Pavement Management for Airports, Roads, and Parking Lots

Pavement Management for Airports, Roads, and Parking Lots

Second Edition

M. Y. Shahin

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To My Parents

Abdallah Shahin

Nazira Ibrahim

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Preface

Pavements need to be managed, not simply maintained. Although it is difficult to change the way we do business, it will be more difficult to explain to future generations how we failed to manage our resources and preserve our infrastructure.

When asked for reasons why they did not use the latest in pavement management technology, pavement managers gave many answers.

"The only time I have is spent fighting fires."

"We normally use a 2-inch overlay."

"Just spray the pavement black at the end of the year."

"I can't afford to do inspections; I'd rather use the money to fix the pavement."

Managers and engineers who have adopted pavement technology understand that pavement management is a matter of "Pay now, or pay much more later." Agencies are finding that they cannot afford to pay later; it is more costly to rehabilitate badly deteriorated pavements. Unfortunately, the pavement infrastructure managed by some agencies is at a point where a large sum of money will be needed for restoration. Agencies blessed with a good pavement infrastructure need to start a pavement management system as soon as possible. They need to: inventory the pavement infrastructure, assess its current and projected condition, determine budget needs to maintain the pavement condition above an acceptable level, identify work requirements, prioritize projects, and optimize spending of maintenance funds. The primary objective of this book is to present pavement management technology to engineering consultants, highway and airport agencies, and universities.

Features New to This Edition

The majority of the chapters in the first edition have been updated to reflect new developments since it was published in 1994. These updates include the following:

- Introduction of virtual databases, Chapter 2
- Automated distress data collection, Chapter 3
- Development of airfields, Foreign Object Damage (FOD) potential index, Chapter 3
- Determination of Aircraft Classification Number / Pavement Classification Number (ACN/PCN) using Nondestructive Deflection Testing (NDT), Chapter 4
- Determining budget requirements to meet specific management objectives, Chapter 10
- Project formulation based on network level work planning, Chapter 10

Three new pavement management special application chapters have been added: Impact of Bus Traffic on Pavement Costs (Chapter 12), Impact of Utility Cuts on Pavement Life and Rehabilitation Costs (Chapter 13), and Development of Council District Budget Allocation Methodology for Pavement Rehabilitation (Chapter 14). A new chapter has also been added that presents pavement management implementation steps (Chapter 15).

Acknowledgments

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1

Introduction

1.1 Background

In the past, pavements were maintained, but not managed. The pavement engineer's experience tended to dictate the selection of Maintenance and Repair (M&R) techniques with little regard given to life cycle costing nor to priority as compared to other pavement requirements in the network. In today's economic environment, as the pavement infrastructure has aged, a more systematic approach to determining M&R needs and priorities is necessary. Pavement networks must now be managed, not simply maintained.

Recent developments in microcomputers and pavement management technology have provided the tools needed to manage pavements economically. A Pavement Management System (PMS) provides a systematic, consistent method for selecting M&R needs and determining priorities and the optimal time of repair by predicting future pavement condition. The consequences of poor maintenance timing are illustrated in Figure 1–1. If M&R is performed during the early stages of deterioration, before the sharp decline in pavement condition, over 50% of repair costs can be avoided. In addition to cost avoidance, long periods of closure to traffic and detours can also be avoided. A PMS is a valuable tool that alerts the pavement manager to the critical point in a pavement's life cycle.

1.2 Project vs. Network Level Management

"Project-level" management often includes performing in-depth pavement evaluation and design for the pavement sections in the project. The end product is to select the specific M&R type(s) to be performed as well as the layer thicknesses when applicable. Project management can be performed with little or no consideration given to the resource requirements of other pavement sections in the network. This is acceptable as long as money is abundant, but this is seldom the case. In the past, most pavement

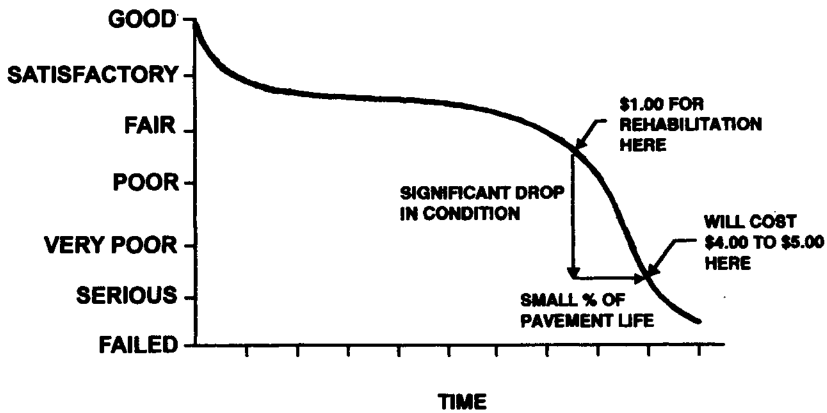

Figure 1–1. Conceptual illustration of a pavement condition life cycle.

engineers have been trained to work at the project level. Top management is now demanding budget projection that considers the agency's entire network before projects are prioritized and executed.

1.3 The Pavement Management Process

The ad hoc approach to pavement management normally leads to gradual deterioration in the overall condition of the pavement network and thus increased backlog of unfunded major M&R requirements. This approach consists of the habitual application of selected few M&R alternatives (such as 1.5 inch overlay) to pavement that are either in very poor condition or politically important. This is normally done regardless of the needs of the other pavements in the network.

A systematic approach to pavement management is needed to insure optimum return on investment. The following approach has evolved over the past thirty years as part of the development of the PAVER pavement management system (Micro PAVER 2004). The approach is a process that consists of the following steps:

1.3.1 Inventory definition (Chapter 2)

The pavement network is broken into branches and sections. A branch is an easily identifiable entity with one use, i.e. a runway, taxiway, roadway, etc.. Each branch is divided into uniform sections based on construction, condition, and traffic. A section can only be of the same pavement type, i.e. asphalt or concrete. A section can also be viewed as the smallest pavement area where major M&R, such as overlay or reconstruction, will be scheduled.

Section identification is normally performed using AutoCAD or Geographical Information systems (GIS). This allows the creation of GIS shape files which are useful to display pavement data.

1.3.2 Pavement Inspection (Chapters 3, 4, 5, and 6)

1.3.2.1 Airfield Pavements

At a minimum, pavement inspection consists of a distress survey every 1 to 5 years. Skid resistance measurement and Nondestructive Deflection Testing (NDT) are normally performed every 5 to 10 years. Runway longitudinal profile measurement is usually not performed unless there is a pilot complaint about pavement roughness.

1.3.2.2 Roadways and Parking Lots

It is recommended that distress surveys be performed every three years in order to meet the GASB 34 requirements. If automated data collection is used for the roadway survey, then both longitudinal and transverse profiles are measured. The longitudinal profile is usually measured for the right and left wheel path. NDT is usually not performed except for project level management.

1.3.3 Condition Assessment (Chapters 3, 4, 5, and 6)

1.3.3.1 Airfield Pavements

The inspection results are reduced to condition indicators that can be used for pavement management. A widely used distress index is the Pavement Condition Index (PCI). The PCI for airfields (Shahin et al. 1977), ASTM D5340, is a score from 0 to 100 that measures the pavement structural integrity (not capacity) and surface operational condition. It correlates with the needed level of M&R and agrees closely with the collective judgment of experienced pavement engineers.

The skid resistance data is reduced to a friction index for the runway. The NDT data is reduced to a structural index such as the Aircraft Classification Number/ Pavement classification Number (ACN/PCN).

1.3.3.2 Roadways and Parking Lots

Similar to airfield pavements, a PCI for roads and parking lots is calculated from the gathered distress data (Shahin et al. 1981), ASTM 6433. The longitudinal profile is used to calculate the International Roughness Index (IRI), ASTM E1926. The pavement section IRI is the average IRI of the right and left wheel path. The transverse profile is used to calculate the pavement rutting depth or rutting index.

1.3.4 Condition Prediction (Chapter 7)

There is no such thing as one prediction model that will work for all locations and conditions. Therefore, it is important that the management system includes a prediction modeling engine that can be used to formulate different models for different locations and conditions. The models are used to predict the future condition of the pavement sections assuming that the traffic will continue to be the same as in the past. An accurate condition prediction is also important for the analysis of different budget consequences.

1.3.5 Condition Analysis (Chapter 9)

Condition analysis allows managers to compare past, current, and future conditions, assuming no major M&R is performed. This provides managers with the ability to assess the consequence of past budget decisions and the value of having a management system, especially if the system has been in place for several years.

1.3.6 Work Planning (Chapters 8, 10, and 11)

Work planning provides the ability to determine budget consequence for a specified budget or, alternately, budget requirements to meet specified management objectives. Typical management objectives include maintaining current network condition, reaching a certain condition in x years, or eliminating all backlog of major M&R in x years. Regardless of the analysis scenario, the output should include the recommended M&R category for each pavement section for each year of the analysis. Projects are formulated by grouping sections to minimize cost and traffic delays.

1.4 Book Organization

The book is organized in the same logical sequence of the pavement management process. Pavement network definition is presented in Chapter 2. Pavement condition measurement is presented in Chapters 3 through 6. The chapters cover distress, deflection, roughness, and skid, respectively. Pavement condition prediction is presented in Chapter 7. It is important to realize that pavement condition prediction is an important part of the pavement management process. The accuracy of the prediction will influence the accuracy of both the network and project level analysis. Chapter 8 presents an introduction to M&R techniques as a background for work planning. The network level pavement management analysis is presented in Chapters 9 and 10. Chapter 9 presents the inventory and condition reporting while Chapter 10 presents the M&R work planning. The project level analysis is presented in Chapter 11. Chapters 12 through 14 present special applications where pavement management technology is used to address specific questions. Chapters 12 and 13 address the impact of buses and utility cut patching on pavement life and rehabilitation cost. Chapter 14 addresses M&R budget allocation among city council districts. Chapter 15 provides a summary of pavement management implementation steps and benefits. Figure 1–2 is a flow chart of the book organization.

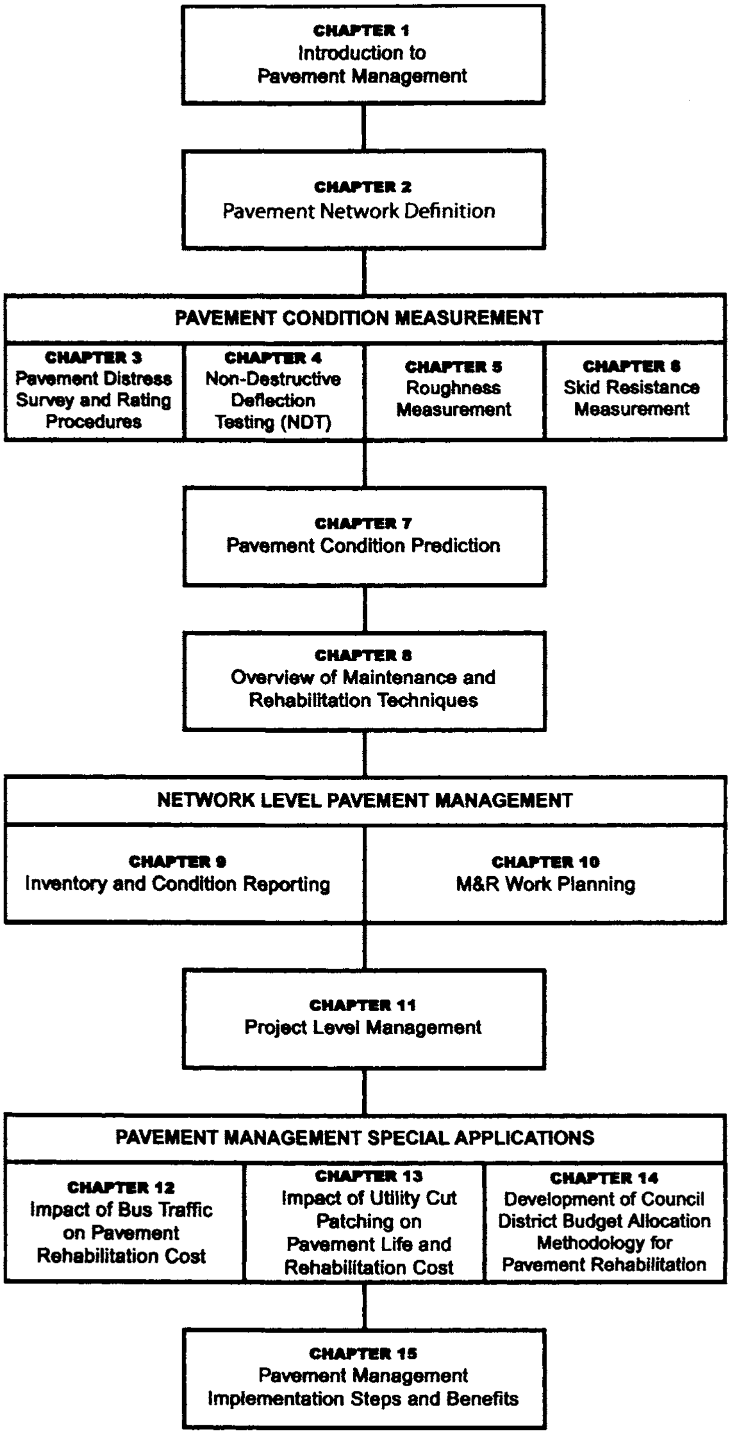

Figure 1-2. Book Organization.

