Generally, a project contains only two or three of these methods. Some of these projects, where these methods have been compared, are discussed.

A degree of reflection cracking occurred on all studies reported, regardless of the overlay thickness used. This was true for thicknesses ranging up to 9 in. of asphaltic concrete (*26, 27*). Figure 11 indicates that an increase in thickness tends to reduce the amount of reflected cracking. It should be noted, however, that a roller was used to break the existing pavement.

Another factor that should be considered, in addition to amount of reflection cracking, is the width of the cracks. The references comparing thickness do not have these data available, but it could be hypothesized that the width would tend to be smaller for the thicker courses.

There are three methods of treating existing portland cement concrete pavements to prevent reflection cracking into an overlay: (1) use an intermediate layer of granular base (cushion course), (2) reseat the slabs by rolling, and (3) break the slabs into small segments. These are sometimes used in combination.

The Texas Highway Department has experimented with a cushion course over regular jointed slabs that had been broken into approximate 1-ft segments (*36*). Surveys over seven years indicated that the section where the cushion course was used was superior to the one where the asphaltic concrete pavement was placed directly on top of the concrete pavement.

The objective in seating slabs is to place the slab in contact with the base at all points, thereby increasing the load-carrying capacity. Only sufficient weight and roller applications are applied to the pavement to eliminate the "rocking slabs." In this approach, the continuity of the slabs is maintained. This method will reduce reflection cracking caused by excessive vertical movement of the rocking slab, but will have little effect on the magnitude of movement due to thermal changes.

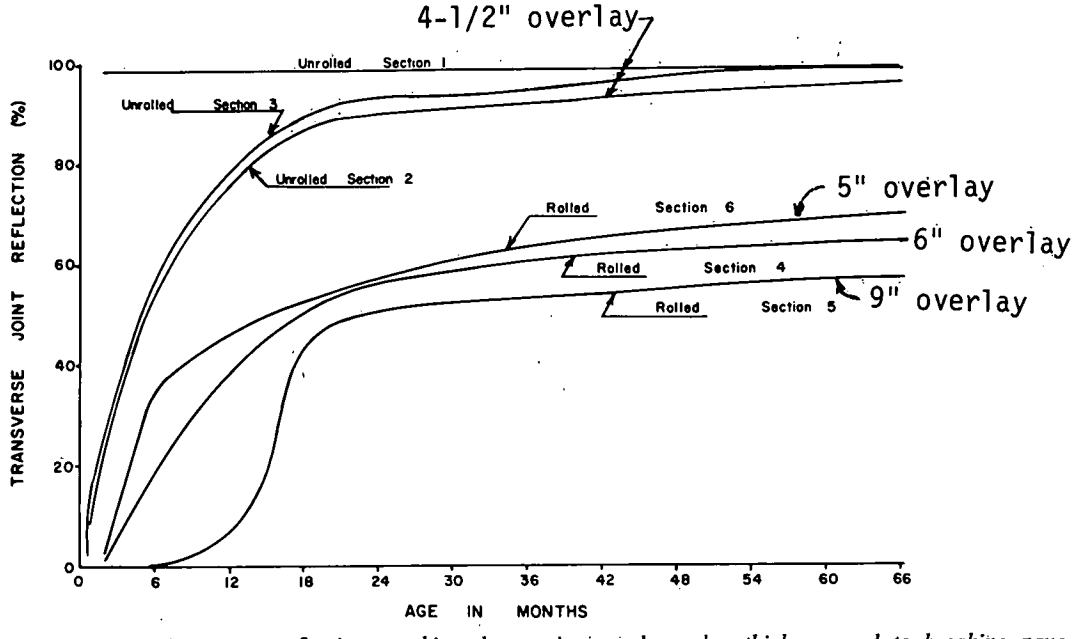


Figure 11. Reduction in reflection cracking due to increased overlay thickness and to breaking pavement by rolling. Source: Korfhage (26).

The Minnesota Highway Department has experimented with breaking a pavement with a 59-ton roller and placing 5 to 9 in. of bituminous overlay directly on top of it (*26*). Their data indicated that, where no rolling was used, 100 percent reflection cracking occurred, whereas approximately 70 percent reflection cracking occurred where pavement breaking was used for approximately equal thicknesses (Fig. 11). A test section constructed by the Florida Highway Department indicated that breaking the existing pavement into 3-ft pieces has practically eliminated reflection cracking (*29*).

There are three methods (Table 2) that may be considered in the design of the asphaltic concrete overlay: reinforcement, asphalt stiffness, and admixtures.

Massachusetts, Florida, California, Texas, New York, Illinois, and others have experimented with the use of welded wire mesh reinforcing in the asphaltic concrete overlay (*27, 29, 32, 37, 38, 39*). Experiments have varied from short strips placed directly over the transverse joints or cracks to continuous reinforcement. The degree of success in the prevention of reflection cracking varied from good to poor. Due to the variability in results, a project involving the use of wire mesh reinforcing is considered to be experimental construction by the Federal Highway Administration.

Polyester fabric reinforcement of asphaltic overlays has been used recently in several locations (*40*). Early results seem to indicate that a reduction in reflection cracking will be obtained; however, additional time is necessary to assess performance completely.

The Iowa Highway Department experimented with asphalt stiffness by using two different grades of penetration (*41*). Table 3 is a comparison of reflection cracking with 80-penetration and 115-penetration asphalt at an age of 51 months. Note that the softer asphalt has reduced the reflection cracking to approximately one-half of that in the stiffer asphalt (80 penetration). From this study it may be surmised that the asphalt stiffness can play a significant role in the amount of reflection cracking.

Several projects have used additives in the asphaltic concrete in an attempt to increase the stretchability of the mix and prevent cracking at the underlying portland cement concrete joints (*28, 31, 33*). The Massachusetts Highway Department experimented with emulsified rubber asphalt, synthetic rubber, and natural rubber crumbs (*28*). In a proportion of 5 percent by weight (of asphalt) there was no difference in performance between sections with additives and those without. Ninety percent of the possible reflection cracking occurred in all sections within six years (Fig. 12). Note that there is no marked difference in the crack widths for the additives. A similar study by the Road Research Laboratory in England (*30*) also indicated that rubber in the asphalt did not reduce cracking.

Several agencies have experimented with the use of asbestos additives in asphaltic concrete overlays, but results have been inconclusive.

Three methods of joint treatment are mentioned in Table 2. The limited data regarding each of these methods indicate that they are not too beneficial.

On one project, incompressibles were placed in the joints in an attempt to reduce the movement and, hence, reflection cracking. Results were disastrous—blowups occurred at every fifth joint.

TABLE 3

EFFECT OF ASPHALT PENETRATION ON REFLECTION CRACKING [a]

SECTION	ORIG. ASPHALT PENE. AT 77° F	REFLECTION CRACKING AT 51 MONTHS (%)
A	80	76[b]
B	79	74
C	115	37

[a] Asphaltic concrete pavement (3 in.) placed directly on jointed concrete pavement. On all pavements, the first course (1½ in.) was 80-penetration asphalt.
[b] Interpolated.

On several projects a bond breaker in the form of a smooth plate or expanded metal strips was used between the portland cement concrete pavement and the asphaltic concrete overlay. This method was used in an attempt to have a longer gauge length for the asphalt to absorb joint movements. An experimental test section by the Road Research Laboratory (*30*), using this method, indicated that the reflection cracking was reduced slightly, but those sections that did crack had severe disintegration.

One state has experimented with controlling reflection cracking by sawing ⅜-in.-wide joints in the asphalt concrete overlay at the existing portland cement concrete joints (*34*). Depth of the sawed joints varied from ½ in. to 1¾ in., and all joints were filled with a hot rubber asphalt sealant. This procedure does not reduce reflection cracking, but does substantially extend the maintenance-free life of the overlay.

Rubber additives and softer asphalts have been used for overlays on bituminous pavements to reduce reflection cracking. The designer should take precautions in the mix design to develop mixes that are not as susceptible to temperature cracking (*42, 43*).

Shrinkage cracks are similar to thermal change cracks and, hence, the method of treatment is similar to that described previously.

Cause: Moisture Changes

Moisture changes in foundation soils also produce random longitudinal and transverse cracks in existing asphaltic concrete pavements. (This phenomenon is generally not associated with portland cement concrete pavements.) The moisture change cracks are generally associated with a drying action or freezing of the natural soils, especially those with high Plasticity Indexes. These cracks may be prevalent in arid or frost areas where droughts or freezing can produce considerable shrinkage beneath the pavement and deep within soil strata (*42*).

The cracks allow excessive amounts of moisture to enter the pavement structure during rainfall, causing a loss of supporting capability and differential vertical volume

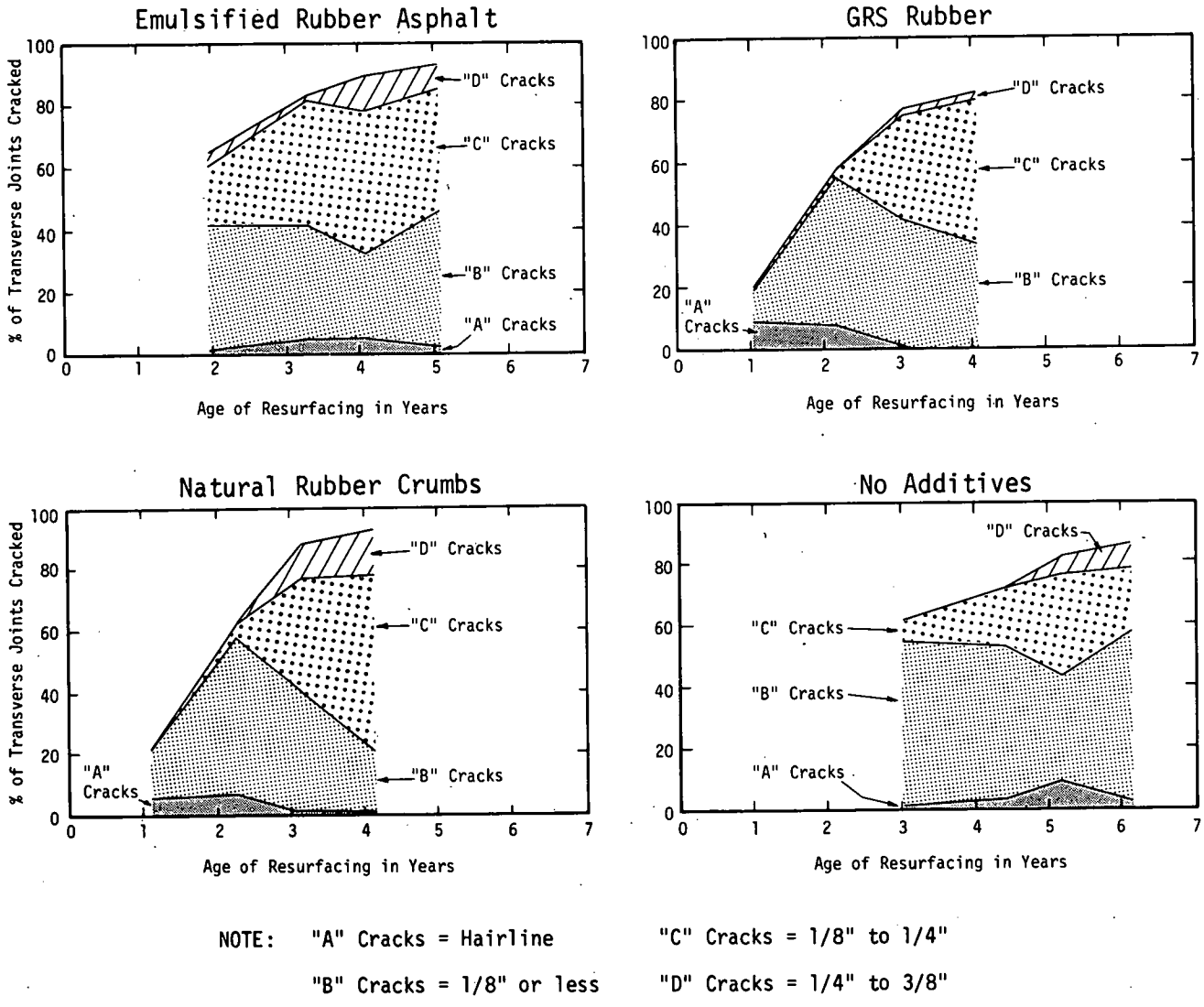


Figure 12. The effects of additives on reflection cracking. Source: Roggeveen and Tons (28).

change. Observations of pavement failure in Wyoming (*44*) have led to the belief that moisture from melting snow collects in pavement cracks. These conditions then lead to additional cracking and distortion (roughness). As a preventive measure, these cracks may be sealed during conventional maintenance operations. If the cracks are extensive, a seal coat or surface treatment after sealing should be considered (*21*).

One possible method of reducing moisture change cracks in asphaltic concrete overlays is to use a stress-relieving course similar to that shown in Figure 10b. The stress-relieving course should be bound with an asphalt cement which has considerable capability for taking strain. This course also serves as a moisture barrier to prevent the formation of a moisture-carrying layer between two impermeable layers. It should be pointed out that an excessive treatment to prevent cracks due to moisture changes may result in two independent layers, which might lead to other types of cracks, such as those caused by excessive and repeated loadings.

Cause: Slippage

Slippage cracks are produced by horizontal forces on the pavement, are normally crescent-shaped, and are generally associated with thin layers of asphaltic concrete. These cracks are found in areas where turning, stopping, or starting movements occur in combination with poor bonding of layers at an interface.

Solutions for Asphaltic Concrete Overlays on Existing Portland Cement Concrete

Slippage cracks may be prevented either by using a thicker asphaltic concrete overlay or by improving the bond between existing portland cement concrete and asphaltic concrete overlay with one or more of the following techniques.

The existing surface should be cleansed of road oils and other foreign materials by sweeping, wire brushing, a high-pressure water spray, or another method. In areas where excessive horizontal forces are expected, the existing pavement may be grooved transversely or roughened. Grooves should be ¼ in. to ⅜ in. wide, ¼ in. deep, and spaced at 1-in. to 1½-in. intervals. A rough surface may be obtained with a continuous row of saw blades and only a shallow cut.

Another alternative is the use of a textured and angular coarse aggregate, a slag aggregate, or a lightweight aggregate to produce a high coefficient of friction. This will provide better bonding at the interface.

Regardless of the type of treatment used, the existing surface should be clean and a light tack coat should be applied before the asphaltic concrete is placed (*18*).

Solutions for Asphaltic Concrete Overlays on Existing Asphaltic Concrete

Slippage may be prevented either by using a thicker asphaltic concrete overlay or by improving the bond between existing asphaltic concrete and asphaltic concrete overlay. The existing surface should be cleansed of oils and other foreign materials by sweeping, a high-pressure water spray, or another method. Another alternative is to use a heater planer to simultaneously cleanse and roughen the surface. If slippage cracks are present in the existing pavement, the area should be removed and replaced using a liquid asphalt tack coat (*18*).