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2

Pavement Network Definition

This chapter presents guidelines for identifying and defining pavement networks, branches, and sections. These guidelines should be viewed just as guidelines and may be modified as necessary to accommodate unusual situations or specific agency requirements. The initial data collection for each pavement section can be very time consuming. This normally occurs if an extensive coring or testing program is undertaken during the initial setup of the pavement management system (PMS). By following the guidelines presented in this chapter, costly errors can be avoided the first time through, resulting in an effective database and quick return on investment in starting a PMS.

2.1 Network Identification

The first step in establishing a PMS is the network identification. A network is a logical grouping of pavements for M&R management. The pavement manager may be responsible for the management of roads, parking lots, airfields, and other types of surfaced or unsurfaced vehicular facilities. The manager should decide which facility types will be identified as separate networks. Other factors to consider besides facility types are funding sources, minimum operational standards, and geographical location. The following are examples of network identifications by different agencies:

- An airport may identify its pavements as two networks, one for airfields and one for roads and parking lots.
- A military base may identify its roads as two networks, one for family housing and one for non-family housing.
- A large city may identify its pavements as many networks, one for each city council district. Alternatively, it may identify all the pavements as one network and then create a separate computerized database for each council district.

- A commercial industry with many geographical locations, such as a department store or a hotel chain, may identify the pavements at each geographical location as one network.

2.2 Branch Identification

A branch is a readily identifiable part of the pavement network and has a distinct use. For example, an individual street or a parking lot would each be considered a separate branch of the pavement network. Similarly, an airfield pavement such as a runway or a taxiway would each be considered a separate branch.

Branch naming conventions should be implemented that are logical to the pavement managers and PMS users. To begin, each street on the network map is identified as a separate branch and given the street name. The process can also be used on parking lots; however, parking lots that do not already have assigned names can be given descriptive names to associate them with their location. For example, the closest building numbers can be used as part of the name. Also, depending on their size and location, many smaller lots can be combined to form one branch if necessary.

2.3 Section Identification

A branch does not always have consistent characteristics throughout its entire area or length. Consequently, branches are divided into smaller components called "sections" for managerial purposes. A section should be viewed as the smallest management unit when considering the application and selection of major maintenance and repair (M&R) treatments. A section must also be of the same surface type (for example, concrete, asphalt over concrete, etc.). Each branch consists of at least one section, but may consist of more if pavement characteristics vary throughout the branch. Factors to consider when dividing branches into sections are: pavement structure, construction history, traffic, pavement rank (or functional classification), drainage facilities and shoulders, condition, and size. Following is a discussion of each of these factors.

2.3.1 Pavement Structure

The pavement structure is one of the most important criteria for dividing a branch into sections. The structural composition (thickness and materials) should be consistent throughout the entire section. Construction records are a good source of this information. The records may be verified by taking a limited number of cores. An extensive coring program should be avoided at the start of the PMS implementation unless resources are unlimited.

A nondestructive deflection testing (NDT) program may also be performed (see Chapter 4) to provide information regarding structural uniformity. Figure 2–1 shows how the results of NDT were used to divide an approximate one–mile branch into two sections, even though the surface appearance was about the same.

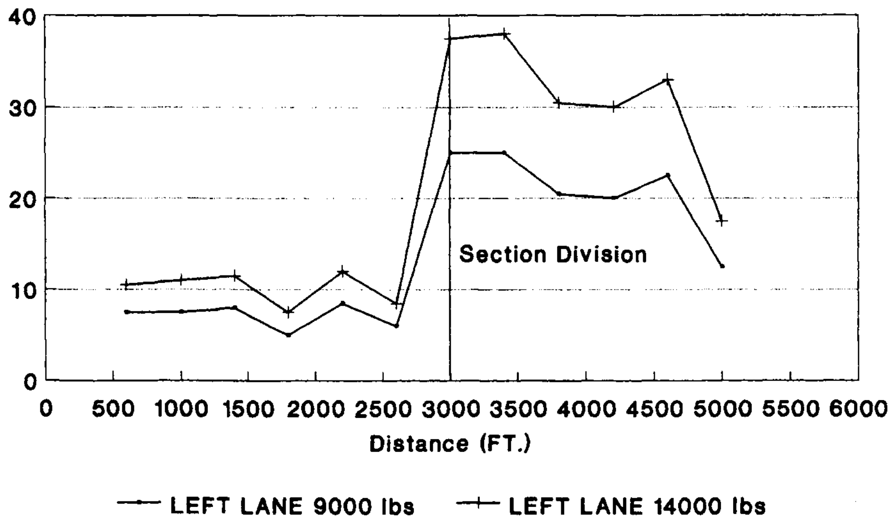

Figure 2-1. Example Use of Nondestructive Deflection Testing (NDT) to Define Pavement Sections.

When initiating a PMS, limiting pavement coring and NDT will minimize costs. When information from additional coring or NDT becomes available in the future, they can be used to verify the pavement sectioning.

2.3.2 Construction History

Pavements constructed during different years, by different contractors, or using different techniques should be considered separate sections. Areas that have received major repairs, such as many slab replacements or patches, should also be divided into separate sections.

2.3.3 Traffic

The volume and load intensity of traffic should be consistent within each individual section. For roads and streets, primary consideration should be given to the number of lanes and truck traffic. For streets with four or more lanes and two directions of traffic, a separate section may be defined for each direction, particularly if the highway is divided. A significant change in truck volume between directions should be a major consideration in section definition. An intersection could be treated as a separate section only if it is likely to receive major rehabilitation independent of the surrounding pavement.

For airfield pavements, it is important that traffic channelization be considered, particularly for aprons and runways. Figure 2–2 is an example runway branch divided into nine sections based on traffic channelization. The runway width of 150 ft. was divided into three lanes, each 50 ft. wide. Traffic on runways is normally channelized within the central 50 to 75 ft. However, the outside areas do receive traffic near taxiway exits, which should be taken into consideration when dividing the runway into sections.

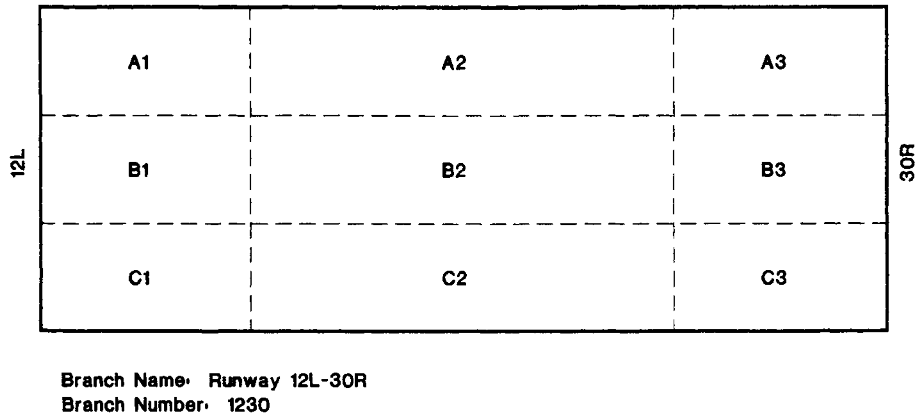

Figure 2-2. Example Runway Division into Sections.

2.3.4 *Pavement Functional Classification (Rank)*

A change in rank normally reflects a change in traffic. If the rank changes along the branch length (for example, from primary to secondary or, from arterial to collector), a section division should be made.

2.3.5 *Drainage Facilities and Shoulders*

To the extent that drainage and shoulder provisions affect pavement performance, it is recommended that these provisions be consistent throughout a section.

2.3.6 *Condition*

Systematic changes in pavement condition should be considered when defining pavement sections. Condition is an important variable because it reflects many of the factors discussed above. Changes in distress types, quantities, or causes should be taken into consideration. Experience has shown that a combination of a distress condition index and NDT profiles leads to very successful section definitions. Figure 2–3 shows the deflection and distress index profiles used to divide a runway into distinct sections.

2.3.7 *Section Size*

Section size can have a considerable impact on the economics of implementation. Defining very short sections, to ensure uniformity, requires a higher implementation effort and cost. The sections may also be too small to schedule individual M&R work productively. If they are too large, the characteristics may not be consistent across the entire area. This situation could result in nonuniform sections which in turn results in inefficient design and budget decisions. The same guidelines for road and street section sizes apply to parking lots. In the case of very small parking lots (designed for few vehicles), the small parking lots can be grouped into one section

It is also recommended that sections be numbered in a consistent way. For example, west to east, north to south, and clockwise for circular roads.

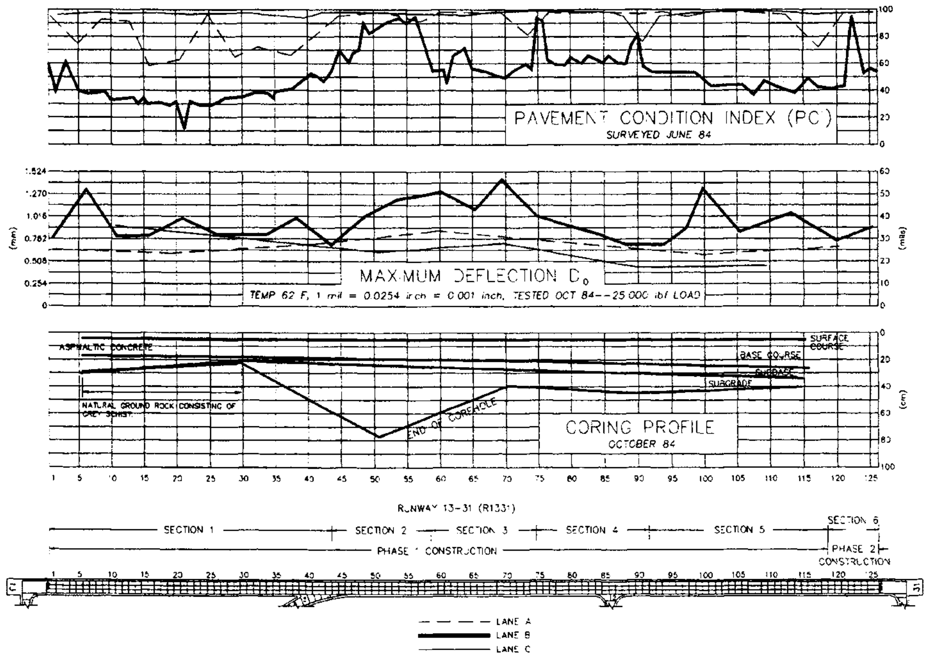

Figure 2-3. Example Use of Distress Condition Index, Deflection, and Coring Profiles for Runway Division into Sections. (From Engineering and Research International Consulting Reports 1984).

2.4 Examples of Network Division into Branches and Sections

Figure 2–4 – *Road Network*; The sections identifications clearly show to which section the road intersection belongs.

Figure 2–5 – *Parking Area*; The driveways to the parking areas are identified as separate sections (sections 2 and 4).

Figure 2–6 – *Department Store/Hotel*; A total of three branches are defined: Road, Parking, and Receiving. The Parking branch is divided into sections to reflect the higher volume of parking closest to the store/hotel entrance.

Figure 2–7 – *Civil Aviation Airfield pavement*; The network is divided into three branches; Runway 8–26, Taxiway, and Apron. The runway, 4,000 ft. long, is divided into two sections, A and B, based on construction history, condition, and traffic. The runway keel is not identified as a separate section due to the width of the runway which is only 100 ft.

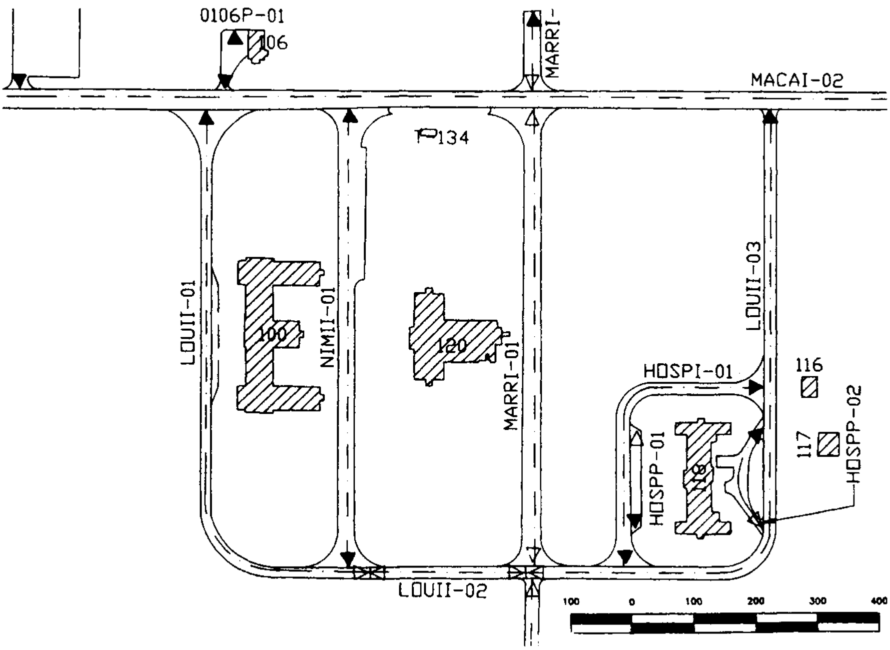

Figure 2-4. Example Road Section Definition For a Road Network. (From Shahin and Walther 1990)

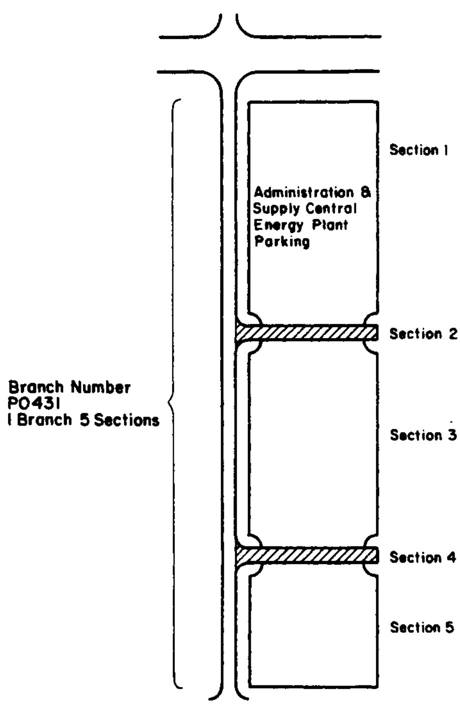

Figure 2-5. Example Parking Area Section Definition. (From Shahin and Walther 1990)

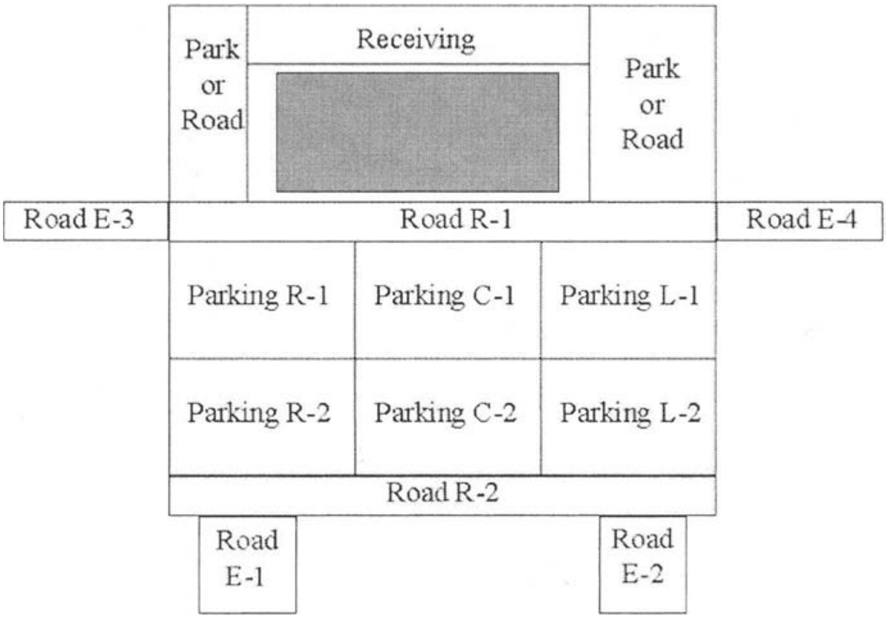

Figure 2-6. Example Department Store Pavement Section Definition.

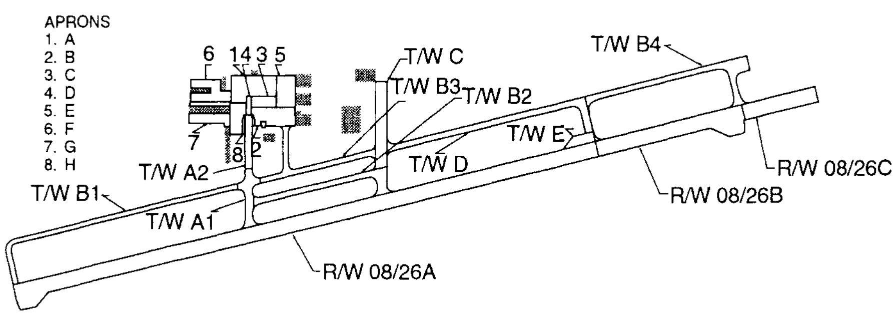

Figure 2-7. Example Civil Aviation Airport Section Definition. (Ohio Department of Transportation Aviation 2004).

2.5 Other Network Definition Considerations for Computerized PMS

2.5.1 Database Combine/Subset

A database in a computerized PMS may contain more than one network. A major advantage to smaller databases is efficient data entry and report generation. However, this advantage can be achieved easily if the computerized PMS allows for the capability of combining or subsetting databases as needed.

2.5.2 Key Field Unique ID

In some computerized systems, such as the Micro PAVER system, when the user makes an entry in a key field (such as Network ID, Branch ID, or Section ID) for the first time, the entry is assigned an additional hidden unique ID that remains associated with the entry even though the user may change the value of the entry in the future. This is a good feature because a user is able to change network, branch, or section name at any time without having to transfer or re-link the associated data, such as inspections or work history. However, for example, if a large city decides to define the pavement in each Council District as a separate network, each network will be automatically assigned a hidden Unique ID. If the networks are combined later, they will retain their unique identity even if the names are changed to be the same.

Therefore, in the above example, if the city wishes to have the ability to place all the pavements in one network at some time in the future, it is best to start with all the pavements in one network (thus one Unique ID). The Micro PAVER software database combine/subset capability can be used then to break the network into different databases (i.e., one for each Council District).

2.5.3 Branch Identification (Branch ID)

In Micro PAVER, each branch is identified in two ways: (1) by an alphanumeric descriptive name called the "Branch Name" and (2) by an alphanumeric code called the "Branch ID." The Branch ID is a unique code used to help store and retrieve data from the database. In selecting the code, review of existing codes at the agency is recommended to ensure compatibility. Also, some reports may list the Branch ID and not the Branch Name. For this reason, abbreviating the Branch Name as a Branch ID may make reports easier to read. For example, the Branch Name "Green Street" could be given the Branch ID "GREEN"; similarly, runway 12–30 would be given the ID "R1230."

2.5.4 Inventory User-Defined Fields

The Micro PAVER system allows the user to define additional inventory fields at the Network, Branch, and Section levels. These fields can be used for generating queries or sorting information. The following are examples of these fields.

2.5.4.1 Example Additional Network-Level Fields

a. Geographical location—this is particularly useful for a commercial industry with pavement networks located in different geographical locations (i.e., different states or countries).

b. Climatic zone—an example use of this field is for combining networks to develop condition prediction models.

c. Classification—an example use of this field is for grouping airports by category of use or, in the case of commercial industry, for grouping by stores by different class of service.

d. Funding source—this is especially useful if the networks are defined based on source of M&R funds.

2.5.4.2 Example Additional Branch-Level Fields

a. Route designation—e.g., state route.

b. Shared use—e.g., use of a runway by both civilian and military.

2.5.4.3 Example Additional Section-Level Fields

a. Maintenance District ID

b. Council District ID

c. Presence of curb and gutter

d. Bus traffic

2.5.5 Virtual Database Formulation

Virtual databases are formulated by creating virtual sections from the physically defined pavement sections. The primary purpose of virtual databases is data presentations and reporting. A virtual section can consist of any number of physical sections that may belong to different branches and networks. For example, an airfield virtual database may contain only three virtual sections; one for runways, one for taxiways, and one for aprons. Such a database may be very useful when briefing upper management.

In formulating a virtual section, the user will have to select the data aggregation rules. For numerical conditions, e.g. PCI, the aggregation can be based on any of the following rules; area weighted average, arithmetic average, average minus one standard deviation, minimum value, etc.

More than one virtual database can be created for a given physical database. Each of the virtual databases can be used for a different reporting requirement.

References

Engineering and Research International (ERI). Consulting Reports, (1984); Savoy, IL.

Ohio Department of Transportation Aviation. 2004. Personal Communication. Andrew Doll and Mark Justice.

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U.S. Army Engineering Research and Development Center-Construction Engineering Research Laboratory (ERDC-CERL), 2004. Micro PAVER Pavement Management System, 2004. e-mail: paver@cecer.army.mil web: www.cecr.army.mil/paver

3

Pavement Condition Survey and Rating Procedure

3.1 Overview

An important feature of a pavement management system (PMS) is the ability to determine both the current condition of a pavement network and predict its future condition. To predict condition reliably, an objective, repeatable rating system for identifying the pavement's condition must be used. The pavement distress condition rating procedure presented here is the Pavement Condition Index (PCI) developed by the U.S. Army Corps of Engineers (Shahin et al. 1976-1994). The use of PCI for airfield pavement, roads, and parking lots has received wide acceptance and has been formally adopted as standard procedure by many agencies worldwide. These agencies include the Federal Aviation Administration, The U.S. Department of Defense, the American Public Works Association, and many others. The PCIs for airfields and roads have also been published as ASTM standards, D5340 and D6433, respectively.

The PCI is a numerical index, ranging from 0 for a failed pavement to 100 for a pavement in perfect condition (Fig. 3–1). Calculation of the PCI is based on the results of a visual condition survey in which distress type, severity, and quantity are identified. The PCI was developed to provide an index of the pavement's structural integrity and surface operational condition. The distress information obtained as part of the PCI condition survey provides insight into the causes of distress and whether it is related to load or climate.

The degree of pavement deterioration is a function of distress type, distress severity, and amount or density of distress. Producing one index that would take into account all three factors was a considerable challenge. To overcome this challenge, "deduct values" were introduced as a type of weighing factor to indicate the degree of effect that each combination of distress type, severity level, and distress density has on pavement condition. The deduct values were developed based on in-depth knowledge of pavement behavior, input from many experienced pavement engineers, field testing and evaluation of the procedure, and accurate descriptions of distress types and severity levels. Figure 3–2 shows a simplified diagram of the process used to develop the deduct values. The sum of the deduct values is corrected based on the number and value of the deducts and the corrected sum is subtracted from 100 to obtain the PCI.

To determine the PCI of a pavement section, the section is first divided into inspection units, called sample units, as described in Section 3.2. Section 3.3 presents methods for determining the number of sample units and identifying which ones to inspect. Section 3.4 presents the survey procedures for asphalt and concrete pavement as well as unsurfaced roads. Section 3.5 covers calculation of the PCI for each sample unit, and determination of the average PCI for a pavement section. Section 3.6 presents an alternative distress survey procedure using automated distress data collection. Section 3.7 compares manual and automated distress data collection results. Section 3.8 discusses the effect of deviating from standard sample unit size on PCI accuracy. Section 3.9 describes how to calculate the PCI using the Micro PAVER system.

3.2 Dividing Pavement Into Sample Units

A sample unit is a conveniently defined portion of a pavement section designated only for the purpose of pavement inspection. For unsurfaced and asphalt surfaced roads (including asphalt over concrete), a sample unit is defined as an area 2500 ± 1000 sq ft. For asphalt surfaced airfields, each sample unit area is defined as 5000 ± 2000 sq ft. It should be noted that sample unit sizes close to the recommended mean are preferred for accuracy (see Section 3.8).

For concrete roads and airfields with joints spaced less than or equal to 25 ft, the recommended sample unit size is 20 ± 8 slabs. For slabs with joints spaced greater than 25 ft, imaginary joints less than or equal to 25 ft apart and in perfect condition, should be assumed. For example, if slabs have joints spaced 60 ft apart, imaginary joints are assumed at 20 ft. Thus, each slab would be counted as three slabs for the purpose of pavement inspection.