PROBLEM: SPALLING

Spalling is generally associated with cracks or joints in portland cement concrete pavements. All the distress mechanisms in Figure 4 can produce spalling if incompressibles fill the opening or if conditions exist that produce high compression and shear stresses at the crack or joint.

To reduce the occurrence of spalling to a minimum, good joint and crack cleaning and sealing procedures should be followed to avoid infiltration of foreign materials. As a corrective measure and prior to an overlay, spall repairs are usually made with bituminous mixtures. Epoxy resin concrete, portland cement concrete bonded with a portland cement-sand grout, or epoxy resin grout have also been used. All deteriorated concrete should be removed and the area should be thoroughly cleaned prior to placing the patching material.

For nonbituminous patches, a vertical saw cut about 1 in. deep is often used around the periphery of the spall to eliminate feather-edging. The spalled surfaces should be primed with an epoxy resin grout or a portland cement-sand grout (*45, 46*). Epoxy resin materials should be used only with aggregates to avoid failure in bond due to extreme differences in thermal properties.

For bituminous concrete repairs, the edges of the area should be squared and cleaned. The area is then sprayed with a 0.05- to 0.15-gal/sq yd tack coat of SS-1 or CSS-1 asphalt emulsion diluted with equal parts of water (*18*). The hole is then filled with a dense-graded asphaltic concrete that is compacted flush with the surrounding pavement (*47*).

Where spalls are along joints, care should be taken to maintain the joint with filler materials during construction of the patch. The filler material is later removed and the groove is cleaned and sealed.

PROBLEM: PERMANENT DEFORMATION

Permanent deformation (Fig. 4) can take several forms, as shown in Figure 13. Each of these produces a change in the pavement surface which causes an increase in roughness or slope variance (refer to Eqs. 1 and 2), thereby reducing the PSI of the pavement. The engineer must first define the type of distress mechanism (Figs. 13 and 14), and then correct the defect, as discussed in the following sections.

Cause: Time-Dependent Deformation

The time-dependent deformation distress mechanism produces transverse corrugations (sometimes called washboarding) across the pavement (Fig. 13a). This is associated with asphaltic concrete pavements and results from plastic movements in the asphalt layer due to insufficient mix stability. Lack of stability can be attributed to a number of factors (e.g., too much asphalt, lack of aggregate texture, high proportion of fines, soft asphalt) (*48*). These deformations are generally found where horizontal forces are imparted to the pavement due to stopping, starting, or turning movements.

The distressed area may be corrected by removing the asphalt layer and replacing it with a stable mix. Stability values of 35 and 750 are minimums recommended for the Hveem and Marshall Tests, respectively (*47*). Higher minimums should be considered for bus stops and roads with high volumes or heavy traffic. Asphaltic concrete for an overlay should also meet these requirements.

An alternative temporary measure is to use a "heater planer" to remove the high spots, thereby producing a smooth surface. This should be followed by a seal coat to waterproof the surface.

Cause: Densification

Rutting (Fig. 13b) occurs in asphaltic concrete pavements. The ruts in the wheel patch develop under traffic due to differential compaction of one or more of the pavement layers or due to lateral displacement of material.

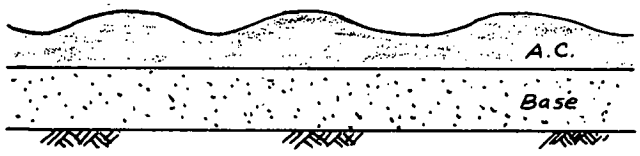
The ruts should be coated with a light tack coat (0.05 to 0.15 gal/sq yd of SS-1 emulsion diluted with equal parts of water) and filled with a dense graded asphaltic concrete. After compaction, a seal coat should be placed over the patch to prevent the entrance of water, or an overlay should be placed on the entire pavement width.

Cause: Consolidation

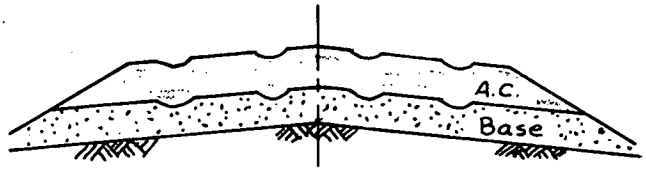
Consolidation causes the depressions shown in Figure 13c and is attributed to settlement in the natural soils or within the embankment.

Solutions for Existing Portland Cement Concrete

For existing portland cement concrete pavements, the pavement structure may be restored to grade and cross section by an asphaltic concrete leveling course or by slabjacking.



a. Edge View of Pavement Structure Showing Transverse Corrugations



b. Cross Section of Pavement Structure Showing Rutting Due to Densification

c. Edge View of Pavement Structure Showing Depression

d. Edge View of Pavement Structure Showing Effect of Swelling

Figure 13. Various types of permanent deformation.

For leveling work, a dense asphaltic concrete patch should be used as described in the previous section on "Densification." The leveling course should be placed as shown in Figure 15, with the shortest lift being placed first and with each succeeding layer extending over and covering the preceding layer. This procedure prevents bumps from occurring at the beginning and end of each layer.

Loss of surface grade due to settlement of the foundation material may also be corrected by slabjacking. The technique involves the forcing of a grout or slurry mixture under the slab to fill voids and lift the slab. Holes are drilled through the portland cement concrete pavement at spacings to provide 3 to 5 sq yd of surface area for each hole. A 12- to 18-in. minimum clearance should be provided between the holes and a pavement edge or joint. The mixture is then forced into the hole until the slab is raised to grade. A wide variety of mixtures have been used, depending primarily on the availability of local materials, but it is recommended that a cementing agent of some type be used to provide additional support strength. The viscosity of the mixture will depend on the nature and size of the voids. Care must be taken to avoid excessive pressures at critical locations so that overstressing and cracking of the slab do not occur (*49, 50, 51, 52*).

Solution for Existing Asphaltic Concrete

Only the leveling course described in the preceding section is applicable to existing asphaltic concrete pavements.

Cause: Swelling and Frost Heave

Swelling (Fig. 13d) may occur with asphaltic concrete or portland cement concrete pavements. It is usually caused by swelling of expansive clays due to moisture increases (*15, 53*).

Frost heave is caused by the freezing of moisture in the subgrade or base layers and may occur with asphaltic concrete or portland cement concrete pavements.

Solutions for Existing Portland Cement Concrete

The high point on the grade line at the center of swell may be deemphasized by transition patching or reduced by bump cutting. For transition patching, an asphaltic concrete as described under "Densification" may be used as shown in Figure 16. A bump cutter (a series of saw blades placed on a shaft to form a cutting cylinder) may be used to remove all or a portion of the high point to reduce roughness.

The correction of serious frost heaves involves removal of existing pavement and provision of uniformity of soils and drainage within the depth of zone of freezing.

Solutions for Existing Asphaltic Concrete

For swells on asphaltic concrete pavements, the high point on the grade line may be smoothed by transition patching, as described previously, or by use of a heater planer. If a heater planer is used, too much surface material should not be removed, thereby reducing the load-carrying capacity of the pavement structure. In some cases it may be necessary to remove the pavement and swelling soil and install a moisture barrier and a deep patch.

Frost heaves should be corrected as described previously.

PROBLEM: FAULTING

Faulting may result from several causes, as indicated in Figure 4, and may be identified by a vertical difference of two adjacent points. Faulting is found in portland cement concrete pavements only, at a joint or crack. Corrective measures include the use of slabjacking, grinding (bump cutting) or a thin asphaltic concrete wedge. Because most faulting today is caused primarily by loss of support by erosion under the pavement, the corrective measures should include provisions to drain water from the base.

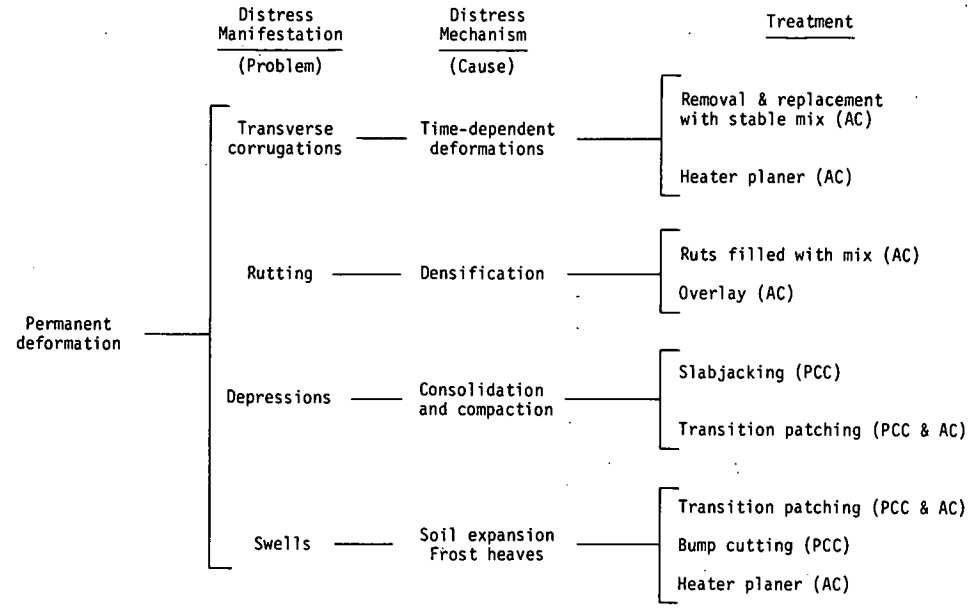


Figure 14. Treatments for various causes of permanent deformation.

PROBLEM: STRIPPING

Stripping is the separation of the asphalt from aggregate particles in the presence of moisture. Thus, the matrix of the mix is lost, first by removal of the asphalt binder and, in the latter stages, by loss of the "loosened" aggregate.

Stripping of the aggregates will destroy the surface profile of the pavement, thereby creating roughness. Hence, the slope variance will increase with a resulting lowering of the PSI rating.

Cause: Adhesion

The loss of adhesion may be attributed to insufficient asphalt content and to water replacing the asphalt coating on an aggregate because of its hydrophilic properties. Insufficient asphalt content may be prevented by sound mix design principles. Loss of adhesion because of hydrophilic properties may be prevented by selecting aggregates that are less hydrophilic and have good surface texture, minimum surface coatings, proper polarity, and adequate porosity (*10*). The stripping problem may be minimized by providing for a denser (less porous) mix through improved gradation (*54*) and through the use of anti-stripping agents.

Several tests are used to measure loss of adhesion. However, none of these tests is considered positive for all types of adhesion problems. Those in common use are the Coating and Stripping of Bitumen-Aggregate Mixtures (stripping test), ASTM D1664; the Effect of Water on Cohesion of Compacted Bituminous Mixtures (immersion compression test), ASTM D1075; and the moisture vapor susceptibility test (*55*) developed by the California Division of Highways. The science of determining the cause and cure of stripping is not well defined in the literature, and research is under way to further explore the problems and possible corrective actions. However, it is generally believed that if the stripping is caused by moisture entering the mix from the top

Figure 15. Proper placement of level-up wedges for distress produced by consolidation.

Figure 16. Proper placement of level-up wedges for distress p duced by swell.

surface of the roadbed the effect of water can usually be stopped or greatly reduced by applying some type of surface treatment or by an overlay. If the water action is coming from moisture beneath the pavement, as has been reported in some instances, only a properly designed overlay will provide adequate support for the traffic because the original mix may eventually revert to gravel.

However, in many cases the best correction technique is removal and replacement.

Cause: Chemical Reactivity

Some aggregates and asphalts exhibit an adverse chemical reaction that produces stripping. As a preventive step, the tests mentioned previously may give an indication of this

reaction, and, of course, previous experience with the asphalts and aggregates should be considered. The best corrective technique is removal and replacement of the problem areas.

PROBLEM: RAVELING

Raveling of asphaltic concrete pavements is the progressive separation of aggregate particles in the pavement. Usually, the fine aggregate comes off first and leaves little pock marks on the surface. As the erosion continues, larger particles are broken free and the pavement surface has a rough and jagged appearance.

Cause: Abrasion by Traffic and Degradation of Aggregate

Some aggregates, depending on mineral composition, may exhibit degradation from traffic and the smaller particles with insufficient bond will separate from the matrix. This may be prevented by selecting aggregates with sufficient hardness to resist crushing under traffic. The placement of an overlay will reduce the stresses in the existing layer, thereby reducing the crushing action.

Cause: Durability of Binder

If undesirable changes occur in an asphalt—e.g., hardening (reduction in penetration or ductility)—the performance will be reduced. Preventive measures suggested are (*48*):

1. Use as soft an asphalt as possible without reducing stability below an acceptable level.
2. Use as high a percentage of asphalt as possible.
3. Prepare mixtures at lowest practical temperatures.
4. Compress all mixtures thoroughly so that they will be impermeable to air.

Corrective procedures for reducing the effect of durability are the use of an overlay or of seal coats.

Cause: Insufficient Asphalt

Insufficient asphalt content may be prevented by sound mix design and adequate control of the mixing operation. The best technique for correction of isolated sections is removal and replacement. If the condition is widespread, a properly designed overlay may be necessary.

PROBLEM: SCALING

Scaling is the progressive disintegration and loss of a portland cement concrete wearing surface. The resulting roughness will lower the PSI rating.

Cause: Chemical Reactivity

Chemical reactivity may be attributed to a reaction between the aggregate and the cement. Susceptible aggregates are attacked by sodium oxide and potassium oxide which results in the formation of silica gel reaction rings around the particles (*56*). The reaction causes the aggregate to expand, thus resulting in scaling. This reaction may be minimized by specifying low alkali cement (*57*). Other procedures include using different aggregates, varying the aggregate gradation, and using admixtures.

An overlay of asphaltic concrete or bonded portland cement concrete may be used as a corrective procedure. Prior to resurfacing, the existing surface should be cleansed of all loose and poorly bonded material. This may be accomplished by use of a scarifying machine, jackhammers, or a chipping tool (*18, 58*).

Cause: Abrasion by Traffic

Abrasion by traffic comes from the normal wear of tires and also from the use of chains and studded tires. An overlay of asphaltic concrete or bonded portland cement concrete may be used as a corrective procedure.

Cause: Freeze-Thaw Action

The freezing of water and actions of deicing chemicals in the voids near the surface of portland cement concrete pavements cause a rupture of the cement gel and consequently scaling. This scaling may be prevented by use of air-entraining admixtures in the original construction. Another preventive measure is application of linseed oil to the surface. An overlay of asphaltic concrete or bonded portland cement concrete may be used as a corrective procedure.

CHAPTER THREE

OVERLAY DESIGN PROCEDURES

When an evaluation of the existing pavement structure shows that additional thickness is required, one of several methods may be used to design an overlay. These methods, and their limitations, are discussed briefly herein. Use of these procedures implies application of the proper construction specifications, adequate drainage, etc. Without such application, performance may be considerably different than expected.

A common practice is to design the pavement structure for the minimum support conditions and apply this thick-

ness throughout the project. However, the available money can be used better if the thickness is varied to meet the different conditions encountered on the project.

AVAILABLE METHODS AND CRITERIA OF APPLICATION

The basic approach used in the present design procedures may be categorized as (1) a layer analysis, or (2) a monolithic field evaluation.

In the layer analysis, a strength value is assigned to each layer, and the behavior of the total pavement structure is predicted using a mathematical model. The strength values for each of the layers may be estimated or obtained by laboratory measurements. The model used to predict performance may be based on theory or experience.

With the monolithic approach, the in-place load-carrying capability of the existing pavement structure is measured and this is used as a basis for estimating future requirements. All of the existing monolithic methods use surface deflection as a measure of the load-carrying capability.

In Figure 17 a number of the existing U.S. and Canadian structural design methods are categorized as to their approach, layer or monolithic, and by overlay type. The figure shows that a choice is available for only 4 of the 10 combinations. The portland cement concrete over asphaltic concrete combination for the layer category may be handled by the agency's portland cement concrete design procedure or other available methods (*25, 59, 60*). The same comment is also applicable to the asphaltic concrete over asphaltic concrete combination.

Another pertinent factor is the extent to which each method considers the variables of loading, environment, etc. In Figure 18, each design method presented in Figure 17 may be examined in terms of its capability for taking into account the variables. The designer may select the most appropriate method based on his needs. Notice that none of the methods provides a wear-out factor to account for the past history of fatigue, disintegration, etc. The basic premise of some methods is that the existing pavement retains the load-carrying capability existing at the time of deflection measurements for the duration of the overlay life. The designer should be aware of this assumption and take it into consideration in designing impermeable surfaces for roads having only an untreated granular or soil surface.

The full meaning of this assumption can be seen in Figure 19. This is a typical fatigue diagram for a material, with stress plotted against number of applications. In general terms, the lower the stress in a material the more stress repetitions required for failure. Therefore, if a material is subjected to a number of applications (n_1) at a given stress level (σ_1), a portion of its life is used up (n_1/N_1). Even if the stress level is lowered because of an overlay (σ_2), the remaining life of the existing layer is still less than unity.

ASPHALTIC CONCRETE OVER ASPHALTIC CONCRETE—MONOLITHIC EVALUATION

Each of the following four methods uses a field evaluation of the in-service pavement structure to estimate the asphaltic concrete or combination of asphaltic concrete and crushed-stone overlay required over an existing asphaltic concrete pavement. Each of these methods uses Benkelman beam measurements of surface deflection with a single-axle wheel load, but the magnitude of wheel load differs. If correlations with other methods of deflection measurements (such as Dynaflect or Road Rater) are used, caution should be exerted to ensure that the correlations are based on similar soil conditions (*8*).

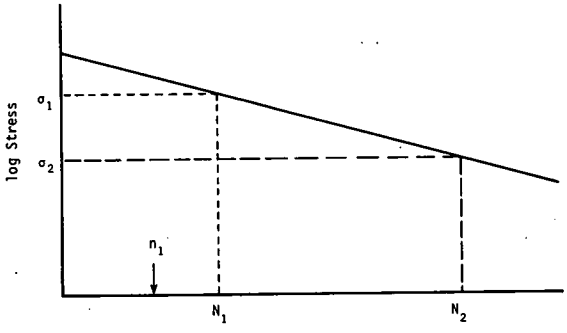
	AC/AC	AC/PCC	PCC/AC	PCC/PCC	AC/Composite
Pavement Layer Analysis	1. AASHO 2. Corps of Eng. 3. Asphalt Inst. (Also refer to agency procedure for A.C. pavements	1. AASHO 2. Corps of Eng. 3. Asphalt Inst.	(Refer to agency procedure for P.C.C. pavements)	1. Portland Cement Association 2. Corps of Eng.	
Monolithic Pavement Analysis	1. Oklahoma 2. California 3. Asphalt Inst. 4. R.T.A.C. (C.G.R.A.)				

Figure 17. Available pavement design methods for various overlay types.

	Oklahoma	California	Asphalt Institute	R.T.A.C.	Corps of Engineers	AASHO
1. Wheel Loads (External)						
a. Magnitude	✓	✓	✓	✓	✓	✓
b. Repetitions		✓	✓	✓		✓
c. Placement (lane, etc.)		✓	✓			✓
d. Growth	I	✓	✓	I	✓	✓
2. Volume Changes (Internal)						
a. Temperature						
b. Shrinkage						
3. Environment	✓		✓	I		
4. Statistical Variations		✓	✓	✓		
5. Wear-out						

I = Indirectly

Figure 18. Variables considered by each design method.



Miner's Hypothesis:

$$\frac{n_1}{N_1} \leq 1 \quad \text{to prevent failure}$$

$$\frac{n_2}{N_2} \leq R_L = 1 - \frac{n_1}{N_1}$$

N_1 = the allowable number of load repetitions on the pavement at the stress level (σ_1) prior to the overlay;

N_2 = the allowable number of load repetitions on the pavement at the stress level (σ_2) anticipated after the overlay;

n_1 = the actual applications experienced during the past pavement history;

n_2 = the number of load applications anticipated on the pavement in the future.

Figure 19. Fatigue loss of materials.

Oklahoma Method

Figure 20 is a flow chart summarizing the Oklahoma method of overlay design. Actually, the design encompasses three separate methods that are used to develop an overlay thickness without assigning weighting factors to the answers derived from each. The premise of the design is to provide a Present Serviceability Index of 2.0 after 20 years of service. The three methods use a condition survey, deflection criteria, and a pavement component analysis using the flexible pavement design procedure (*61*). Because the last method does not fall into the monolithic approach, it is not considered further here.

For the first method, data obtained from a detailed pavement condition survey are used to assign a pavement rating that ranges from 0 to 100 percent (i.e., failed to excellent). The depreciation (100 minus the rating) and the pavement age, counting from the last major rehabilitation or new construction, are entered in Figure 21, and the intersection gives the expected design life. The expected design life curve is then used to determine the depreciation at an age of 10 years. This information can be used with Figure 22 to estimate the Benkelman beam deflection. The required overlay thickness is then determined through the deflection method described in the following paragraph.

The second method (Fig. 20) consists of measuring the deflection along a roadway with a 9,000-lb wheel load. The deflections are averaged for areas of similar performance, and the average is adjusted for seasonal variation. The corrected deflection value is used to determine the required overlay thickness of asphaltic concrete from the nomograph in Figure 23. The amount of overlay is a function of the average deflection greater than 0.022 in. This limiting value was derived from a field study of in-service pavements which found the maximum allowable pavement deflection, with a 9,000-lb wheel load, should not exceed 0.037 in. for a pavement to perform satisfactorily for 20 years (*62*). A consideration of the fact that the design wheel load for Interstate highways is 15,000 lb, and that a relationship between deflections for 9,000- and 15,000-lb wheel loads had been established, led to the value of 0.022 in.

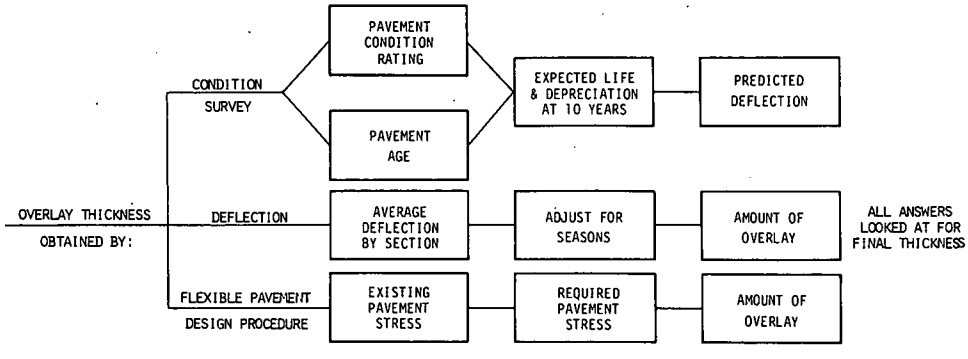


Figure 20. Oklahoma overlay design procedure for flexible pavements.

California Method

The California method was developed using observations of in-service overlayed pavements, and principles from the flexible pavement design procedure (*63*). The premise of this method is that a pavement structure has a limiting deflection which is a function of the surface thickness and number of wheel-load applications. For this method, the pavement deflection, under a 7,500-lb wheel load, is measured at intervals along the roadway. The deflection profile is separated into sections of fill, cut, cracked, uncracked, etc., or areas of supposedly similar deflection based on visual conditions and changes in structural section. For each of these sections, an 80-percentile deflection value is obtained (a value that is exceeded in only 20 percent of the cases). The 80 percentile is used to avoid placing too much emphasis on isolated conditions by using the maximum value or under-designing through use of the average.

The number and axle-load magnitudes expected during the analysis period are estimated. These data are then used with the following formula to compute the total 5,000-lb equivalent wheel loads:

$$EWL_{5000} = \sum_{i=1}^{n} e_i N_i \qquad (3)$$

in which

e_i = 5,000-lb equivalent wheel-load factor for the ith group; and

N_i = number of wheel loads in the ith group.

The appropriate wheel-load factors for each load and a method for developing the number of wheel loads appear in the "Highway Design Manual of Instruction" (*64*).

Next, a limiting deflection value based on the existing pavement structure thickness and the number of equivalent wheel loads expected during the design life is selected from Figure 24. This value is subtracted from the 80 percentile deflection and expressed as a percentage reduction, R, as follows:

$$R = \frac{d_1 - d_2}{d_1} \times 100 \qquad (4)$$

in which

d_1 = surface deflection of existing pavement structure under 7,500-lb load, inches (Benkelman beam, WASHO Procedure); and

d_2 = deflection of the surface after an additional layer of material, inches.

The percentage reduction is used with Figure 25 to determine the increase in gravel equivalent. This can then be related with traffic conditions to determine the overlay thickness required. No attempt is made with this method to quantitatively measure the condition of the existing pavement.

The extent and nature of pavement distress is considered in final selection of overlay thickness. Depending on the severity of alligator cracking, a decision must be made as to whether the existing asphaltic concrete surfacing will act as a bonded or unbonded flexible layer.

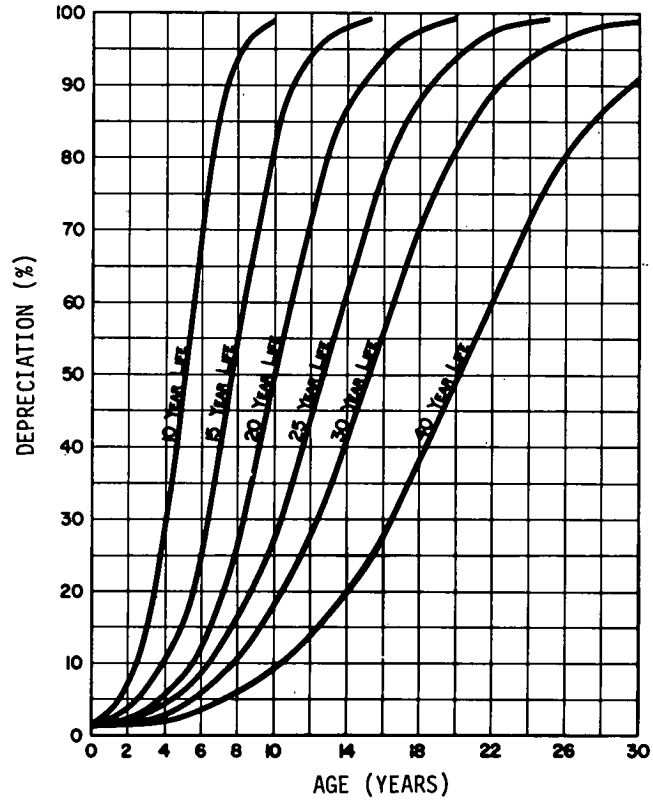


Figure 21. Life curves for flexible pavements—Oklahoma.

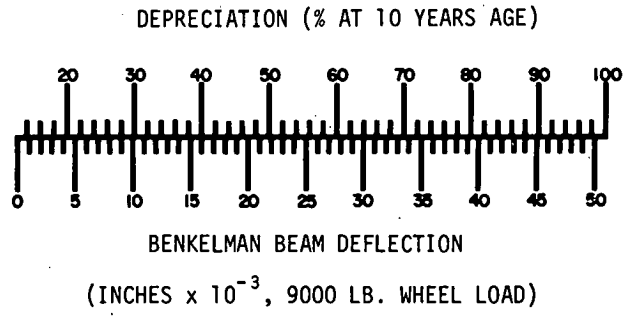


Figure 22. 9,000-lb wheel load deflection versus depreciation for Interstate pavements (flexible)—Oklahoma.

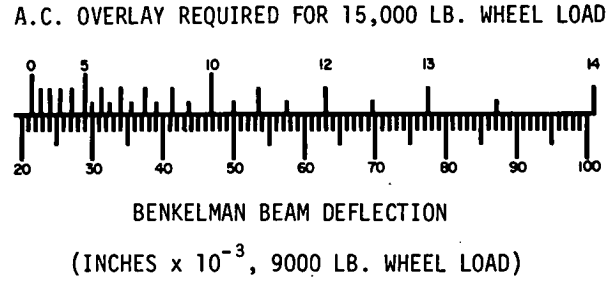


Figure 23. Asphaltic concrete overlay requirements based on Benkelman beam deflection—Oklahoma.

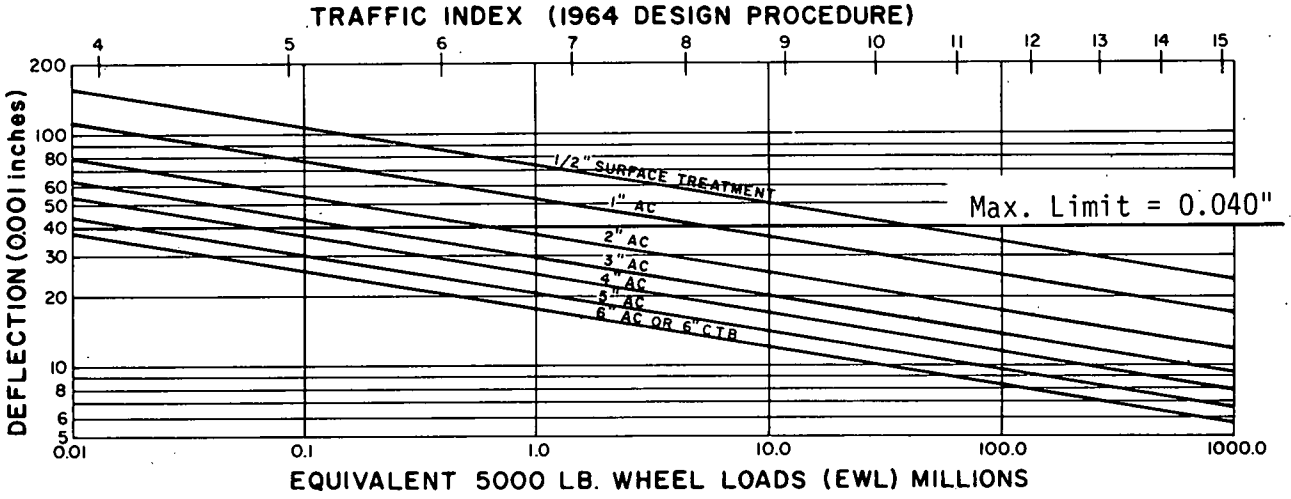


Figure 24. Variation in tolerable deflection, based on asphaltic concrete fatigue tests—California.