An important consideration in dividing a pavement section into sample units is convenience. For example, an asphalt pavement section 22 ft wide by 4720 ft long (Fig. 3–3) can be divided into sample units 22 ft wide by 100 ft long, for a sample unit size of 2200 sq ft. Because of the section's length some sample units may have to be a different length than the others. Not all sample units are required to be the same size, but they do have to fit within the guidelines for recommended sample unit size to ensure an accurate PCI. The section in Figure 3–3 can be divided into 46 units that are each 100 ft long, plus one unit that is 120 ft long. Therefore, this last sample unit has an area of 22 ft by 120 ft, or 2640 sq ft. Figure 3–4 is an example of roads divided into sections and sample units. The sample units in this example are consistently numbered west to east, and north to south. Figure 3–5 is an example parking lot divided into sample units. Figure 3–6 shows an example airfield pavement network divided into sample units. Figure 3–7 is an example civil aviation airfield divided into sections and sample units.

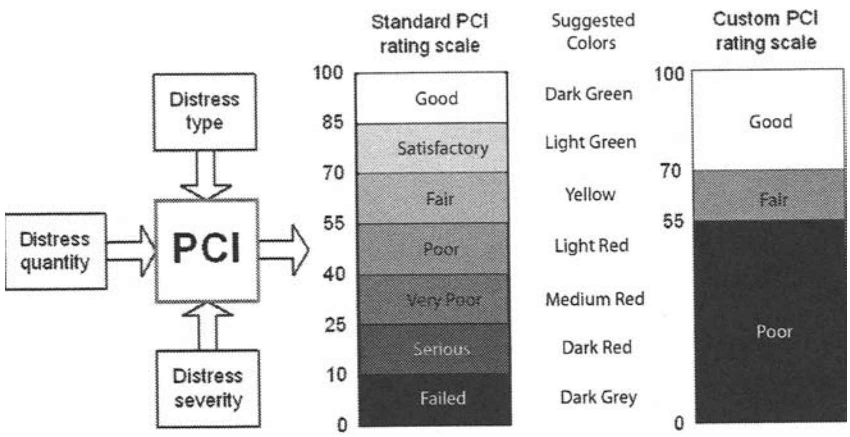


Figure 3-1. Pavement Condition Index (PCI) Rating Scale.

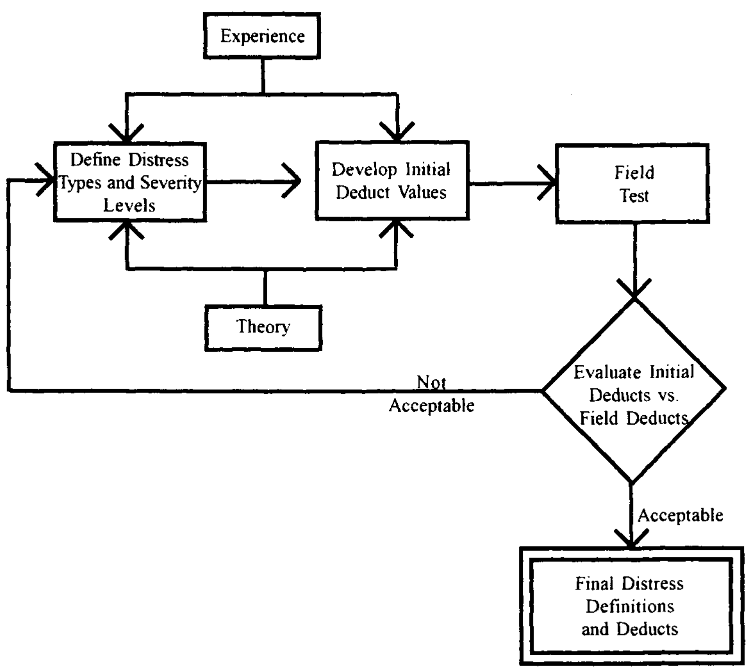


Figure 3-2. Process for Developing the PCI Deduct Values.

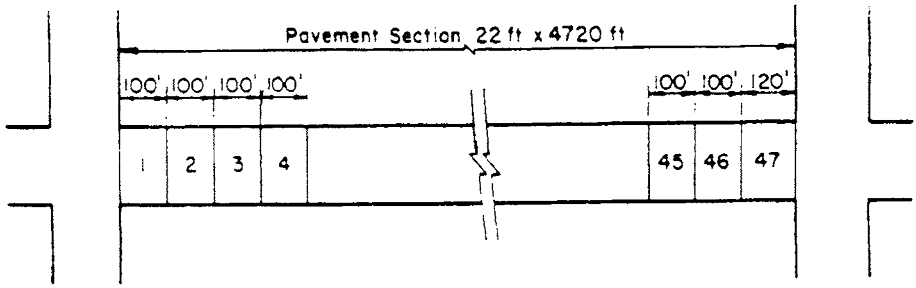


Figure 3-3. Example of an Asphalt Section Divided into Sample Units.

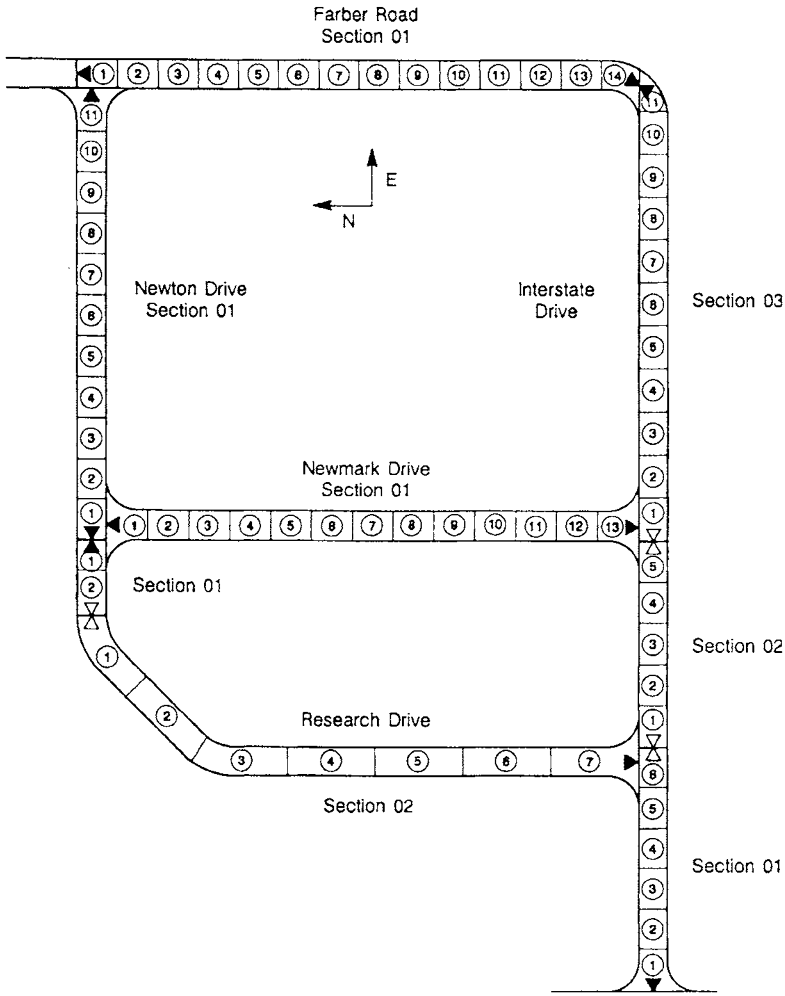


Figure 3-4. Example Road Network Divided into Sample Units.

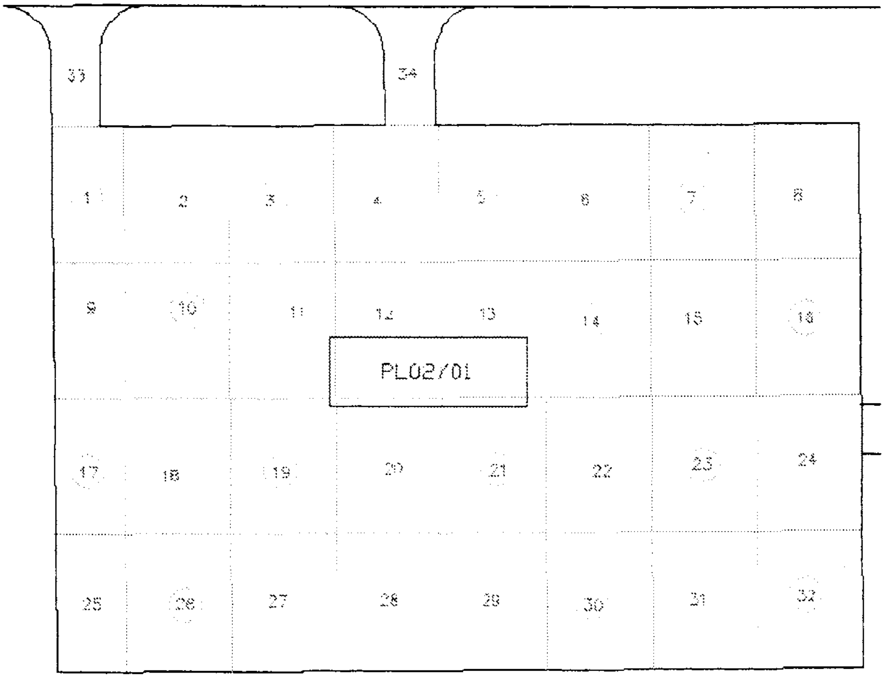


Figure 3-5. Example Parking Lot Divided into Sample Units.

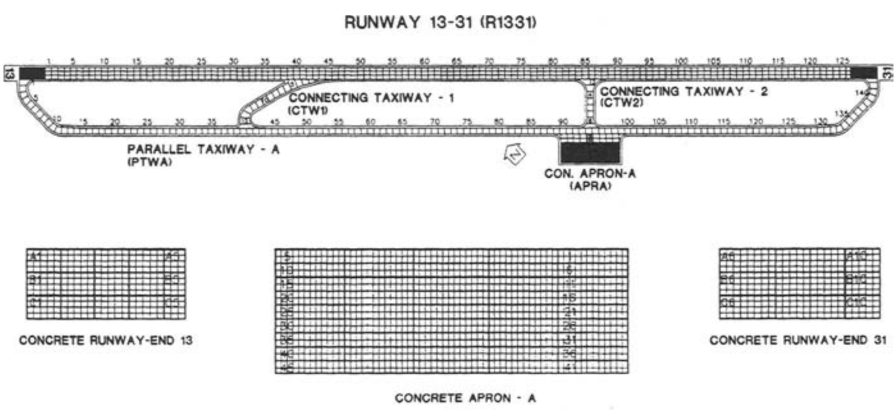


Figure 3-6. Example Airfield Pavement Network Divided into Sample Units. (From ERI Consulting Reports 1984)

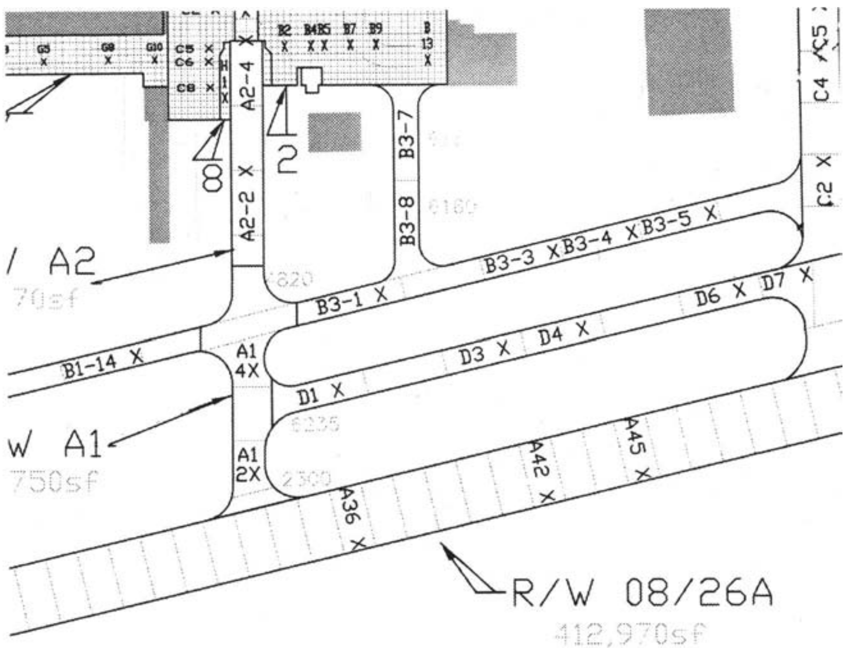


Figure 3-7. Example Civil Aviation Airfield Divided into Sections and Sample Units. (Ohio Department of Transportation Aviation 2004)

For each pavement section being inspected, it is strongly recommended that sketches be kept showing the size and location of sample units. These sketches can be used to relocate sample units for future inspections. In a computerized management system, these sketches should be stored as image(s) associated with the pavement section.