The Asphalt Institute Method

The Asphalt Institute monolithic method is also based on the premise that a pavement structure has a limiting deflection that is a function of wheel-load applications (*65*). The first step is to obtain a design traffic number that is the function of the initial average daily traffic, percentage of heavy trucks, average gross weight of heavy trucks, single-axle load limit, and character of the traffic. The next step is to obtain a series of random deflection measurements (minimum of 20 per mile or 10 per test section). These data are used to compute the mean and standard deviation of deflection for a section of road or test section, and a representative rebound deflection value under a 9,000-lb wheel load is computed as follows:

$$d_2 = (\bar{X} + 2S)fc \qquad (5)$$

in which

$\bar{X}$ = arithmetic mean of the adjusted individual values;
S = standard deviation of deflection of test section;
f = temperature adjustment factor; and
c = critical period, adjustment factor ($c = 1$, tests were made in the most critical period).

The temperature adjustment value in Eq. 5 may be obtained from Figure 26. The critical period adjustment factor may be obtained by measuring either at the critical period ($c = 1$) or from a continuous record of deflection measurements by taking the ratio of the critical period deflection to the deflection for the date of test. Eq. 5 provides for 97 percentile; i.e., 3 percent of the deflections are greater than the design value. The representative rebound deflection and the design traffic number may then be used with Figure 27 to obtain the overlay thickness in inches of asphaltic concrete.

Roads and Transportation Association of Canada Method

The RTAC method (*66*) is similar to The Asphalt Institute method. A Benkelman beam rebound value is established using a procedure similar to that described previously and by Eq. 5. Next, a spring rebound deflection value between 0.030 in. and 0.050 in. is established as a design value for pavements that have average daily traffic volumes of 1,000 or more and 10 percent trucks. The RTAC recommends that the choice of design value in this range be based on cost and funds available. In addition to this criterion, it is recommended that the larger value (0.050 in.) be associated with existing asphaltic concrete thicknesses less than 2 in., and the lower value (0.030 in.) be used with existing asphaltic concrete thicknesses in the range of 5 in. The desirable design value and the measured rebound value (Eq. 5) are then used with Figure 28 to establish additional thicknesses of granular base course. This thickness value may be converted to asphaltic concrete using a layer equivalency factor of 2 to 1; e.g., if 6 in. of granular base is obtained from the figure, 3 in. of asphaltic concrete would be used. If a combination of granular base and asphaltic concrete is used, the recommended minimum thickness for the granular base is 4 in.

Evaluation of Methods

A desirable evaluation would be a comparison of all the results for the same problem, but this is difficult because the input parameters differ. One study comparing the California and Oklahoma methods indicates that the overlay thicknesses for the California method are less for light traffic and greater in most cases for the heavier traffic when compared to the Oklahoma design. When the existing pavement surfaces are thin (1 in.), the overlay thicknesses from the California method are much less for all ranges of traffic.

In using these methods, the designer should recognize that research as well as experience indicate that the maximum allowable value of deflection is a function of the existing surface thickness and the expected traffic. Therefore, it should be recognized that erroneous answers can be obtained with these methods if traffic volumes or existing pavement thicknesses are outside the limits for which the procedure in question was derived.

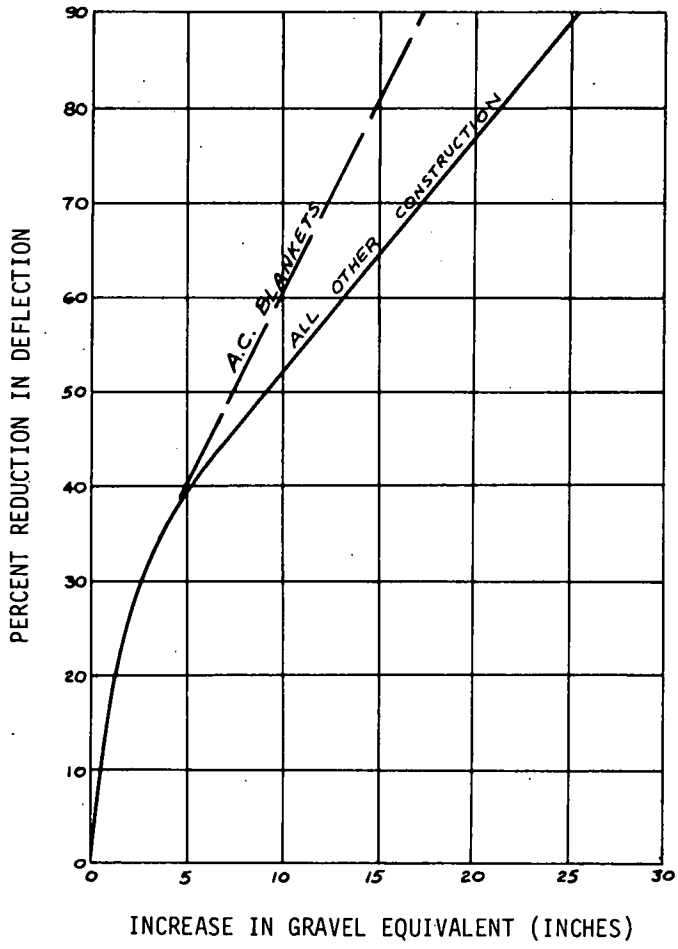


Figure 25. Reduction in deflection resulting from pavement reconstruction—California.

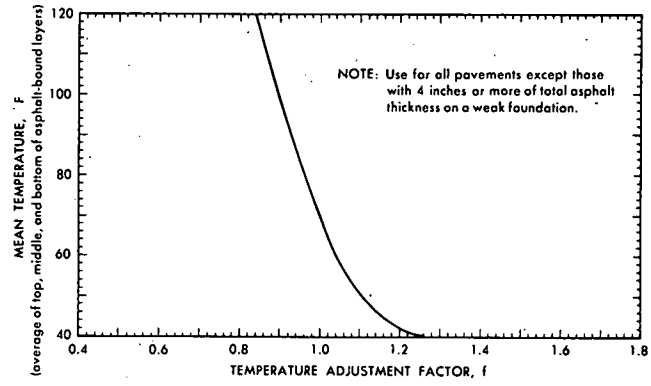


Figure 26. Temperature adjustment factors for Benkelman beam deflections.

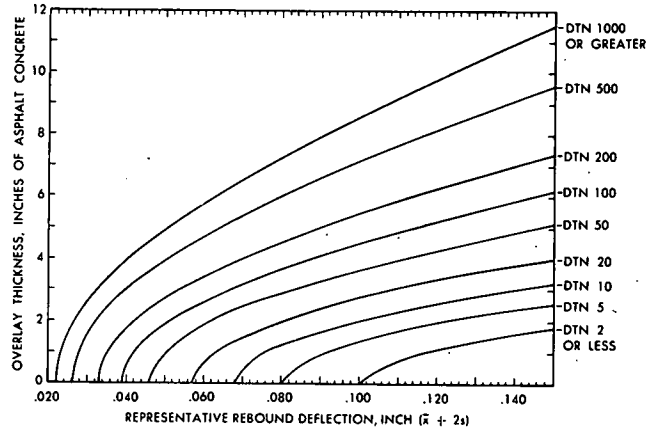


Figure 27. Asphaltic concrete overlay thickness required to reduce deflection from a measured to a design deflection value (rebound test).

ASPHALTIC CONCRETE OVER ASPHALTIC CONCRETE—LAYER ANALYSIS

Procedures for asphaltic concrete over asphaltic concrete are not discussed in detail because they generally involve the use of conventional procedures for flexible pavement design. The general premise of the overlay design methods using existing procedures is based on

$$T_o = T - T_e \qquad (6)$$

in which

T_o = thickness of overlay, inches;
T = thickness of pavement structure required by conventional pavement analysis, inches; and
T_e = effective thickness of the existing pavement structure, inches.

Eq. 6 is shown graphically in Figure 29. The factors for converting existing thickness of the pavement components to effective thickness should take into account the loss of load-carrying capability with repetitions of wheel load. The Asphalt Institute method, for example, has a different set of factors for overlay design than for original pavement

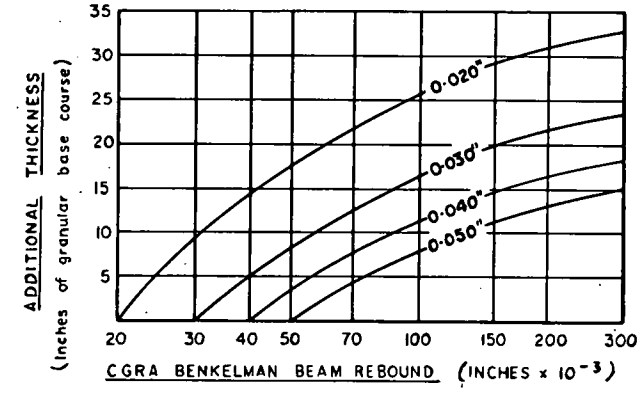


Figure 28. Additional depths of granular base course required to reduce a Benkelman beam rebound on an existing surface to various designated rebound values.

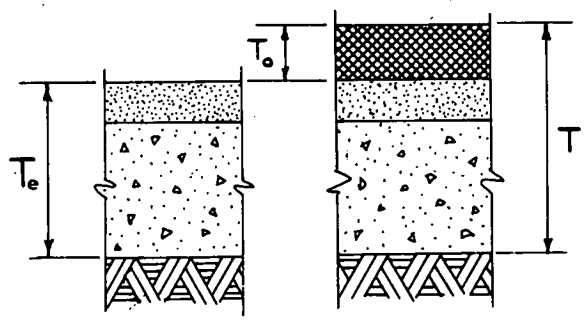


Figure 29. Use of flexible guides in overlay design.

design (Table 4). Regardless of which method is used, an attempt should be made to use a wear-out factor.

ASPHALTIC CONCRETE OVER PORTLAND CEMENT CONCRETE—LAYER ANALYSIS

Figure 17 indicates three possible methods of analysis for asphaltic concrete over portland cement concrete; i.e., AASHO, Corps of Engineers, and The Asphalt Institute. Although the formulas used in the design differ, the basic principles and concepts are similar. The basic premise of these methods is to convert the existing portland cement concrete pavement into an equivalent thickness of asphaltic concrete and use the results in a flexible pavement design procedure as described previously. Therefore, Eq. 6 is also applicable as a basic design equation. Because the procedures are similar, only the Corps of Engineers method is described.

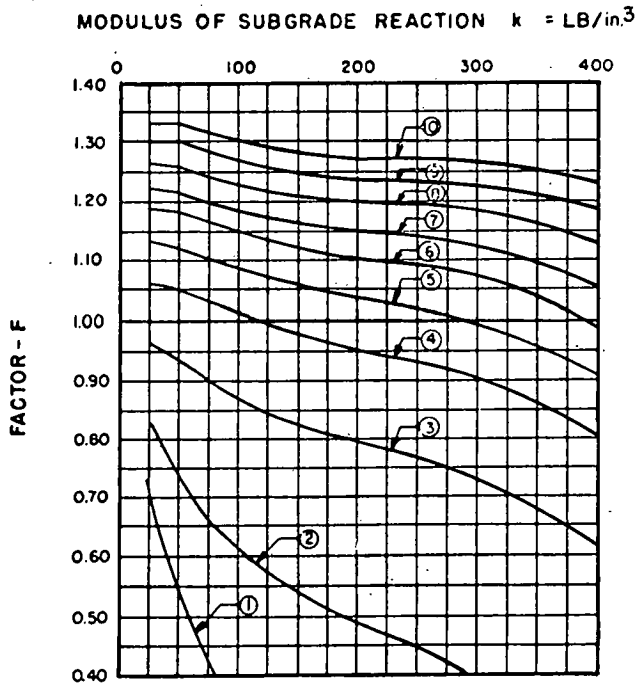


Figure 30. Nongrid overlay design (Corps of Engineers).

The Corps of Engineers method is based on the following empirical formula that assigns a structural equivalency to relate the thickness of asphaltic concrete and portland cement concrete:

$$t = 2.5(Fh_d - Ch) \tag{7}$$

in which

t = thickness of the overlay, inches;
F = a factor related to condition of the existing pavement;
h_d = the exact design thickness determined from the regular portland cement concrete pavement design procedures, inches;
h = the existing pavement thickness, inches; and
C = a factor correcting for the condition of the existing pavement.

A critical step in the design procedure is the selection of the proper F-value which is a function of the in-place subgrade modulus of support and the type of loading expected. The F-value decreases as the support value increases (Fig. 30). The Corps of Engineers' manual gives guidance for selecting the F-value for general conditions of traffic loading, but the exact value for a specific condition is difficult to ascertain.

PORTLAND CEMENT CONCRETE OVER ASPHALTIC CONCRETE—LAYER ANALYSIS

Procedures for portland cement concrete over asphaltic concrete are not presented because they involve the conventional methods used in design of rigid pavements. Therefore, the procedures of the specific highway agency should be referred to. In most cases, a modulus of support value (K) would be estimated or measured on top of the existing pavement. This value is then used in the design chart to estimate thickness of the portland cement concrete overlay. Figure 31 is a correlation between K-value and pavement deflection developed by the California Division of Highways which may be used to estimate a K-value if other values are not available.

PORTLAND CEMENT CONCRETE OVER PORTLAND CEMENT CONCRETE—LAYER ANALYSIS

As indicated in Figure 17, the Portland Cement Association and the Corps of Engineers methods are available for portland cement concrete over portland cement concrete. A study of the design manuals for the two organizations indicates that the procedures are similar; therefore, only the Corps of Engineers procedures are presented herein.

The Corps of Engineers procedure is based on one of two formulas, depending on whether a partially bonded or an unbonded overlay is used. The unbonded overlay is considered for conditions where an asphaltic concrete layer is placed between the two layers of portland cement concrete pavement.

The design equations for each condition are as follows:

TABLE 4

FACTORS FOR CONVERTING THICKNESS OF EXISTING PAVEMENT COMPONENTS TO EFFECTIVE THICKNESS (T_e)

Classification	Description of Material	Conversion Factors[a]
I	Native subgrade in all cases.	0.0
II	(a) Improved subgrade constructed with predominantly granular materials that may contain some silt and clay but have PI of 10 or less. Improved subgrade is any course or courses of improved material between the native subgrade soil and the pavement structure.	0.0 to 0.2
	(b) Lime-modified subgrade constructed from high-plasticity soils having a PI greater than 10. Lime-modified subgrade is a prepared and mechanically compacted unhardened or semihardened intimate mixture of lime, water, and soil below the pavement system.[c]	
III	(a) Granular subbase or base constructed with reasonably well-graded, hard aggregates having some plastic fines and CBR not less than 20. Upper part of range is used if PI is 6 or less; lower lower part of range is used if PI is more than 6.	0.2 to 0.3
	(b) Cement-modified subbases and bases constructed from low plasticity soils that have a PI of 10 or less. Cement-modified subbase is an unhardened or semihardened intimate mixture of pulverized soil, portland cement, and water used as a layer in a pavement system between the subgrade and the base course. Cement-modified base is an unhardened or semihardened intimate mixture of pulverized soil, portland cement, and water and is used as a layer in the pavement system to reinforce and prtect the subgrade or subbase.[c]	
IV	(a) Granular base constructed with a nonplastic granular material complying with established standards for high-quality aggregate base. Upper part of range is used.	0.3 to 0.5
	(b) Asphalt surface mixtures that have large, well-defined crack patterns, spalling along the cracks, and appreciable deformation in the wheel paths showing some evidence of instability.	
	(c) Portland cement concrete pavement that has been broken into small pieces, 2 ft or less in maximum dimension, prior to overlay construction. Upper part of range is used when subbase is present; lower part of range is used when slab is on subgrade.	
	(d) Soil-cement bases that have developed extensive crack patterns as shown by reflected surface cracks, and may exhibit pumping; pavement shows minor evidence of instability. Soil-cement base is a hardened material formed by curing a mechanically compacted intimate mixture of pulverized soil, portland cement, and water and is used as a layer in a pavement system to reinforce and protect the subgrade or subbase.[c]	
V	(a) Asphalt surfaces and underlying asphalt bases[b] that exhibit appreciable cracking and crack patterns, have little or no spalling along the cracks, and remain essentially stable even though exhibiting some wheel path deformation.	0.5 to 0.7
	(b) Appreciably cracked and faulted portland cement concrete pavement that cannot be effectively undersealed. Slab fragments, ranging in size from approximately 1 to 4 sq yd, are well seated on the subgrade by heavy pneumatic rolling.	
	(c) Soil-cement bases that exhibit little cracking, as shown by reflected surface crack patterns, and that are under stable surfaces. (See definition of soil-cement base under IV-d of this table.)	
VI	(a) Asphalt concrete surfaces that exhibit some fine cracking, small intermittent cracking patterns, and slight deformation in the wheel paths though they remain stable.	0.7 to 0.9
	(b) Liquid asphalt mixtures that are stable, generally uncracked, show no bleeding, and exhibit little deformation in the wheel paths.	
	(c) Asphalt treated base, other than asphalt concrete.[b]	
	(d) Portland cement concrete pavement that is stable and undersealed, has some cracking, but contains no pieces smaller than about 1 sq yd.	
VII	(a) Asphalt concrete, including asphalt concrete base, that is generally uncracked and has little deformation in the wheel paths.	0.9 to 1.0
	(b) Portland cement concrete pavement that is stable, undersealed, and generally uncracked.	
	(c) Portland cement concrete base, under asphalt surface, that is stable, nonpumping, and exhibits little reflected surface cracking.	

[a] Values and ranges of conversion factors are multiplying factors for conversion of thickness of existing structural layers to equivalent thickness of asphalt concrete. These conversion factors apply only to pavement evaluation for overlay design. In no case are they applicable to original thickness design.

[b] Asphalt concrete base, asphalt macadam base, plant-mixed base, and asphalt mixed-in-place base.

[c] Terms follow the definitions prepared by the Committee on Soil-Portland Cement Stabilization

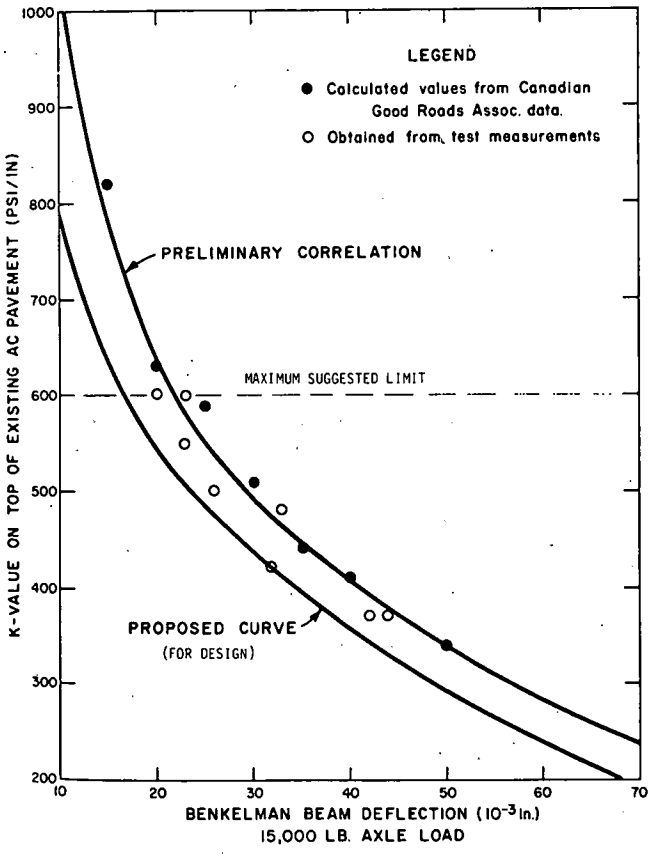


Figure 31. Correlation between K-*value and pavement deflection.*

Partially bonded:

$$h_o = \sqrt[1.4]{h_d^{1.4} - Ch^{1.4}} \quad (8)$$

Unbonded:

$$h_o = \sqrt[2]{h_d^2 - Ch^2} \quad (9)$$

in which

h_o = thickness of overlay slab, inches;
h_d = thickness of new pavement from regular portland cement concrete pavement design analysis, inches;
h = thickness of the existing pavement, inches; and
C = a coefficient depending on the condition of the existing slab—1.0 for slabs in excellent condition, 0.75 for initial breaks, and 0.35 for badly cracked slabs.

The formula for the unbonded case requires thicker overlays than the one for the bonded case, because the former assumes the slabs move independently. The C-term in the question is not a true estimate of the load-carrying capacity, but rather a factor qualitatively associated with previous performance as measured by the amount of cracking present. The choice of the proper C-term for an existing pavement is at best a crude estimate, because no quantitative guidance is given other than general ranges.

SUMMARY

This review of the current overlay design practices indicates that the design methods available account for only wheel-load stresses, with no consideration of volume change stresses. Of the procedures available, only those using deflection evaluate the load-carrying capacity of the in-place pavement structure.

Flexible pavement procedures have been used in the design of asphaltic concrete overlays over existing portland cement concrete pavement, but these are an extension of an empirical equation beyond its limits of derivation. The assignment of the layer equivalency to concrete is an unknown factor that has a pronounced effect on design and should be recognized by the designer.

All of the pavement layer procedures are extremely sensitive to the selection of coefficients for reflecting the load-carrying capacity of the existing pavement. Large errors in thickness can result from improper coefficient selection. Furthermore, the selection is tied to pavement condition rather than load-carrying capacity.

The pavement evaluation procedures using deflection are also subject to error. An acceptable deflection may vary with the thickness combinations for the various pavement structure layers. Therefore, some combinations that present acceptable deflections may not be satisfactory from the stress standpoint.

Emphasis is again placed on the inherent assumption in all the design methods that the proper specifications and construction procedures will be used.

CHAPTER FOUR

ECONOMIC CONSIDERATIONS FOR PAVEMENT REHABILITATION

Even though a decision has been made to proceed with rehabilitation and the most effective application has been chosen, the engineer must still select one of several satisfactory design schemes.* The controlling factor in this selection is economic considerations.

In recommending a given scheme of rehabilitation, the engineer should plan on a long-term basis, rather than consider only the immediate future. The costs of several strategies should be compared over some finite period of time (the cost-analysis period). In addition to a consideration of construction costs, consideration must be given to economic consequences of a given strategy on the user of the facility (i.e., the cost incurred by a user on a facility due to delays experienced during rehabilitation and maintenance).

Figure 32 shows three alternate design strategies that may be used in rehabilitating a given facility. The figure shows a Present Serviceability Index history during a cost-analysis period for three different overlay patterns, ranging from one overlay of sufficient thickness to last the entire cost-analysis period to three different overlay placements. It should be noted that the total thickness of multiple overlays may be greater than a single overlay. In addition to the overlay considerations, seal coats may be used to rejuvenate the asphalt or restore skid resistance. In order to select the optimum design strategy, the designer must consider all the costs occurring during the cost-analysis period.

This chapter provides information and techniques available to the designer to consider total cost of a given rehabilitation scheme over a cost-analysis period. Many of the conceptual aspects herein were taken from a paper by Scrivner et al. (*67*) that was prepared in connection with development of a systems approach to pavement design. Much of this work is used directly herein and was revised to comply with the needs of this synthesis.

A number of cost considerations are associated with the investment in a pavement structure, but, in general, they may be divided into present and future costs. Thus, the investment in a rehabilitation scheme consists of a present cost plus one that is accrued during the cost-analysis period. These cost considerations may be outlined as follows:

1. Present or immediate costs due to an overlay or other methods of treatment.
   a. Overlay construction costs (including maintenance of traffic).
   b. User costs, including traffic delays and safety.
2. Future costs that will be incurred during the cost-analysis period.
   a. Overlay or treatment costs.
      (1) Overlay construction costs (including maintenance of traffic).
      (2) User costs, including traffic delays and safety.
   b. Annual routine maintenance.

Because a portion of the cost will be accrued in the future, the present value of these costs must be established in order to make a valid cost comparison of several rehabilitation schemes. The present value of the total costs may be computed as follows:

$$TC = (OC)_P + (UC)_P + (OC)_F + (UC)_F + MC \quad (10)$$

in which

TC = present value of total cost;
$(OC)_P$ = cost of overlay or treatment that is to be placed at the present time;
$(UC)_P$ = user cost due to traffic delays from overlay construction or other method of treatment;
$(OC)_F$ = sum of present values of the overlay construction or other treatment costs that are to be incurred in the future;
$(UC)_F$ = sums of present values of user costs due to overlay construction or other treatment; and
MC = sum of present values of annual routine maintenance costs.

The techniques used in computing each of the foregoing costs are presented in the following sections. All costs

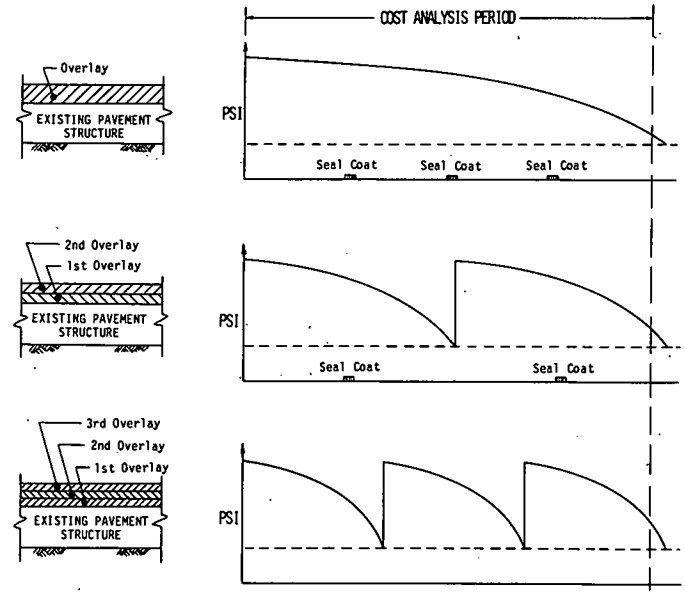


Figure 32. Comparison of alternate design strategies.

* Design scheme or strategy as used herein refers to a selective series of rehabilitation steps that will occur over a given analysis period. This consists of general maintenance operations as well as placing additional thickness on the pavement structure.

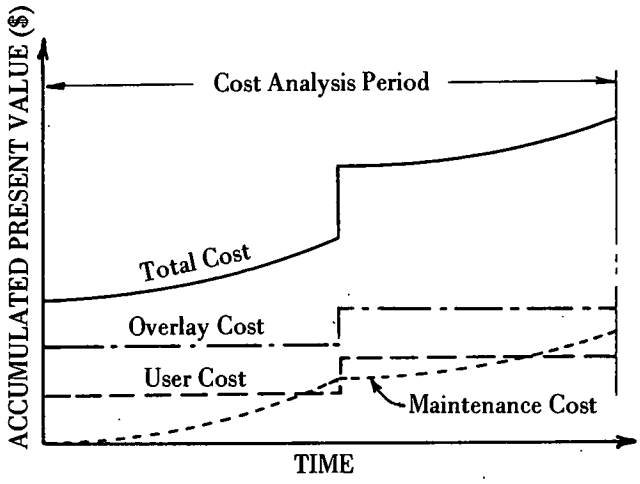


Figure 33. Development of total cost for a design strategy.

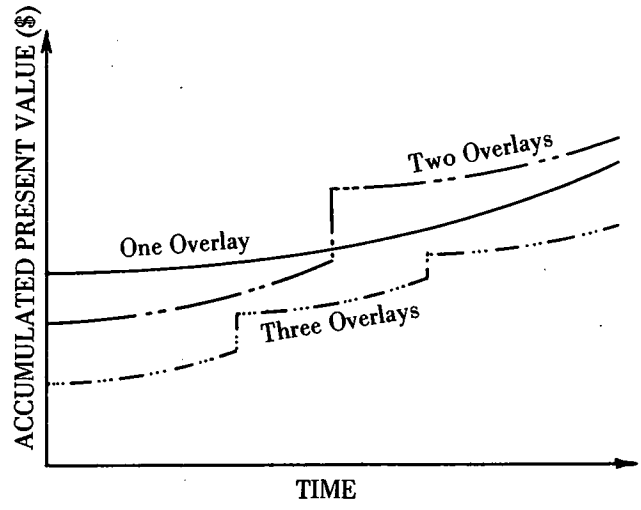


Figure 34. Comparison of total cost for alternate design strategies.

should be calculated on the same basis in order to have a standard for comparison and summation in Eq. 10. The costs herein are given on a square-yard basis, but any other basis (e.g., cost per mile) could be used.

Figures 33 and 34 show the total cost concept over a cost-analysis period. Figure 33 shows the development of the total cost of a given strategy, considering overlay, general maintenance, and user costs. The overlay and user costs occur during the placement period(s), whereas the maintenance costs occur continually, but at an increasing rate until the next overlay occurs. The designer should develop a total cost curve for each strategy and then compare the strategies as shown in Figure 34. In Figure 34, the design strategy with three different overlay periods is the optimum one. The designer is cautioned that each project has different conditions, and that the optimum economic design strategy is a function of the project conditions. Hence, each project must be analyzed to ascertain the optimum design for the conditions of that project.