Guidance on the minimum number of sample units from a pavement section to be inspected is provided in Section 3.3.

3.3 Determining Sample Units to Be Surveyed

The inspection of every sample unit in a pavement section may require considerable effort, especially if the section is large. To limit the resources required for an inspection, a sampling plan was developed so a reasonably accurate PCI could be estimated by inspecting only a limited number of the sample units in the pavement section. The required degree of sampling depends on the use of the pavement and whether the survey is conducted at the network or project level.

If the objective is to make network-level decisions such as budget planning, a survey of a limited number of sample units per section is sufficient. If the objective is to evaluate specific pavement sections for project development, a higher degree of sampling for a section may be required.

3.3.1 Project-Level Inspection

3.3.1.1 Determining the Number of Sample Units to Be Inspected

Management at the project level requires accurate data for the preparation of work plans and contracts. Therefore, more sample units are inspected than are usually sampled for network-level management. The first step in sampling is to determine the minimum number of sample units (*n*) that must be surveyed to obtain an adequate estimate of the section's PCI. This number is determined for a project-level evaluation by using the curves shown in Figure 3–8. Using this number, a reasonable estimate of the true mean PCI of the section will be obtained. There is 95% confidence that the estimate is within ± 5 points of the true mean PCI (the PCI obtained if all the sample units were inspected). The curves in Figure 3–8 were constructed using Equation 3–1:

$$n = \frac{N \times s^2}{\left(e^2/4\right)\left(N-1\right)+s^2} \tag{3-1}$$

where

N = total number of sample units in the pavement section

e = allowable error in the estimate of the section PCI (e was set equal to 5 when constructing the curves of Fig. 3–8)

s = standard deviation of the PCI between sample units in the section.

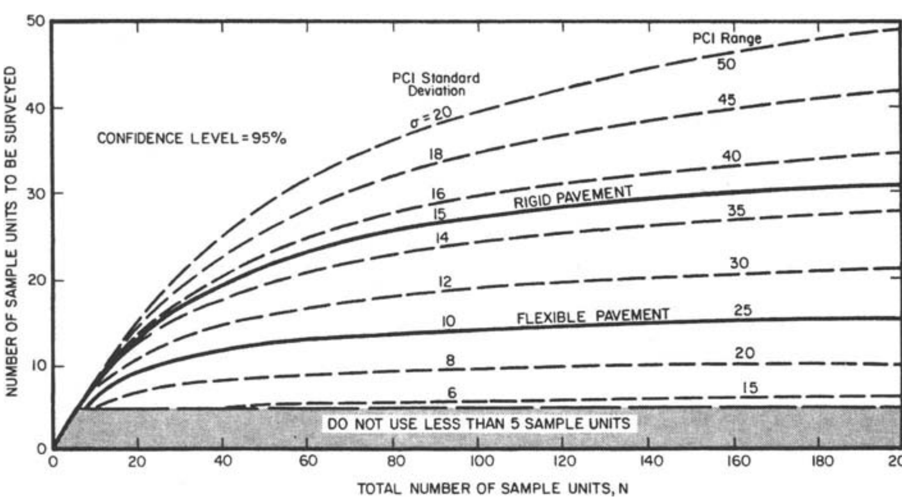


Figure 3-8. Selection of the Minimum Number of Sample Units. (From Shahin et al. 1976-84)

The curves in Figure 3–8 can be used based on the PCI standard deviation among sample units or PCI range (i.e., lowest sample unit PCI subtracted from the highest sample unit PCI). When performing the initial inspection, the PCI standard deviation for a pavement section is assumed to be 10 for asphalt concrete (AC) surfaced pavements (or PCI range of 25) and 15 for Portland cement concrete (PCC) surfaced pavements (or PCI range of 35). These values are based on field data obtained from many surveys; however, if local experience is different, the average standard deviations reflecting local conditions should be used for the initial inspection. For subsequent inspections, the actual PCI standard deviation or range (determined from the previous inspection), should be used to determine the minimum number of sample units to be surveyed. When the total number of samples within a section is less than five, it is recommended that all of the sample units be surveyed.

3.3.1.2 Selecting Sample Units to Inspect

It is recommended that the sample units to be inspected be spaced equally throughout the section, and that the first one be chosen at random. This technique, known as "systematic random," is illustrated in Figure 3–9 and described by the following three steps:

1. The sampling interval (i) is determined by $i = N/n$, where N equals the total number of available sample units and n equals the minimum number of sample units to be surveyed. The sampling interval (i) is rounded off to the smaller whole number (e.g., 3.6 is rounded to 3.0).
2. Random start(s) is/are selected at random between sample unit 1 and the sampling interval (i). For example, if $i = 3$, the random starts would be a number from 1 to 3.
3. The sample units to be surveyed are identified as s, $s + i$, $s + 2i$, etc. If the selected start is 3, and the sampling interval is 3, then the sample units to be surveyed are 6, 9, 12, etc.

Total Number of Sample Units In Section (N) = 47

Minimum Number of Units to be Surveyed (n) = 13

Interval (i) $= \frac{N}{n} = \frac{47}{13} = 3.6 = 3$

Random Start (S) = 3

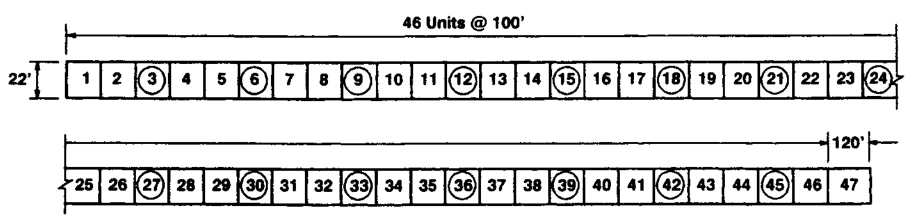


Figure 3-9. Example of Systematic Random Sampling.

3.3.2. Network-Level Inspection

3.3.2.1 Determining the Number of Sample Units to Be Inspected

A network-level survey can be conducted by surveying only a few sample units per section. Figure 3–10 provides an example of criteria used by agencies for determining the number of sample units to survey at the network level. The number of units to be inspected (n) is increased by 1 for every increase of five units in the section (N) until N equals 15. When N equals 16 to 40, the value of n is set at 4. When the value of $N > 40$, n is set at 10 % of N and rounded up to the next whole sample unit. For example, if $N = 52$, then $n = 6$ (rounded up from 5.2).

Figure 3–11 differs slightly from Figure 3–10. It is based on Equation 3–1 assuming a standard deviation, s, equal to the allowable error, e, of 5. There is no scientific basis for this assumption, but it provides a consistent method for selecting the number of units to inspect for different size sections. The criteria in Figure 3–11 result in a higher n when $N < 5$, whereas those in Figure 3–10 result in a higher n when N is > 40.

The values in Figure 3–10 and 3–11 are provided as examples. The degree of sampling presented in either table is sufficient for developing network-level maintenance work plans, assessing the condition of the pavement, and identifying candidate sections that may warrant detailed project-level inspections.

3.3.2.2 Selecting Sample Units to Inspect

When selecting sample units to inspect, as recommended in Figure 3–10 or Figure 3–11, the sample units selected should be representative (not random) of the overall condition of the section. The main objective for budget estimating and network condition assessment is to obtain a meaningful rating with the least cost.

No. of Sample Units in Section (N)	No. of Units to be Inspected (n)
1 to 5	1
6 to 10	2
11 to 15	3
16 to 40	4
over 40	10% (round up to next whole sample unit)

Figure 3-10. Example of Network Level Sampling Criteria Used by Some Agencies.

No. of Sample Units in Section (N)	No. of Units to be Inspected (n)
1	1
2 to 4	2
5 to 20	3
over 20	4

Figure 3-11. Network Level Sampling Based on Eq. 3-1 ($e = 5, s = 5$).

3.3.3 Special Considerations

3.3.3.1 Airfield Pavement Inspection

Airfield pavements are normally held to higher maintenance standards than roads and parking lots because loose objects from spalled pavements or unfilled cracks can cause serious damage to aircraft engines and propellers. On the central 50 or 75 ft of runways (the keel section), where 95% of the traffic takes place, it is not unreasonable to survey 50% of the sample units, or even every sample unit. On the outside of a runway, and on taxiways and aprons, a 25% to 33% sampling may be sufficient. This level of inspection may be appropriate both at the network and project levels.

3.3.3.2 Roads and Parking Lots Pavement Inspection

For roads and parking lots, it is difficult to justify a high degree of sampling unless a project-level evaluation is being performed. A 10% to 25% degree of sampling, as presented in Figures 3–10 and 3–11, is normally sufficient at the network level. The project-level inspection is sampled as discussed in Section 3.3.1. However, every sample unit may be surveyed if accurate distress quantities are to be determined for contractual purposes.

3.3.3.3 Selecting Additional Sample Units

One of the major drawbacks to both systematic random sampling at the project level and representative sampling at the network level is that sample units in exceptionally bad condition may not necessarily be included in the survey. At the same time, sample units that have a one-time-occurrence type of distress (e.g., railroad crossings) may be included inappropriately as a random sample.

To overcome these drawbacks, the inspection should identify any unusual sample units and inspect them as "additional" units rather than as random or representative units. When additional sample units are included in the survey, the calculation of the Section PCI is slightly altered to prevent extrapolation of the unusual conditions across the entire section. This procedure is discussed in more detail in Section 3.5.

3.4 Performing the Condition Survey

The procedures used to perform a PCI condition survey will vary depending on the surface type of the pavement being inspected. For all surface types, the pavement section must first be divided into sample units and the units to be inspected chosen as described in the previous section. The inspection procedures for asphalt and concrete surfaced pavements and unsurfaced roads are described in the sections that follow. Blank field condition survey sheets are provided in Appendix A. The distress definitions must be followed so that an accurate PCI can be determined. These definitions are provided in Appendices B and C for surfaced roads, D and E for airfield pavements, and F for unsurfaced roads.

3.4.1 *Asphalt-Surfaced Pavements*

The condition survey of all asphalt, tar-surfaced, and asphalt-over-concrete pavements involves the following:

Equipment

Inspectors need a hand odometer to measure distress lengths and areas, a straightedge, a ruler to measure the depth of ruts or depressions, and the PCI distress manual.

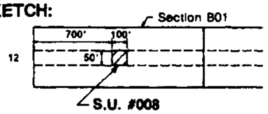
Procedure

A sample unit is inspected by measuring the distress type and severity according to the PCI distress manual, and recording the data on the flexible pavement survey sheet. Figure 3–12 is an example asphalt-surfaced airfield sample unit condition survey sheet.. The distress definitions and procedures for measuring distresses for asphalt-surfaced roads and airfield pavements are provided in Appendices B and D, respectively. These definitions should be followed closely when performing the PCI survey. One data sheet is used for each sample unit. The distress codes provided on the data sheets correspond to the distress identification codes used in the Micro PAVER system. Each row on the data sheet is used to represent a distress type at a given severity level.

In Figure 3–12, for example, number 48 (longitudinal and transverse cracking, low severity) is measured in linear feet, so 10 indicates 10 ft of low-severity cracking and so on. All distress data are used to compute the PCI of the sample unit, as discussed later in this chapter.

AIRFIELD ASPHALT PAVEMENT CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT

SKETCH:



BRANCH R1230 SECTION B01 SAMPLE UNIT 008
SURVEYED BY MYS DATE MAR/15/92 SAMPLE AREA 5000 sf

41. Alligator Cracking	45. Depression	49. Oil Spillage	53. Rutting
42. Bleeding	46. Jet Blast	50. Patching	54. Shoving from PCC
43. Block Cracking	47. Jt. Reflection (PCC)	51. Polished Aggregate	55. Slippage Cracking
44. Corrugation	48. Long. & Trans. Cracking	52. Raveling/Weathering	56. Swell

DISTRESS SEVERITY	QUANTITY										TOTAL	DENSITY %	DEDUCT VALUE
48 L	10	20	17								47	0.94	4.8
48 M	7	9									16	0.32	6.7
41 L	53										53	1.06	21.0
45 L	10	5									15	0.3	1.6
53 L	20	45	10								75	1.5	17.1
53 M	25										25	0.5	20.1

Figure 3-12. Example Airfield AC Sample Unit Condition Survey Sheet.

3.4.2 Concrete-Surfaced Pavements

The following equipment and procedure are used to inspect both plain and jointed reinforced concrete pavements:

Equipment

Inspectors need a hand odometer for measuring slab size, a straightedge and ruler for measuring faulting and lane/shoulder drop-off, and the PCI distress manual.