PRESENT OVERLAY COST

The overlay costs incurred in the immediate future may be estimated using an overlay thickness from one of the methods described in Chapter Three or by the method normally used by the specific highway agency. Data concerning costs of material, etc., may be obtained from the general operations of the highway agency (e.g., laboratory, project cost estimating).

$$(OC)_P = \frac{D_o C_o \gamma_c}{36\gamma_l} \qquad (11)$$

in which

D_o = the design thickness of overlay or other methods of treatment, inches;
C_o = estimated costs of the material per cubic yard as delivered from a plant;
γ_c = estimated density of the material in place; and
γ_l = estimated loose density of the material as delivered.

PRESENT USER COSTS

Delay time, vehicle operation, and accidents are the user costs associated with overlay construction (Fig. 35). Only delay time and vehicle operation costs are considered here because of lack of data concerning accidents. In the future, as appropriate data are developed, the cost of accidents as affected by overlay construction may also be considered.

In developing user costs, the first consideration is the speed profile of the vehicles in the vicinity of the overlay construction. The second step involves developing the time and operating cost associated with different movements described by the speed profiles. Finally, the method of handling traffic in the construction area must be defined. This involves taking into consideration the proportion of vehicles stopped and the average time stopped when traffic congestion arises as a result of the overlay construction.

The techniques for handling each of these procedures are presented in Appendix B, including the formulas and techniques involved in developing these inputs. Caution must be exercised in using the cost data given in Tables B-1 through B-4, because some of the cost information was developed for conditions in the Southwestern U.S. This material can be used as a guideline, but cost data should be developed for the region in which it is used. These input data then may be used in the following to develop the user cost:

$$(UC)_P = \frac{9P(PS)HTr_tK}{5{,}280L_oL_w} \qquad (12)$$

in which

P = proportion of average daily traffic, r_t, that will pass through the overlay area during each hour of the day that overlay takes place, and is assumed to be 0.06 for rural areas and 0.055 for urban areas;
PS = number of overlay passes made;

Figure 35. Delays due to overlay construction.

H = number of hours per day that overlay construction takes place (and probably is between 6 and 12 hr);

T = length of time, in days (or portion of a day), per pass that it takes to make one overlay pass of 2-in. (or less) thickness of length L_o;

r_t = average daily traffic at time t, where t is the time of overlay;

L_o = length of road being overlaid at any one time, in miles;

L_w = width of one traffic lane, in feet;

$K = [PO_1(CO_1 + CO_2 + CO_3) + (1 - PO_1)(CO_3 + CO_4) + PO_2(CO_5) + PN_1(CN_1 + CN_2 + CN_3) + (1 - PN_1)(CN_3 + CN_4) + PN_2(CN_5)]$ where the variables with an O refer to the overlay direction and with an N refer to the non-overlay direction;

PO_1, PN_1 = proportions of traffic that are stopped due to the capacity of the restricted section being less than the demand for it (i.e., the proportions stopped because of congestion);

PO_2, PN_2 = proportions of traffic that are stopped due to the movement of overlay personnel and equipment in the restricted area;

CO_1, CN_1 = excess costs per vehicle per hour of stopping from the approach speed, divided by 1,000; $CO_1 = CN_1$ in all cases;

CO_2, CN_2 = costs per vehicle per hour of idle time for vehicles stopped and are derived by multiplying DO_1 and DN_1 by total idling cost, divided by 1,000;

CO_3, CN_3 = excess costs per vehicle per hour of traveling through the restricted area of length L_s at a reduced speed and are equal to L_s multiplied by the difference between the cost of operating at the reduced speed SO (or SN) and the cost of operating at the approach speed SA, divided by 1,000;

CO_4, CN_4 = excess costs per vehicle per hour of slowing from the approach speed, SA, to through speed SO or SN and returning to the approach speed, divided by 1,000; and

CO_5, CN_5 = excess costs per vehicle per hour of stopping from the through speed SO or SN and divided by 1,000, plus the cost of idling while stopped due to the movement of overlay personnel and equipment which is obtained by multiplying DO_2 and DN_2 by appropriate values and dividing by 1,000.

FUTURE OVERLAY COSTS

The future overlay costs are obtained by computing the cost of all future overlays during the cost-analysis period and establishing the present value of these costs. The present value is equal to the summation of all the future costs unless interest is taken into account, in which case the present value will be less. If an interest rate is assigned, the present value of an overlay placed at 15 years will be lower than a similar one placed at 5 years. Therefore, with this concept, the time schedule for a rehabilitation strategy may have a pronounced influence on costs. The decision to use or not to use an interest rate should be carefully con-

sidered. If it is used, care must also be exercised in the selection of the proper interest rate.

The present value of a future overlay may be computed from

$$(OC)_F = \sum_{j=1}^{j=k} \left(\frac{D_k C_o}{36(1+i)^{t_k}} \right) \left(\frac{\gamma_c}{\gamma_l} \right) \qquad (13)$$

in which

k = the number of additional overlays or treatments constructed during the cost-analysis period;
i = interest rate, if used; and
t_k = time in which k overlay is placed.

All other terms are as previously defined.

Note that if an interest rate of zero is used, this equation is a summation of Eq. 11.

FUTURE USER COSTS

The future user costs may be derived by using Eq. 12 for each overlay or treatment and summing the totals after reducing each treatment to a present value cost. This concept may be expressed as follows:

$$(UC)_F = \sum_{j=1}^{j=k} \frac{(UC)_k}{(1+i)^{t_k}} \qquad (14)$$

where all terms are as previously defined.

If a zero interest rate is used, this formula becomes a simple summation of Eq. 12.

ANNUAL ROUTINE MAINTENANCE COSTS

With any pavement structure, maintenance costs will accrue during each year, with the rate depending on pavement type and maintenance technique. The cost of annual routine maintenance is assumed to increase at a uniform rate starting with a rate, C_1, during the first year after an overlay or other treatment.

$$\text{Annual Cost} = C_1 + (m-1)C_2$$

in which

C_1 = routine maintenance cost/sq yd during the first year after initial or overlay construction;
m = number of years after initial or overlay construction; and
C_2 = incremental increase in routine maintenance cost/sq yd per year.

It is also assumed that all costs occurring within a year are paid at the beginning of that year. After an overlay, the cost of routine maintenance during the following year is again C_1. The present value of all routine maintenance charges during the analysis period can be found from:

$$RM = \sum_{m=1}^{M_k} \frac{C_1 + (m-1)C_2}{(1+i)^{m-1}} \qquad (15)$$

in which

RM = present value of the total cost/sq yd of all routine maintenance during the analysis period;
i = interest rate;
M_k = number of years in the kth performance period, $= t_k - t_{k-1}$, where t_k and t_{k-1} are rounded to the nearest integer before M_k is calculated; and
K = number of performance periods within the cost analysis period.

If a zero interest rate is used, this equation becomes:

$$RM = mC_1 + \frac{m(m-1)}{2} C_2 \qquad (16)$$

EXAMPLE PROBLEMS

The information presented in the previous sections represents numerous concepts that will require study by the designer before he can fully use them. Therefore, the solutions of several variations of typical field problems are presented to illustrate: (1) the application of the concepts, and (2) that the optimum design strategy may change with project conditions.

A rural four-lane freeway with the project parameters as shown in Figure 36 was selected to illustrate the methodology. Two different problems are considered, one with an initial ADT of 40,000 vpd and one with 20,000 vpd, to illustrate that the user costs may change the optimum design strategy. For the heavier traffic conditions, it is assumed that the overlay operations will cause considerable traffic delay, with a reduction of speed from 60 to 10 mph. For the lower density (20,000 vpd) the congestion is less; hence, the speed reduction is only 60 to 45 mph through the overlay zone.

Next it is assumed that two different rehabilitation strategies are developed based on a 20-year analysis. Strategy A calls for an initial 4-in. overlay that will last the entire analysis period. Strategy B requires an initial overlay of 2½ in. followed by a second overlay of 2½ in. after 10 years. For this problem, a value of 5 percent interest is assigned as possible alternate uses of the funds. Obviously, if only costs of the overlay materials were considered, strategy A would be the most economical due to a smaller thickness, but with consideration of interest and user costs the optimum design strategy will vary with project conditions.

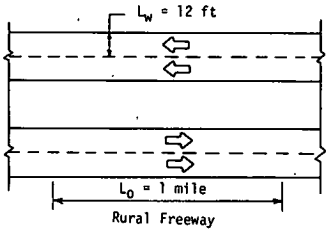
For strategy A (4-in. overlay), only the present overlay and user costs are computed, using Eqs. 11 and 12, respectively, because no additional overlays are anticipated during the analysis period. These costs are derived using Tables B-1, B-3, and B-4 (Appendix B). Because the facility is a divided highway, it is assumed that no delay will be incurred in the other roadway; hence, Eq. 12 is reduced considerably. The routine maintenance cost over a 20-year period is computed using Eq. 15, assuming an initial cost of $50 per mile per year, with an annual increase of $10 per mile. At the end of 20 years the annual maintenance cost is $240 per mile. The total cost can then be derived using Eq. 10. These computations are summarized in Table 5.

For strategy B (two overlays), the present overlay and user costs are computed as just described. The future overlay and user costs for the 2½-in. overlay after 10 years are

computed using Eqs. 13 and 14, respectively. In this case, an interest rate of 5 percent is applied to derive the present value of the monies. The routine maintenance costs for this case will be less than for strategy A because the annual cost increases with time from the last pavement overlay. Therefore, when the second overlay is placed at 10 years, the annual maintenance cost will be reduced to $50. These cost computations are also summarized in Table 5.

Considering the heavy traffic conditions (problem 1), the total costs are $4.92 and $6.31 per square yard for strategies A and B, respectively (see Table 5). Here, the design strategy A is the optimum one because the heavier traffic volume and delay costs due to traffic congestion result in a higher user cost. Because two different overlays are required, the user cost for strategy B is approximately 57 percent of the total cost, thus making it an undesirable alternative. For the lower traffic condition (problem 2), the total costs are $3.30 per square yard and $3.20 per square yard for strategies A and B, respectively. Thus, the optimum economical design is strategy B in this case because user costs are minimum.

These problems emphasize the importance of considering the total costs in selecting a design strategy. It is also evident that the optimum design strategy depends on traffic and on site conditions and thus will vary considerably from project to project.



General Project Parameters:

Cost of Asphalt Concrete - C_0 = $18/cy
Compacted A.C. density - γ_C = 140 #/ft^3
Loose A.C. density - γ_L = 100 #/ft^3
No cars are stopped in non-overlay direction - PN_1 = 0
Cars stopped due to const. traffic - PO_2 = 2%
Routine maint. cost first year - C_1 = $50/mile/year
Incremental increase in routine maint. cost - C_2 = $10/mile/year
Interest rate - i = 5%

Design Strategies:

(A) - 4-inch A.C. overlay to last 20 years
(B) - 2 ½ inch A.C. overlay initially and 2 ½ inch overlay at the end of 10 years

Variable Project Parameters:

	Prob. 1	Prob. 2
ADT initially* - r_0	40,000 vpd	20,000 vpd
ADT at end of analysis period - r_{20}	60,000 vpd	20,000 vpd
Approach traffic speed - V_1	60 mph	60 mph
Overlay traffic speed - V_2	10 mph	45 mph

* Two-way traffic - 50% split between roadways

Figure 36. Values of assumed design parameters for example problems.

DERIVATION OF COSTS

Problem #1-A

$$(OC)_P = \frac{4 \times 18 \times 140}{36 \times 100} = \$2.80/\text{sq yd}$$

$(OC)_F = 0$ No future overlay for this case.

$$(UC)_P = \frac{9 \times 0.06 \times 4 \times 12 \times 0.5 \times 20{,}000 \times 0.4603}{5{,}280 \times 1 \times 12} = \$1.88$$

$$K = [0 + (1 - 0)(0.28351 + 0.106797) + 0.02(3.49976)] = 0.4603$$

Note: CO_3, CO_4, and CO_5 from Appendix B, Tables B-3, B-1 and B-4, respectively. No delay to non-overlay direction; therefore, all N terms are zero.

$(UC)_F = 0$ No future overlay.

$RM = \$0.24/\text{sq yd}$ From Table 6

Problem #1-B

$$(OC)_P = \frac{2\frac{1}{2} \times 18 \times 140}{36 \times 100} = \$1.75/\text{sq yd}$$

$$(OC)_F = \frac{2\frac{1}{2} \times 18}{36(1 + 0.05)^{10}} \times \frac{140}{100} = \$1.07/\text{sq yd}$$

$(UC)_P$ = Same as #1-A = $1.88/sq yd

$$(UC)_F = \frac{(UC)_K}{(1 + 0.05)^{10}} = \frac{1.88 \times \frac{25{,}000}{20{,}000}}{(1 + 0.05)^{10}} = \$1.44/\text{sq yd}$$

$(UC)_K$ = Present costs $(UC)_P$ increased in proportion to ADT

$RM = \$0.17/\text{sq yd}$ See Table 6

Problem #2-A

$(OC)_P$ = Same as Problem #1-A = $2.80/sq yd

TABLE 5

SUMMARY OF COMPUTED COSTS FOR VARIOUS PROBLEMS AND DESIGN STRATEGIES

	PRESENT VALUE ($/SQ YD)			
	PROBLEM NO. 1 (HEAVY TRAFFIC)		PROBLEM NO. 2 (LIGHT TRAFFIC)	
	DESIGN STRATEGY		DESIGN STRATEGY	
COST	A	B	A	B
$(OC)_P$	2.80	1.75	2.80	1.75
$(OC)_F$	—	1.07	—	1.07
$(UC)_P$	1.88	1.88	0.26	0.13[a]
$(UC)_F$	—	1.44	—	0.08[a]
RM	0.24	0.17	0.24	0.17
Total	4.92	6.31	3.30	3.20

[a] Assumed overlay time less because thickness is less.

A = One overlay, 4 in. thickness.
B = Two overlays, 2½-in. thickness.

$(OC)_P$ = Present overlay cost.
$(OC)_F$ = Future overlay cost.
$(UC)_P$ = Present user cost.
$(UC)_F$ = Future user cost.
RM = Routine maintenance cost.

TABLE 6

COSTS OF ROUTINE MAINTENANCE

m	$(1+i)^{m-1}$	Single Overlay		Two Overlays	
		$C_1+(m-1)C_2$	Col 3 / Col 2	$C_1+(m-1)C_2$	Col 5 / Col 2
1	1	50	50.00	50	50.00
2	1.05	60	57.14	60	57.14
3	1.1025	70	63.52	70	63.52
4	1.157	80	69.14	80	69.14
5	1.215	90	74.07	90	74.07
6	1.276	100	78.37	100	78.37
7	1.34	110	82.08	110	82.08
8	1.406	120	85.34	120	85.34
9	1.478	130	87.96	130	87.96
10	1.550	140	90.32	140	90.32
11	1.630	150	92.02	50	30.67
12	1.710	160	93.56	60	35.08
13	1.795	170	94.71	70	39.00
14	1.885	180	95.49	80	42.44
15	1.980	190	95.96	90	45.45
16	2.08	200	96.15	100	48.07
17	2.18	210	96.33	110	50.46
18	2.29	220	96.07	120	52.40
19	2.40	230	95.33	130	54.16
20	2.53	240	94.86	140	55.33
			$\sum$ = \$1688.12/mi		$\sum$ = \$1191.00/mi
			RM = \$0.24/sy		RM = \$0.17/sy

$(OC)_F = 0$ No future overlay for this case.

$$(UC)_P = \frac{9 \times 0.06 \times 4 \times 12 \times 0.5 \times 10{,}000 \times 0.1253}{5{,}280 \times 1 \times 12}$$

$$= \$0.26/\text{sq yd}$$

$$K = [0 + (1 - 0)(0.0109 + 0.0444) + 0.2(3.49976)]$$
$$= 0.1253$$

Note: CO_3, CO_4, and CO_5 from Appendix B.

$(UC)_F = 0$ No future overlay.

RM = Same as Problem #1-A = \$0.24 sq yd

Problem #2-B

$(OC)_P$ = Same as Problem #1-B = \$1.75/sq yd

$(OC)_F$ = Same as Problem #1-B = \$1.07/sq yd

$(UC)_P$ = ½ of Problem #2-A because construction time assumed to be ½ = \$0.13/sq yd

$$(UC)_F = \frac{\$0.13}{(1 + 0.05)^{10}} \$0.08/\text{sq yd}$$

RM = Same as Problem #1-B = \$0.17 sq yd

CHAPTER FIVE

CURRENT AND FUTURE RESEARCH

CURRENT RESEARCH

Several agencies are currently engaged in research in the area of pavement rehabilitation. Many of these are identified in Table 7.

FUTURE RESEARCH

Although much is presently known concerning materials and techniques for pavement rehabilitation, this synthesis revealed certain areas where additional research may prove beneficial.

Pavement Evaluation

There is a need for new equipment and methods for evaluating existing pavements. This would include nondestructive testing to determine material properties, fatigue damage, thickness of in-place layers, and location of voids under pavements. Also required is a method for estimating the remaining life of a pavement.

Because the Present Serviceability Index and other similar quantitative rating systems give definitive levels for judging the performance of a pavement, there is an immediate need for equipment to rapidly measure the parameters required in the rating procedures. Beyond this, however, there is a need for a more exact definition of pavement performance and for better ways of measurement.

In addition, research is needed to establish the reliability and suitability of present overlay design procedures for various environmental conditions.

Materials

Research is needed to produce materials with properties that will overcome various distress mechanisms. The following are readily apparent, but these are by no means the only ones.

1. Patching materials for portland cement concrete that will allow rapid application with a minimum of equipment in areas of heavy and high-speed traffic.
2. Materials for use in overlays on portland cement concrete pavements that will minimize reflection cracking over joints and cracks.

Other Research Needs

There is a need for criteria to determine how often and when maintenance should be performed on thermal cracks in bituminous concrete. Techniques are needed for treating rigid pavements to prevent blow-ups.

TABLE 7

SUMMARY OF KNOWN RESEARCH ACTIVITIES RELATED TO PAVEMENT REHABILITATION [a]

RESEARCH PROJECT TITLE	RESEARCH AGENCY	HRIP NO. [b]
Continuously Reinforced Concrete Pavement	Illinois Division of Highways, Bureau of Res. & Dev.	25 007725
AASHO Road Test (Rehabilitated Study)	Illinois Division of Highways, Bureau of Res. & Dev.	25 007729
Strengthening of Flexible Pavement by Application of Overlay	Road Research Laboratory (UK)	25 060262
Test Road Sections in Mexico	National University of Mexico, Eng. Inst., Road Res. Dept.	25 060900
Design of the Strengthening of Old Pavements	State Road Laboratory (Norway)	25 062407
Utilizing Deflection Measurements to Upgrade Pavement Structures	Texas Highway Department	25 082413
Application of Deflections and Deflection Basins to Construction Control, Pavement Design and Maintenance Requirements	Utah State Department of Highways, Materials & Tests Div.	25 202164
Field Study of the Effect of Overlays on Flexible Pavement Deflections	Minnesota Department of Highways	25 202638
Pavement Deflection as Pavement Overlay Criteria	Washington State University	25 208213
Utilization of Dynaflect Deflections in Overlay Design	Virginia Highway Research Council	25 214256
South Dakota Flexible Pavement Study	South Dakota Department of Highways	26 003389
Statewide Flexible Pavement Performance and Deflection Study	California Division of Highways, Materials & Research Dept.	26 004744
Pavement Deflection Study	Calgary, Alberta (Canada), Eng. Dept.	26 050176
Pavement Evaluation and Maintenance Programming	Montreal, Quebec (Canada), Control & Res Laboratory	26 050345
Concrete Overlay of Concrete Pavement	Birmingham University (UK), Dept. of Transportation & Environmental Planning	26 064050
Evaluating Pavement Serviceability	Utah State Dept. of Highways	26 082366
Bituminous Resurfacings on Flexible Pavements	New York State Department of Transportation, Bureau of Physical Research	26 089267
Pavement Resurfacing	Portland Cement Association	32 069256
Overlays and Tack Coats	Nat. Inst. Road Research (S. Africa)	40 065361
PCC Pavement Overlays	California Division of Highways, Materials & Research Dept.	40 088943
Pavement Patching Techniques and Materials	New Jersey Dept. of Transportation	40 203755
Criteria for Need of Seal Coats for Bituminous Pavements	Minnesota University	40 207122
Lime-Cement Mud Jacking	Louisiana Department of Highways	40 207637
Use of Wire Reinforcement in Bituminous Overlays	University of Tennessee, Dept. of Civil Engineering	40 207591
Bituminous Resurfacings on Rigid Pavements	New York Department of Transportation	
Pavement Deflection as Pavement Overlay Criteria	Washington Department of Highways	

[a] As of December 1971. [b] Acquisition number assigned by Highway Research Information Service of the Highway Research Board; HRIP = publication entitled *Highway Research in Progress* (current issue).

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APPENDIX A

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APPENDIX B

TRAFFIC COST OF OVERLAY *

This appendix describes the procedure for calculating users' (or motorists') increase in costs due to overlaying operations. These costs include those associated with delay (time), vehicle operation, and accidents. Only delay and vehicle operating costs are considered, because little information exists on how accident rates are affected by overlaying.

SPEED PROFILES

It is assumed that all vehicles approach the overlay area at the same speed, called the "approach speed." It is further assumed that there is a "restricted area," the length of which is L_s (in miles), through which vehicles travel at a reduced speed. For some methods of handling traffic this reduced speed may be the same for vehicles traveling in both directions, but for other methods it may be different for each direction; it is assumed to always be the same for all vehicles going in the same direction. Generally, the vehicles traveling in the "overlay direction" will have their speed reduced as much as, or more than will those traveling in the "non-overlay direction."

The length, L_s, of the restricted area generally will be longer than the length, L_o, of the actual overlay operation. The amount by which L_s is longer than L_o will be determined by the road geometrics and the method of handling traffic. This is discussed more fully in the following section on methods of handling traffic.

It is assumed that vehicles approach the restricted area at an "approach speed," SA. If there were no overlay taking place, the vehicles would travel through the area which is restricted during overlay at the "approach speed." During overlay, however, most vehicles travel through the restricted area at a reduced speed, called the "through speed, overlay direction," denoted by SO, or the "through speed, non-overlay direction," denoted by SN. It is assumed that vehicles maintain these "through speeds" all the way through the restricted area.

A proportion (called PO_1 in the overlay direction and PN_1 in the non-overlay direction) of all vehicles will be stopped as they approach the restricted area. It is assumed that these vehicles stop and then accelerate back to the through speed, which is reached at the moment they enter the restricted area of length L_s; the vehicles then travel at the reduced speed (SO or SN) through the restricted area for a distance of L_s, and as soon as they leave the restricted area they return to a speed which is the same as their approach speed.

Figure B-1 shows the speed profile for a vehicle that makes a stop. The letters along the horizontal axis denote points where speeds are changed. The vehicle approaches the restricted area at a speed of SA, begins decelerating at point A, and is stopped by the time it reaches point B. It remains stopped for a time (DO_1 in the overlay direction and DN_1 in the non-overlay direction) at point B, then accelerates back to the through speed SO or SN which is reached at point C, the beginning of the restricted area of length L_s. The vehicle then travels from point C to point D at the through speed, and at point D begins accelerating back to the approach speed, which is reached at point E.

Vehicles that do not stop are slowed down when they pass through the restricted area, and it is assumed that their deceleration is such that they reach the through speed (SO or SN) at the moment they enter the restricted area. The proportion of vehicles that do not stop equals $1 - PO_1$ for the overlay direction and $1 - PN_1$ for the non-overlay direction.

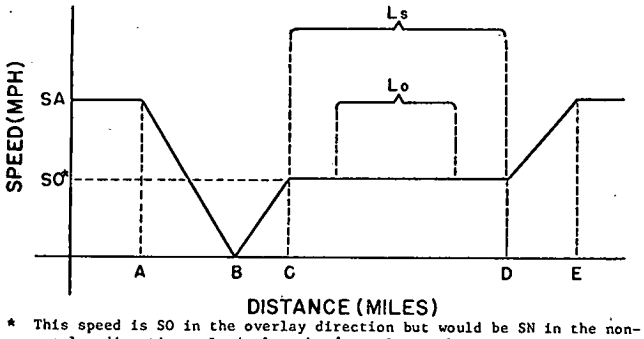
Figure B-2 shows the speed profile for a vehicle that is not stopped but is slowed by the overlay operation. The letters along the horizontal axis denote points where speeds are changed. The vehicle approaches the restricted area at a speed of SA, begins decelerating at point A, and reaches the through speed SO or SN at point B, the beginning of the restricted area of length L_s. It continues at the through speed from point B to point C, which is at the end of the restricted area, then at point C begins accelerating back to the approach speed, which is reached at point D.

Because of overlay operations, vehicles will travel through the overlay area with speed profiles as shown in Figures B-1 and B-2. In the absence of overlay operations they would have traveled at the approach speed through the overlay area. The excess traffic costs due to overlay include the excess time and operating costs due to reducing from the approach speed to a stop (from A to B in Fig. B-1) and returning back to that speed (from B to C and from D to E in Fig. B-1), the excess time and operating (idling) costs due to being stopped (at point B in Fig. B-1), the excess time and operating costs due to reducing from the approach speed to the through speed (from A to B in Fig. B-2) and returning to the approach speed (from C to D in Fig. B-2), and the excess time and operating costs due to traveling a distance L_s (from C to D in Fig. B-1 and from B to C in Fig. B-2) at a reduced speed (SO or SN) instead of traveling at the approach speed (SA). The excess costs of the aforementioned types are discussed in the next section.

TIME AND OPERATING COSTS

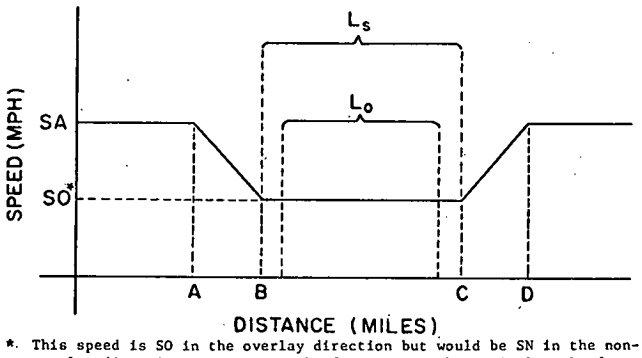
The excess user costs due to an overlay include the excess cost of stopping and slowing down, the cost of delay while stopped, and the excess cost of traveling at a reduced speed through the restricted area. The information needed for

* Source: SCRIVNER, F. H., ET AL., "A Systems Approach to the Flexible Pavement Design Problem." *Research Report 32-11*, Texas Transportation Institute (Oct. 1968).



* This speed is SO in the overlay direction but would be SN in the non-overlay direction. L_o is length of overlay work; L_s is length of restricted area.

Figure B-1. Speed profile for vehicles that are stopped during overlay.



*. This speed is SO in the overlay direction but would be SN in the non-overlay direction. L_o is length of overlay work; L_s is length of restricted area.

Figure B-2. Speed profile for vehicles that are not stopped but are slowed during overlay.

calculating these costs is given in Tables B-1 through B-4. The difference between the urban and rural costs is the vehicle distributions used to derive the costs. The operating costs for different types of vehicles were taken from Winfrey (*1*); that source was used for the excess time of making speed changes. The values of time used in calculations were based on information in studies by the Stanford Research Institute (*2*), Lisco (*3*), and Adkins (*4*). The proportion of vehicles of different types for urban and rural areas in Texas is taken from proportions for 1966 given in a study by the Texas Highway Department (*5*). The reason that the costs are higher for rural areas than for urban areas is that there is a higher proportion of trucks in rural areas and their costs are higher than those for passenger cars.

Tables B-1 and B-2 give the excess time and operating costs for stopping (in the first column) and for slowing down (in the other columns). Table B-3 gives the time and operating costs for operating at a uniform (constant) speed. Table B-4 gives the time and operating costs of delay (or idling).

TABLE B-1

EXCESS OPERATING AND TIME COST OF SPEED CHANGE CYCLES—EXCESS COST ABOVE CONTINUING AT INITIAL SPEED, FOR RURAL ROADS

INITIAL SPEED (MPH)	COST ($) PER 1,000 CYCLES, BY SPEED REDUCED TO AND RETURNED FROM (MPH)					
	0	10	20	30	40	50
10	8.473					
20	18.200	9.413				
30	31.550	21.491	11.354			
40	50.360	39.609	28.422	15.795		
50	77.932	66.233	53.917	39.941	22.612	
60	120.546	106.979	92.482	76.022	56.405	32.485

TABLE B-2

EXCESS OPERATING AND TIME COST OF SPEED CHANGE CYCLES—EXCESS COST ABOVE CONTINUING AT INITIAL SPEED, FOR URBAN ROADS

INITIAL SPEED (MPH)	COST ($) PER 1,000 CYCLES, BY SPEED REDUCED TO AND RETURNED FROM (MPH)					
	0	10	20	30	40	50
10	5.869					
20	11.769	5.602				
30	19.500	12.857	6.501			
40	30.030	22.933	15.976	8.607		
50	45.002	37.338	29.610	21.448	11.856	
60	67.868	58.992	49.114	40.242	29.360	16.432

METHODS OF HANDLING TRAFFIC

There are several methods of handling traffic during an overlay operation. The method used depends mainly on highway geometrics, especially the number of lanes, the type of median (if any), and the presence or absence of pavement shoulders, frontage roads, or other alternate routes. Following are the five methods of handling traffic which are most commonly used, and the way of calculating average delay and proportion of vehicles stopped for each method.