Procedure

The inspection is performed by recording the distress found in each slab on the concrete pavement field inspection data sheet. Figure 3–13 is an example PCC airfield sample unit condition survey sheet. The definitions and procedures for measuring distresses for concrete-surfaced roads and airfield pavements are provided in Appendices C and E, respectively. These definitions should be followed very closely when conducting the PCI survey. One data sheet is used for each sample unit. The sample unit is sketched using the dots as joint intersections. The appropriate number code for each distress found in the slab is entered in the square representing the slab. These number codes correspond to the distress identification codes used in the Micro PAVER system. The letter L (low), M (medium), or H (high) is included with the distress number code to indicate the severity level of the distress. For example, 62L indicates that a slab has low-severity linear cracking.

Space is provided on the concrete pavement inspection data sheet for summarizing the distresses for the sample unit. Remember to record the overall severity level of the joint sealant (i.e., L, M, or H). The number of slabs should default to the number of slabs in the sample unit. Calculation of the PCI is discussed in Section 3.5.

3.4.3 Unsurfaced Roads

The unsurfaced road PCI procedure was developed by the U.S. Army Cold Regions Research and Engineering Laboratory through funding from the Federal Highway Administration (Eaton, et al. 1988). An unsurfaced road, as used in this procedure, is defined as a road that does not have AC, PCC, or surface treatment.

Equipment

A hand odometer, straightedge, ruler, and the unsurfaced road distress manual are needed.

Procedure

Two kinds of inspections are performed on unsurfaced roads: a "windshield inspection" and an inspection based on detailed distress measurement within the sample units.

The windshield inspection is conducted by driving the full length of the road at 25 mph. The speed may be higher or lower, depending on road conditions or local practice. Surface and drainage problems are noted during the inspection. Windshield inspections are performed once each season or four times a year. The results can be used for estimating maintenance needs and setting priorities.

AIRFIELD CONCRETE PAVEMENTS
CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT

BRANCH R1230 SECTION B03 SAMPLE UNIT 005
SURVEYED BY MYS DATE MAR/15/90 NUMBER OF SLABS 20

Distress Types

61. Blow up
62. Corner Break
63. Long/Trans/ Diagonal Crack
64. Durability Crack
65. Joint Seal Damage
66. Patching, 5 sf
67. Patching/Utility Cut
68. Popouts
69. Pumping
70. Scaling/Map Crack/ Crazing
71. Settlement/Fault
72. Shattered Slab
73. Shrinkage Crack
74. Spalling-Joints
75. Spalling-Corner

DIST TYPE	SEV	NO. SLABS	DENSITY %	DEDUCT VALUE
65	H			12
62	L	2	10	7.8
62	M	1	5	8.6
63	L	3	15	11.5
63	M	3	20	24.0
72	L	1	5	11.0
74	L	2	10	3.5
75	L	3	15	5.5
75	M	1	5	3.6

SKETCH:

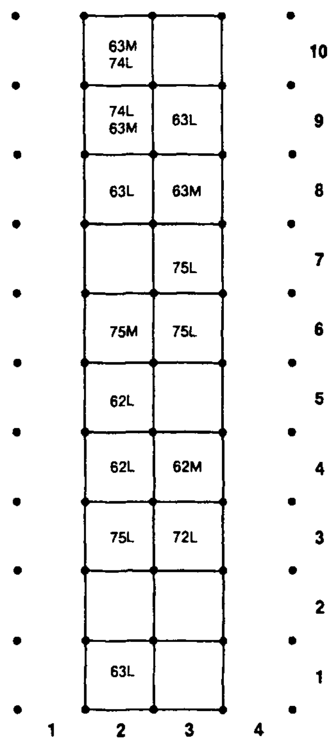


Figure 3-13. Example Airfield PCC Sample Unit Condition Survey Sheet.

The detailed distress inspection may be conducted less frequently but should be done at least once every 3 years. The inspection should be performed at the time of year when the roads are in their best and most consistent condition (Eaton, et al. 1988). Seven distresses have been defined for unsurfaced road: improper cross section, inadequate roadside drainage, corrugations, dust, potholes, ruts, and loose aggregate. Figure 3–14 shows an example of a completed field condition survey sheets. Appendix F provides the distress definitions and procedures for measuring them.

UNSURFACED ROADS
CONDITION SURVEY DATA SHEET
FOR SAMPLE UNIT

SKETCH: 100' 16.5' N

BRANCH CENGRO2 **SECTION** AGO3 **SAMPLE UNIT** 2
SURVEYED BY KTL **DATE** Jul 13 '92 **SAMPLE AREA** 1650 ft²

1. Improper Cross Section (Linear feet)
2. Inadequate Roadside Drainage (Linear feet)
3. Corrugations (Square feet)
4. Dust
5. Potholes (Number)
6. Ruts (Square feet)
7. Loose Aggregate (Linear feet)

DISTRESS SEVERITY	QUANTITY									TOTAL	DENSITY %	DEDUCT VALUE
1M	12	18								30	1.8	6
3L	16	20								36	2.2	2
3M	18									18	1.1	2
5L	9	12	7							28	1.6	23
5M	2	6								8	0.48	16
6L	180									180	14.5	18

Figure 3-14. Example Unsurfaced Road Sample Unit Condition Survey Sheet.

3.4.4 Performing Inspection Using Pen Tablet Computers

The PCI inspection can be expedited by using pen-based computers that receive data through a pen-shaped instrument or a keyboard. This eliminates the tedious, error prone process of manual data reduction and entry into the pavement management system (PMS).

The most direct and convenient method is to use a pen tablet computer with a full Windows (Microsoft) operating system. These computers allow for loading the entire pavement management system and therefore the inspector can enter data in the field, similar to being in the office except with the use of the pen instrument. Figure 3–15 is an example pen tablet computer with full operating system. A key requirement of such a device is that it has to be outdoor viewable using technology such as a reflective LCD screen. Figure 3–16 shows an example PCI distress data entry screen when the inspection is performed using the Micro PAVER management system.

A second option is to use pen tablets with Windows CE operating system rather than a full operating system. The Windows CE devices are normally smaller, lighter, and have a longer battery operating life than the full version. Also, the inspector has to download pavement data for the sections to be inspected which include previous inspection data for these sections. This information then becomes available for viewing as needed in the field. After the inspection is complete, a file is created in the pen tablet and uploaded into the PMS.

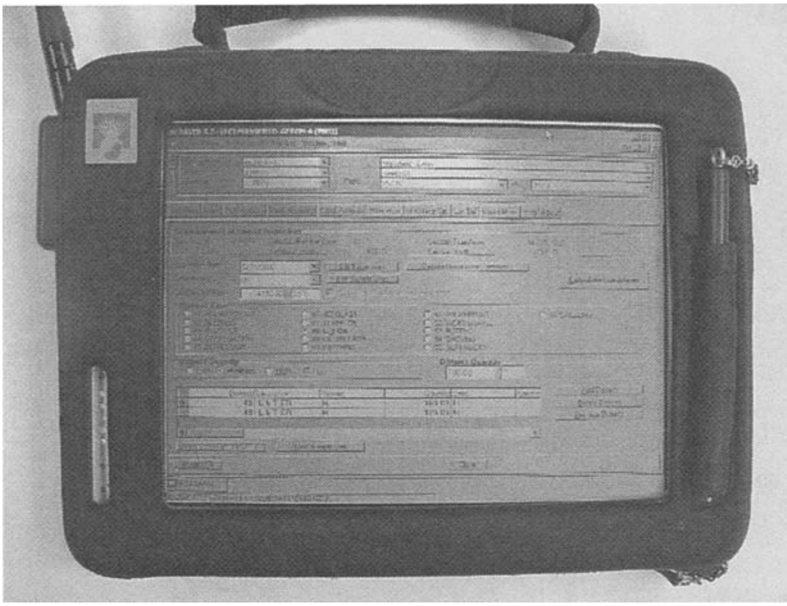


Figure 3-15. Example Pen Tablet Computer with Full Operating System (Ohio DOT 2004).

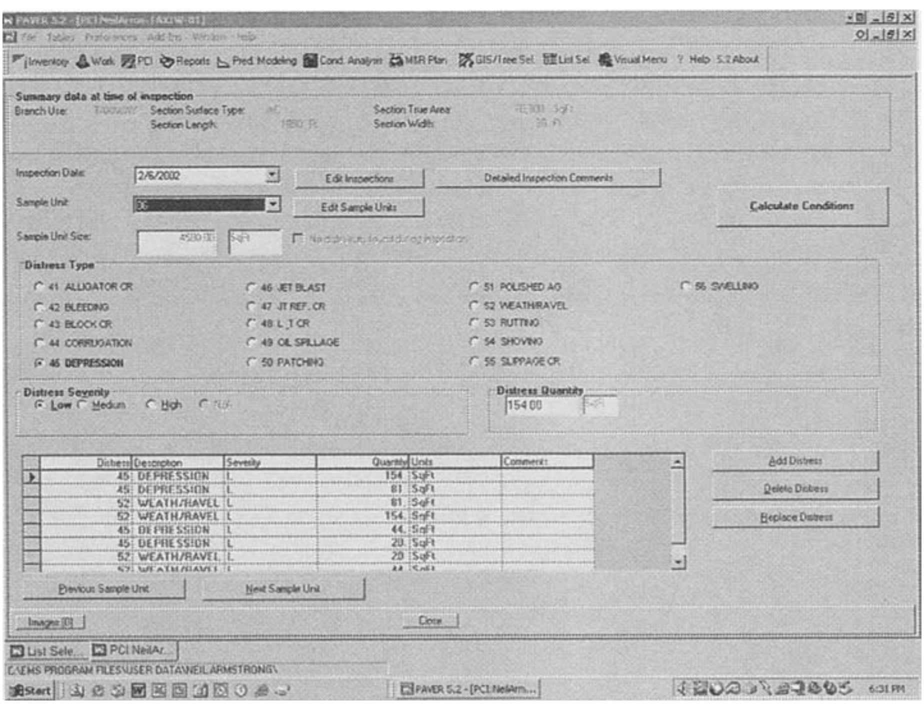


Figure 3-16. Example PCI Distress Data Entry Screen.

3.5 Calculating the PCI

The PCI is calculated for each inspected sample unit. The PCI cannot be computed for the entire pavement section without computing the PCI for the sample units first. The PCI calculation is based on the deduct values—weighing factors from 0 to 100 that indicate the impact each distress has on pavement condition. A deduct value of 0 indicates that a distress has no effect on pavement structural integrity and/or surface operational condition, whereas a value of 100 indicates an extremely serious distress.

3.5.1 Calculation of a Sample Unit PCI for Asphalt Surfaced Pavements and Unsurfaced Roads

The calculation steps are similar for roads and airfields. They are summarized in Figure 3–17. Following is a description of each step.

Step 1: Determine deduct values.

1a. Add the totals for each distress type at each severity level and record them under "Total" on the survey form. For example, Figure 3–12 shows two entries for distress type 48M. The distress is added and entered under "Total" as 16. Quantities of distress are measured in square feet (square meters), linear feet (meters), or number of occurrences, depending on the distress type.

1b. Divide the quantity of each distress type at each severity level by the total area of the sample unit, and then multiply by 100 to obtain the percentage of density per sample unit for each distress type and severity.

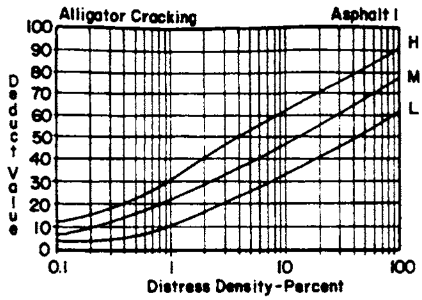
1c. Determine the deduct value for each distress type and severity level combination from the distress deduct value curves. Figure 3–18 shows an example of a deduct curve for distress type 41, "Alligator Cracking," for airfield pavements. Deduct curves for all distresses are provided in Appendix B (for asphalt roads), Appendix D (for asphalt airfield), and Appendix F (for unsurfaced roads).

Step 2: Determine the maximum allowable number of deducts (m).

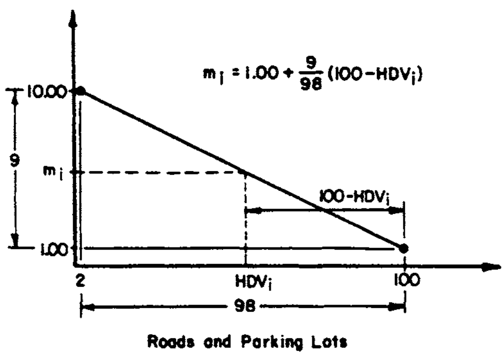
2a. If only one individual deduct value (or none) is > 5 for airfields and unsurfaced roads, or > 2 for surfaced roads, the total deduct value is used in place of the maximum corrected deduct value (CDV) in Step 4 and the PCI computation is completed; otherwise, the following steps should be followed.