The Five Methods

The first two methods of handling traffic are two-lane roads (with or without shoulders) and the other three methods are for roads with four or more lanes. Figures B-3 through B-7 show in a general way the five situations. In each figure, L_o and L_s are shown. L_o is the amount of road that is overlayed at any one time—not the total overlay job length. L_s is the distance over which traffic is slowed down by the overlay operation at any one time. In the situations shown in Figures B-3, B-4, and B-5, L_s is only slightly

TABLE B-3

OPERATING AND TIME COST PER 1,000 VEHICLE-MILES AT UNIFORM SPEEDS IN TEXAS

UNIFORM SPEED (MPH)	COST ($) PER 1,000 VEHICLE-MILES	
	RURAL ROADS	URBAN ROADS
5	750.20	692.06
10	393.47	362.43
15	274.15	252.21
20	214.53	197.06
25	179.13	164.16
30	156.05	142.57
35	140.18	127.58
40	129.03	116.84
45	121.06	108.98
50	115.51	103.24
55	111.94	99.18
60	110.16	96.73

TABLE B-4

OPERATING AND TIME COST OF DELAY (OR IDLING) PER 1,000 VEHICLE-HOURS

TYPE OF DELAY COST	COST ($) PER 1,000 VEHICLE-HOURS	
	RURAL ROADS	URBAN ROADS
Operating	132.22	122.89
Time	3,367.54	3,140.22
Total	3,499.76	3,263.11

longer than L_o by, say, 0.1 mile, but in the situations shown in Figures B-6 and B-7, L_s may be considerably longer than L_o. L_s and L_o are job variables provided by the designer and should be "expected" average values. If L_o and L_s are not constant, the total overlay should be divided into several L_o and L_s sections and the traffic costs should be calculated separately for each. In the figures the "overlay direction" is always to the left (or westward).

Method I (Fig. B-3)

For two-lane roads with shoulders, one lane of traffic can be diverted onto a shoulder. Traffic going "west" in the "overlay direction" can be diverted onto the shoulder (Fig. B-3) and such traffic will generally be slowed down. Traffic in the non-overlay direction proceeds as usual, but also must slow down, although probably not as much as the traffic going in the overlay direction. Another version of this method is to divert the traffic in the non-overlay direction onto the shoulder and divert the traffic going in the overlay direction into the eastward lane. In addition to the delay due to traveling at a reduced speed, traffic may be further delayed by having to stop due to movement of overlay personnel and equipment in the overlay area and by the inability to overtake other traffic in the overlay area.

Method II (Fig. B-4)

For two-lane roads without shoulders it is sometimes necessary to post flagmen at each end of the overlay operation and to stop traffic in one direction while traffic from the other direction proceeds through the overlay area. The flagmen determine from which direction traffic is let through the overlay area at any one time. The vehicle arriving first usually has priority. If an additional vehicle arrives while other vehicles going in the same direction are proceeding through the overlay area, this additional vehicle usually may also proceed through the area, except that it will be stopped when the vehicles in the queue from the other direction are of a number or have been waiting a time that justifies priority for them. Traffic from each direction travels through the overlay area at a reduced speed. Some vehicles from each direction are stopped to give way to vehicles from the opposite direction, or because of the movement of overlay personnel and equipment in the overlay area, or because vehicles do not have the ability to overtake other vehicles in the overlay area. Generally, the proportion of traffic stopped to give way to vehicles from the opposite direction will be higher the longer is L_s and the larger is the traffic volume.

Method III (Fig. B-5)

For roads with two or more lanes in each direction and with a nontraversable median, it is assumed that traffic in only one direction will be affected by the overlay operation. It is also assumed that at least one lane in the overlay direction remains open for traffic. For low traffic volumes the effects on traffic in the overlay direction will be that of reduced speed through the area, stops due to movement of overlay personnel and equipment, and inability to overtake other vehicles as easily. For higher volumes for which the flow of traffic is greater than the capacity of the restricted roadway, a queue will result upstream of the overlay operation and will lead to vehicles being stopped due to congestion.

Method IV (Fig. B-6)

For roads with two or more lanes in each direction and with no medians or with medians that can be crossed at any point or that have median openings, it may sometimes be desirable to block all lanes in the overlay direction and divert the overlay-direction traffic to non-overlay-direction lanes. Figure B-6 shows the situation for a road with median openings; the difference between this situation and those with either no median, or situations for which it is intended that traffic will cross the median immediately in front and behind the overlay job, is that L_s in these latter two cases would bear approximately the same relation to L_o as is the relation in Figures B-3, B-4, and B-5, instead of being the distance between median openings at each end of the overlay job, as is shown in Figure B-6. For those situations both lanes of traffic will be affected, and the method of calculating average delay and proportion of vehicles stopped is the same as that used for Method III, but in Method IV traffic going in both directions is affected.

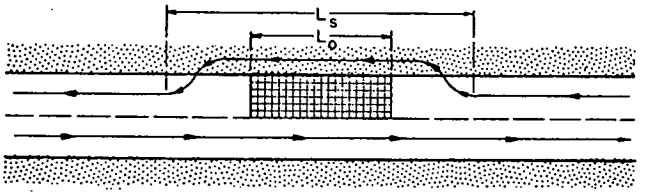


Figure B-3. Method I. Traffic routed to shoulder.

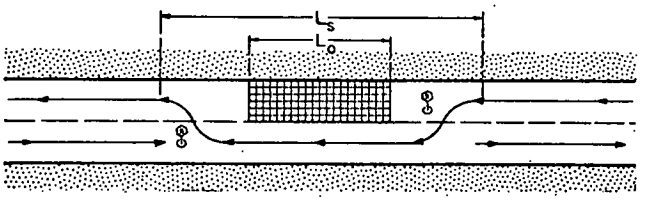


Figure B-4. Method II. Alternating traffic in one lane.

Method V (Fig. B-7)

In some situations traffic in the overlay direction is diverted to an alternate route such as a frontage road or some other parallel road or street.

Average Delay and Proportion of Vehicles Stopped

The delay to traffic due to overlay is of four basic types which are related to vehicles:

1. Traveling at a reduced "uniform" speed in the restricted area.
2. Not having the ability to overtake and pass other vehicles traveling in the same direction.
3. Having to stop because of the movement of overlay personnel and equipment in the overlay area.
4. Having to stop because of congestion when the traffic demand exceeds the capacity of the restricted area.

The delay per vehicle due to traveling at a reduced speed equals the travel time at the reduced speed through the restricted area minus the travel time vehicles would have had through the restricted area had it not been restricted.

The delay due to vehicles not having the ability to overtake and pass other vehicles traveling in the same direction because of the overlay operation may result both in the restricted area and outside the restricted area. This delay, when it occurs in the restricted area, is included in the delay discussed in the preceding paragraph, if accurate estimates of speeds through the restricted area are used. The delay due to the inability to pass outside the restricted area because of the overlay is a more complicated calculation and is probably largest in the cases where the capacity of the restricted area is smaller than the demand (or input) for use of the area. In such cases where demand exceeds capacity, there will be congestion and queuing of vehicles. These queues must disperse after leaving the restricted area and, before such dispersion occurs, there probably will be some inability to pass outside the restricted area. Such effects probably will be small in all cases except Method II with considerable queuing. If the length of the restricted area is fairly long and/or the traffic volumes are fairly large, in Method II fairly long queues of vehicles will arise. Such queues arise in one direction while vehicles are traveling through the restricted area from the other direction; then, when this queue is allowed to proceed through the restricted area it will emerge as a moving queue, sometimes of considerable length, and extra delay will result outside the restricted area until the queue disperses. The rate of dispersion of this moving queue will depend mainly on the amount of traffic from the opposite direction, the road geometrics, and the type of vehicles in the queue. This moving queue will also impede passing by traffic from the opposite direction; however, there will be a considerable length of road in front of this queue in which there will

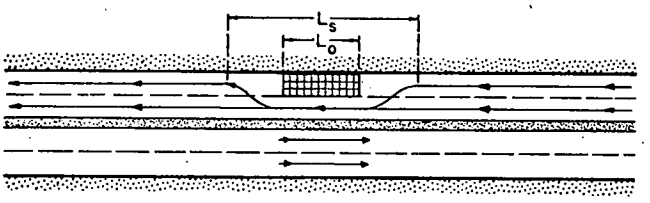


Figure B-5. Method III. Two lanes merge, non-overlay direction not affected.

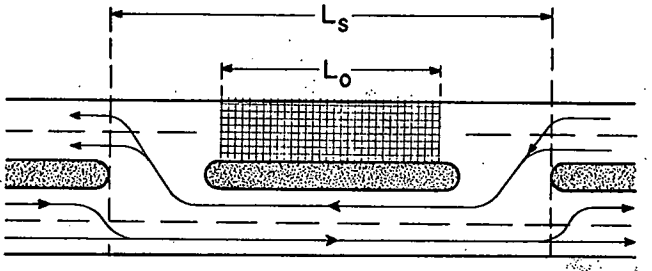


Figure B-6. Method IV. Overlay direction traffic routed to non-overlay lanes.

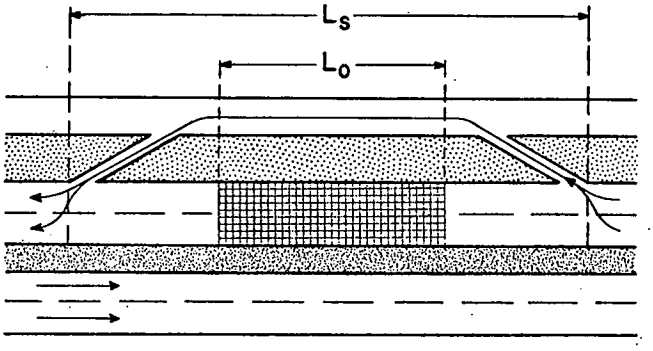


Figure B-7. Method V. Overlay direction traffic routed to frontage road or other parallel route.

be no (or very few) vehicles, which will somewhat offset the effects of the moving queue on traffic in the opposite direction. Because the type of delay discussed in this paragraph probably is small in most cases and because it is difficult to calculate, it is ignored (i.e., assumed to be zero).

The third type of delay (that resulting from vehicles having to stop because of the movement of overlay personnel and equipment in the overlay area) is calculated by multiplying the number of vehicles stopped by the excess delay of stopping. The designer must estimate (1) PO_2 and PN_2, the proportion of vehicles stopped due to the movement of overlay personnel and equipment, and (2) DO_2 and DN_2, the average time that each vehicle stopped (because of such movement) remains stopped. It is expected that the overlay operation will be conducted in a way such that the number of vehicles stopped due to the movement of overlay personnel and equipment will be small. Therefore, in the absence of information on the number of such stops, the designer probably should estimate that number to be near or equal to zero.

The last type of delay (denoted by DO_1 in the overlay direction and by DN_1 in the non-overlay direction) results when a proportion of vehicles (denoted by PO_1 in the overlay direction and PN_1 in the non-overlay direction) have to stop because of congestion that results when traffic demand or input per hour (Q) exceeds the hourly capacity or output (θ) of the restricted area.

When Method I of handling traffic is used, this type of delay should be almost nonexistent and is assumed to be zero; i.e., $DO_1 = DN_1 = 0$ and $PO_1 = PN_1 = 0$ for Method I.

When Method II of handling traffic is used, it is assumed that the hourly traffic is evenly divided between directions. It is also assumed that vehicle arrivals from each direction are Poisson. The proportion of vehicles stopped due to congestion from each direction (PO_1 or PN_1) is estimated by

$$PO_1(=PN_1) = \frac{(1 - e^{-aQ})^2}{2} \qquad \text{(B-1)}$$

TABLE B-5

ONE-DIRECTIONAL CAPACITY, URBAN AND RURAL HIGHWAYS

NO. OF LANES		OUTPUT RATE, θ (VPH) [a]		RECOVERY RATE, R (VPH)	
NORMAL OPER.	RESTRICTED OPER.	0–10% TRUCKS [b]	> 10% TRUCKS [c]	0–10% TRUCKS [b]	> 10% TRUCKS [c]
2	1	1400	1350	3000	3000
3	2	2800	2700	4700	4500
4	3	4500	4350	6400	6200

[a] Input rates greater than the output rates given will normally result in formation of a queue.
[b] Urban.
[c] Rural.

in which

a = the time it takes a vehicle to travel through the restricted area, hours; and
Q = the number of vehicles arriving at the overlay area per hour in the overlay direction.

An equation for estimating the average delay per stopped vehicle (DO_1 or DN_1) for Method II is developed from equations formulated by Tanner and given by

$$DO_1(=DN_1) = \frac{(1 + e^{2aQ})(e^{aQ} - aQ - 1)}{2Q(e^{2aQ} - e^{aQ} + 1)PO_1} \qquad \text{(B-2)}$$

in which a, Q, and PO_1 are as previously defined.

When Method III or V is used, both PN_1 and DN_1 will equal zero. When Method III, IV, or V is used to handle traffic, DO_1 for all three methods and DN_1 for Method IV are calculated using the following equation developed and verified by the California Division of Highways (*6*):

$$DO_1 = \frac{H(Q - \theta)(R - \theta)}{2Q\,PO_1(R - Q)} \text{ if } Q > \theta \qquad \text{(B-3a)}$$

$$DO_1 = 0 \qquad \text{if } Q \leq \theta \qquad \text{(B-3b)}$$

in which

H = the number of hours per day that overlay construction takes place;
R = the recovery rate, vehicles per hour;
θ = the restricted output rate, vehicles per hour; and

Q and PO_1 are as previously defined.

It is assumed that the input rate Q is the same for both directions of travel and equals 6 percent of average daily traffic in rural areas and 5 percent of average daily traffic in urban areas. The output rate θ and recovery rate R are taken from Smith (*6*) and are given in Table B-5. Smith also gives the number of vehicles that will be stopped at the time when recovery begins; i.e., when all lanes are reopened for travel, and this number equals $H(Q - \theta)$. The average number of vehicles that will be stopped at any one time during overlay will be $H(Q - \theta)/2$. By assuming that each vehicle that is stopped stays stopped for some average amount of time D, which is assumed to equal 1/12 hr, not including the time stopped after recovery begins, and assuming that no vehicles are stopped after recovery begins, the total number of vehicles stopped can be estimated as $H^2(Q - \theta)/2D$ if $Q > \theta$ and is zero if $Q \leq \theta$. Thus, for Method III, IV, or V, the proportion of vehicles stopped PO_1 (and also PN_1 for Method IV) is estimated by dividing the total number of vehicles stopped by the total input of vehicles during overlay ($H \times Q$):

$$PO_1 = \frac{H(Q - \theta)}{2DQ} \text{ if } Q > \theta \qquad \text{(B-4a)}$$

$$PO_1 = 0 \qquad \text{if } Q \leq \theta \qquad \text{(B-4b)}$$

with the constraint that: if this value exceeds 1.0, it is assigned a value of 1.0.

More research, including field observations, needs to be

done on the precise number of stops under stop-and-go operation which results from congestion. Use of the foregoing formulas gives reasonably accurate estimates of hours of delay but may underestimate the vehicle operating costs associated with stop-and-go operation.

Formulas for calculating user costs appear in Chapter Four.

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