2b. List the individual deduct values in descending order. For example, the values in Figure 3–12 would be sorted as follows: 21, 20.1, 17.1, 6.7, 4.8, and 1.6.

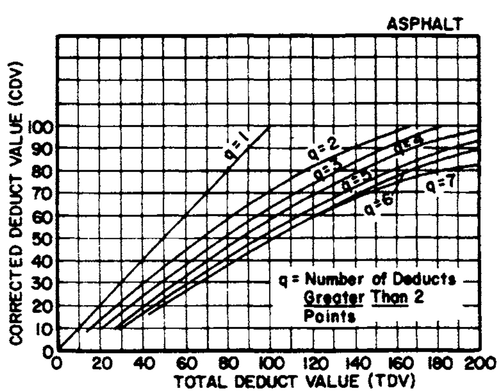
Step 1. Determine Deduct Values



Step 2. Determine Maximum Allowable Number of Deducts (m)



Step 3. Determine Maximum Corrected Deduct Value



Corrected deduct value curves for asphalt-surfaced pavements

Step 4. Calculate PCI

PCI = 100 – Maximum CDV

Figure 3-17. PCI Calculation Steps for a Sample Unit.

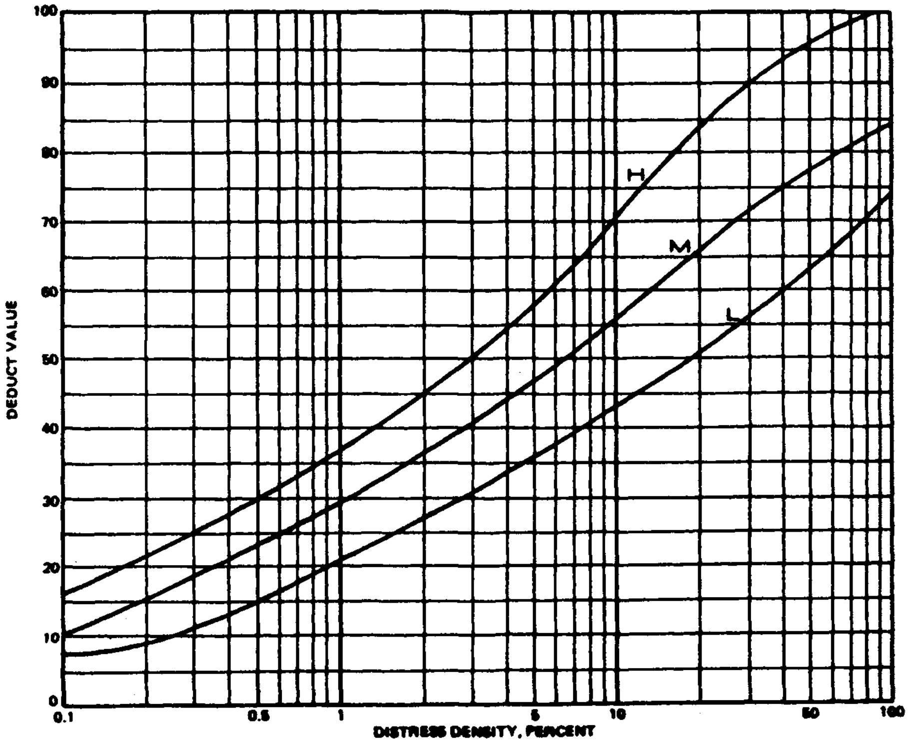


Figure 3-18. AC Pavement Deduct Curve for the Alligator Cracking Distress.

2c. Determine the allowable number of deducts, m (Fig. 3–19), using the following formulas:

$$m_i = 1 + (9/95)(100 - HDV_i) \quad \text{(for airfields and unsurfaced roads)} \tag{3-2}$$

$$m_i = 1 + (9/98)(100 - HDV_i) \quad \text{(for surfaced roads)} \tag{3-3}$$

where:

m_i = allowable number of deducts, including fractions, for sample unit *i*.

HDV_i = highest individual deduct value for sample unit *i*.

For the example, in Figure 3–12:

$$m = 1 + (9/95)(100 - 21.0) = 8.48$$

2d. The number of individual deduct values is reduced to m, including the fractional part. If fewer than m deduct values are available, then all of the deduct values are used. For the example in Figure 3–12, all the deducts are used since they are less than m.

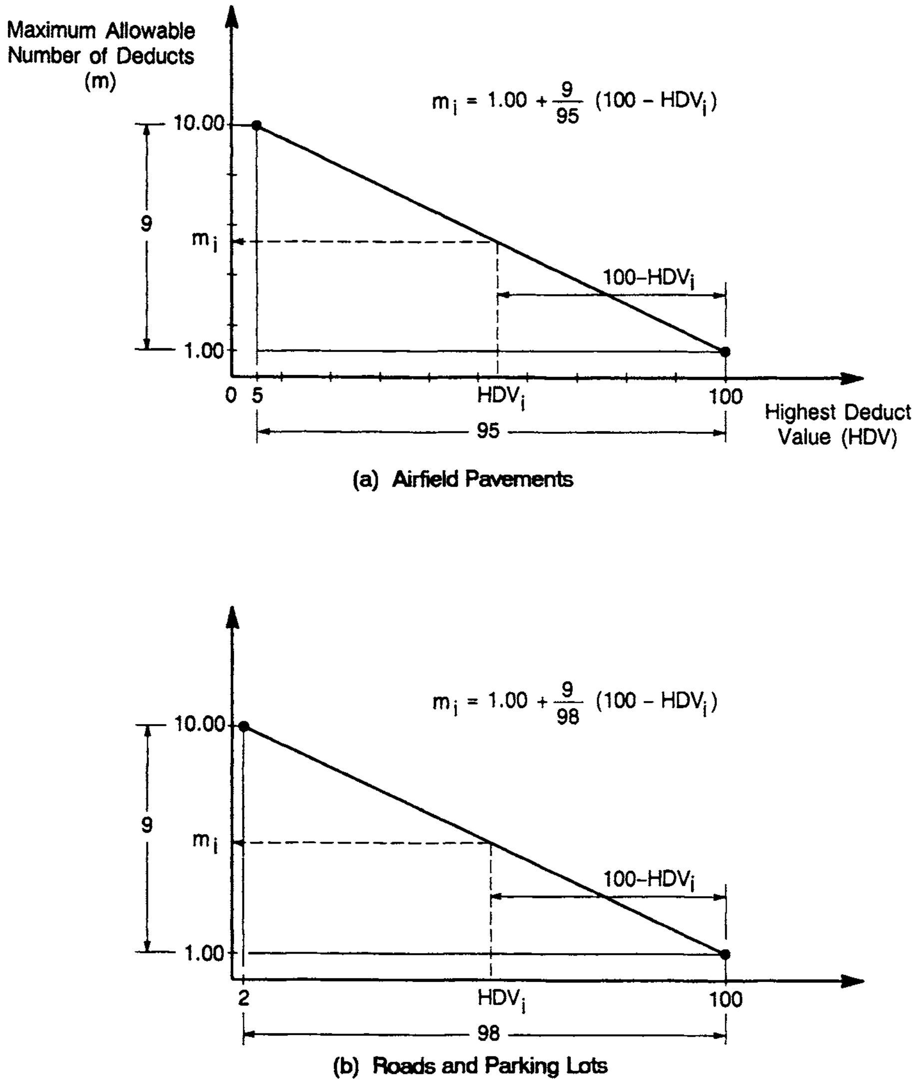


Figure 3-19. Determination of Maximum Allowable Deducts (m).

Step 3: Determine the maximum corrected deduct value (CDV). The maximum CDV is determined iteratively as follows:

3a. Determine the number of deducts with a value > 5.0 for airfields and unsurfaced roads, and > 2 for surfaced roads. For the example in Figure 3–12, q=4.

3b. Determine total deduct value by adding all individual deduct values. In the current example, the total deduct value is 71.3.

3c. Determine the CDV from q and total deduct value by looking up the appropriate correction curve. Figure 3–20 shows the correction curve for asphalt-surfaced airfield pavements. Other correction curves are provided at the end of the individual deduct curves in Appendices B through F.

3d. For airfields and unsurfaced roads, reduce to 5.0 the smallest individual deduct value that is > 5.0. For surfaced roads, reduce to 2.0 the smallest individual deduct value that is > 2.0. Repeat Steps 3a through 3c until q is equal to 1.

3e. The maximum CDV is the largest of the CDVs determined.

Step 4: Calculate PCI by subtracting the maximum CDV from 100.

Figure 3–21 summarizes the PCI calculation for the example of AC pavement data shown in Figure 3–12. A blank PCI calculation form is included in Appendix A.

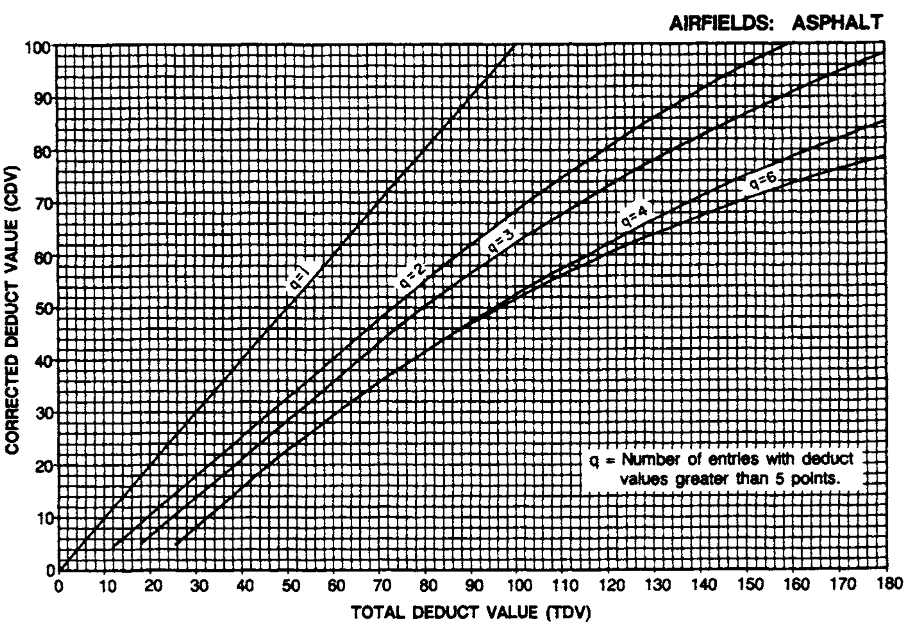


Figure 3-20. Correction Curves for AC Surfaced Airfield Pavements.

AC m = 8.48 > 6

#	Deduct Values										Total	q	CDV
1	21	20.1	17.1	6.7	4.8	.1.6					71.3	4	37
2	21	20.1	17.1	5.0	4.8	.1.6					69.6	3	43
3	21	20.1	5.0	5.0	4.8	.1.6					57.5	2	38
4	21	5.0	5.0	5.0	4.8	.1.6					42.4	1	42.4
5													
6													
7													
8													
9													
10													

PCI = 100 - 43 = 57

Figure 3-21. PCI Calculation Sheet for Example Sample Unit Shown in Figure 3-12.

3.5.2 Calculation of a Sample Unit PCI for Concrete Surfaced Pavements

Step 1: Determine deduct values.

1a. For each unique combination of distress type and severity level, add up the number of slabs in which they occur. For example, in Figure 3-13 there are two slabs with two low-severity corner breaks.

1b. Divide the number of slabs from 1a above by the total number of slabs in the sample unit, then multiply by 100 to obtain the percentage of density per sample unit for each distress type and severity combination.

1c. Determine the deduct values for each distress type and severity level combination using the appropriate deduct curve in Appendix C (for roads and parking lots) or Appendix E (for airfields).

Step 2: Determine maximum allowable number of deducts (m).

This step is the same as for asphalt surfaced pavements outlined in Section 3.5.1 (step 2c). For the example in Figure 3–13, based on a highest deduct value (HDV) of 24, m is calculated as $m = 1.0 + 9/95(100 - 24) = 8.2$. There are nine deducts; the ninth smallest deduct (=3.5) is multiplied by 0.2 and reduced to 0.7.

Step 3: Determine the maximum CDV.

Determine the maximum CDV by following the procedures in Section 3.5.1 (step 3), but using the appropriate correction curve at the end of Appendix C (for concrete roads) or Appendix E (for concrete airfields).

Step 4: Calculate the PCI by subtracting maximum CDV from 100.

Figure 3–22 summarizes the PCI calculation for the example of PCC pavement data given in Figure 3–13.

*** 0.7 = (0.2) x 9 th Deduct Value**
= 0.2 x 3.5

PCC m = 8.20 < 9

#	Deduct Values										Total	q	CDV
1	24	12	11.5	11.0	8.6	7.8	5.5	3.6	0.7*		84.7	7	54
2	24	12	11.5	11.0	8.6	7.8	5.0	3.6	0.7		84.2	6	56
3	24	12	11.5	11.0	8.6	5.0	5.0	3.6	0.7		81.4	5	56.5
4	24	12	11.5	11.0	5.0	5.0	5.0	3.6	0.7		77.8	4	54.5
5	24	12	11.5	5.0	5.0	5.0	5.0	3.6	0.7		71.8	3	54
6	24	12	5.0	5.0	5.0	5.0	5.0	3.6	0.7		65.3	2	54.5
7	24	5.0	5.0	5.0	5.0	5.0	5.0	3.6	0.7		58.3	1	58.3
8													
9													
10													

PCI = 100 - 58 = 42

Figure 3-22. PCI Calculation Sheet for Example Sample Unit Shown in Figure 3-13.

3.5.3 Calculation of the PCI for a Section

If all surveyed sample units are selected either by using the systematic random technique or on the basis of being representative of the section, and are equal in size, the PCI of the section is determined by averaging the PCIs of the sample units inspected. If the inspected sample units were not equal in size, area weighted averaging should be used as shown in the following equation:

$$PCI_s = PCI_r = \frac{\sum_{i=1}^{R} PCI_{ri} \times A_{ri}}{\sum_{i=1}^{R} A_{ri}} \tag{3-4}$$

where

PCI_s = PCI of pavement section

PCI_r = area weighted average PCI of random (or representative) sample units

PCI_{ri} = PCI of random sample unit number i

A_{ri} = area of the random sample unit *i*

R = total number of inspected random sample units

If additional sample units are inspected (Section 3.3.3.3), in addition to the random or representative units, the section PCI is computed as follows:

$$PCI_a = \frac{\sum_{i=1}^{A} (PCI_{ai} \times A_{ai})}{\sum_{i=1}^{A} A_{ai}} \tag{3-5}$$

$$PCI_s = \frac{PCI_r\left(A_s - \sum_{i=1}^{A} A_{ai}\right) + PCI_a \times \sum_{i=1}^{A} A_{ai}}{A_s} \tag{3-6}$$

where

PCI_a = area weighted average PCI of additional sample units

PCI_{ai} = PCI of additional sample unit number *i*

A_{ai} = area of additional sample unit *i*

A_s = total section area

For example, if in a section of 60,000 sq ft, five random sample units were inspected and determined to have PCIs of 56 (5,000 sq ft), 72(5,000 sq ft), 65(5,000 sq ft) 69(4,000 sq ft) and 61(4,000 sq ft), and two additional sample units with PCIs of 42(3,500 sq ft) and 39(3,000 sq ft) were included, the PCI of the section would be:

$$PCI_r = \frac{(56 \times 5{,}000) + (72 \times 5{,}000) + (65 \times 5{,}000) + (69 \times 4{,}000) + (61 \times 4{,}000)}{5{,}000 + 5{,}000 + 5{,}000 + 4{,}000 + 4{,}000}$$

$$PCI_r = 64.6$$

$$PCI_a = \frac{(42 \times 3{,}500) + (39 \times 3{,}000)}{3{,}500 + 3{,}000}$$

$$PCI_a = 40.6$$

$$PCI_s = \frac{64.6(60{,}000 - 6{,}500) + 40.6 \times 6{,}500}{60{,}000}$$

$$PCI_s = 62$$

3.5.4 Extrapolating Distress Quantities for a Pavement Section

When a pavement has been inspected by sampling, it is necessary to extrapolate the quantities and densities of distress over the entire pavement section to determine total quantities for the section. If all sample units surveyed were selected at random, the extrapolated quantity of a given distress at a given severity level would be determined as shown in the following example for an asphalt surfaced road with medium-severity alligator cracking:

Surface Type: Asphalt concrete

Section Area: 24,500 sq ft

Total Number of sample units in section: 10

Five sample units were surveyed at random, and the amount of medium-severity alligator cracking was determined as follows:

Sample Unit ID Number	Sample Unit Area (ft^2)	Medium-Severity Alligator Cracking (ft^2)
02	2,500	100
04	2,500	200
06	2,500	150
08	2,500	50
10	2,000	100
Total Random	12,000	600

The average density for medium-severity alligator cracking, then, is 600 divided by 12,000 or 0.05. The extrapolated quantity is determined by multiplying density by section area (i.e., 0.05 x 24,500 = 1,225 sq ft).

If additional sample units were included in the survey, the extrapolation process would be slightly different. In the example given above, assume that sample unit number 01 was surveyed as an additional unit, and that the amount of medium-severity alligator cracking was measured as follows:

Additional Sample Unit ID	Sample Unit Area (ft^2)	Medium-Severity Alligator Cracking (ft^2)
01	2,500	1,000
Total Additional	2,500	1,000

Since 2,500 sq ft were surveyed as additional in this example, the section's randomly represented area is 24,500 - 2500 sq ft, or 22,000 sq ft. The extrapolated distress quantity is obtained by multiplying the distress density by the section's randomly represented area, then adding the amount of additional distress. In this example, the extrapolated distress quantity equals (0.05 x 22,000) + 1,000, or 2,100 sq ft.

3.5.5 Distress Classification by Cause

Examination of the pavement section extrapolated distress types, severities, and quantities provides valuable information used to determine the cause of pavement deterioration and eventually its maintenance and repair (M&R) needs. Figure 3–23, and 3–24 classify distress causation for paved roads and airfield pavements, respectively, based on load, climate, and other factors. Quantification of the relative effect of each can be determined on the corresponding deduct value for the extrapolated section distresses as illustrated in the following example:

> The following distresses were measured on an asphalt airfield pavement section and the deduct values for the extrapolated distresses were determined from the deduct curves in Appendix D.

Distress Type	Severity	Density, %	Deduct Value
Alligator Cracking	Medium	6.4	50
Transverse Cracking	Low	2.0	8
Rutting	Low	2.7	20

Code	Distress	Cause
Asphalt-Surfaced Roads and Parking Areas		
01	Alligator cracking	Load
02	Bleeding	Other
03	Block cracking	Climate
04	Bumps and sags	Other
05	Corrugation	Other
06	Depression	Other
07	Edge cracking	Load
08	Joint reflection	Climate
09	Lane/shoulder drop-off	Other
10	Longitudinal and transverse cracking	Climate
11	Patching and utility cut patching	Other
12	Polished aggregate	Other
13	Potholes	Load
14	Railroad crossings	Other
15	Rutting	Load
16	Shoving	Load
17	Slippage cracking	Other
18	Swell	Other
19	Weathering and ravelling	Climate
Portland Cement Concrete Roads and Parking Areas		
21	Blowup/buckling	Climate
22	Corner break	Load
23	Divided slab	Load
24	Durability ("D") cracking	Climate
25	Faulting	Other
26	Joint seal damage	Climate
27	Lane/shoulder drop-off	Other
28	Linear cracking	Load
29	Patching, large	Other
30	Patching, small	Other
31	Polished aggregate	Other
32	Popouts	Other
33	Pumping	Other
34	Punchout	Load
35	Railroad crossing	Other
36	Scaling/map cracking/crazing	Other
37	Shrinkage cracks	Climate
38	Spalling, corner	Climate
39	Spalling, joint	Climate

Figure 3-23. PAVER Distress Classification for Roads and Parking Lots.

Code	Distress	Cause
Asphalt-Surfaced Airfields		
41	Alligator cracking	Load
42	Bleeding	Other
43	Block cracking	Climate
44	Corrugation	Other
45	Depression	Other
46	Jet blast	Other
47	Joint reflection/cracking	Climate
48	Longitudinal and transverse cracking	Climate
49	Oil spillage	Other
50	Patching	Other
51	Polished aggregate	Other
52	Weathering/ravelling	Climate
53	Rutting	Load
54	Shoving	Other
55	Slippage cracking	Other
56	Swelling	Other
Portland Cement Concrete Airfields		
61	Blowup	Climate
62	Corner break	Load
63	Linear cracking	Load
64	Durability cracking	Climate
65	Joint seal damage	Climate
66	Small patch	Other
67	Large patch/utility cut	Other
68	Popouts	Other
69	Pumping	Other
70	Scaling/crazing	Other
71	Faulting	Other
72	Shattered slab	Load
73	Shrinkage cracking	Other
74	Joint spalling	Other
75	Corner spalling	Other

Figure 3-24. PAVER Distress Classification for Airfields.

The total deduct value attributable to load is 70 (50 + 20), and that attributable to climate is 8. There is no distress classified as "other." The percentage of deducts attributable to load, climate, and other causes is computed as follows:

$$Load = 70/78 \times 100 = 90\,percent$$

$$Climate = 8/78 \times 100 = 10\,percent$$

The percentage of deduct values attributed to each cause is an indication of the cause(s) of pavement deterioration. In this example, distresses caused primarily by load have resulted in 90 percent of the total deducts, whereas all other causes have produced only 10 percent. Thus, traffic load is by far the major cause of deterioration for this pavement section.

3.5.6 Calculation of Other Distress Indices

The PCI calculation procedure presented in this chapter can be used to calculate other specialty distress indices using specific distress types and severities. For example, one can decide to calculate a structural index based on structurally caused distresses only such as alligator cracking, rutting and potholes. Similarly, a roughness index can be calculated based on roughness causing distresses such as bumps, corrugation, depressions, potholes, shoving, and rutting.

Calculating such distress indices could be misleading without full understanding of how the PCI procedure was developed. Also the developed index should be field-calibrated for its intended use.

The PCI was developed to rate the pavement structural integrity and surface operational conditions. The PCI also correlates with M&R needs. The distress deduct values, therefore, reflect the distress type/severity, effect on the pavement structural integrity and/or surface operational conditions. For example, the deduct values for low-severity alligator cracking reflect the effect on structural integrity rather than the surface operational condition (i.e., roughness or skid resistance). Meanwhile, shoving reflects surface operational condition rather than structural integrity. All these factors should be carefully considered in the definition and calibration of other distress indices based on the existing PCI procedure.

The following is an example development of a distress based index for the evaluation of Foreign Object Damage (FOD) potential for aircraft engines.

3.5.6.1 Development of FOD Potential Index

The development of the FOD index (USAF-ETL, 2004) is summarized in the following steps:

Step1: Formulate an expert panel that is knowledgeable about FOD potential to aircraft engines and the process for developing a distress based index.

Step2: Identify distresses/ severity levels that are capable of producing FOD potential. The distresses identified are shown in bold in Figures 3–25, and 3–26 for asphalt and concrete surfaced pavements respectively.

Step 3: Develop a rating scale. Figure 3–27 shows the rating scale developed by the expert panel. The scale ranges from 0 to 100 with zero being no FOD potential. The decision was made to design the scale such that the index increases with time, i.e., the lower the FOD potential rating the better the pavement.

Step 4: Identify factors affecting the rating. The FOD potential was determined to be dependent on the type of the pavement (i.e. asphalt or concrete surfaced), and the type of aircraft using the pavement. The following three aircrafts were selected for the FOD index development; F-16, KC-135, and C-17. They were selected to represent a range with regard to engine height above the pavement surface, and engine susceptibility to FOD (e.g., engine type, size, air flow, and thrust). Other aircrafts were grouped accordingly as shown in Figure 3–28.

Step 5: Develop correlation between field FOD rating and calculated FOD index based on distress, the process is summarized in Figure 3–29. The expert team visits several airfield pavements and for each pavement performs FOD potential ratings using the rating scale as well as collects existing distress information to be used for the FOD index calculation. Three ratings are performed for each pavement, one for each of the three standard aircrafts. Field visits continue till a satisfactory correlation is obtained. In the process, modifications may have to be made to the selected list of distresses used in the calculation as well as whether specific distress deduct values may be multiplied by an adjustment factor. For the FOD index calculation, it was determined that a multiplier of 0.6 should be applied for alligator cracking and a multiplier of 4.0 should be applied for joint seal damage. Figures 3–30, and 3–31 show the final correlation curves developed for asphalt surfaced and concrete surfaced pavement respectively.

Step 6: Define FOD index limits. Using the preestablished FOD potential rating limits (see rating scale, Figure 3–27), and the developed correlation curves, FOD index limits are established. For example, by entering a FOD rating of 45 on the vertical axis in Figure 3–30, a corresponding FOD index of 32 is obtained for F-16. Figure 3–32 shows the established FOD index limits.

The Micro PAVER program calculates the FOD index based on the distress information already available from the PCI survey. Figure 3–33 shows a FOD potential rating for a civil aviation airport.

Distress Type	Severity Levels (L = Low, M = Medium, H = High)
Alligator Cracking (modification factor: 0.6)	**L, M, H**
Bleeding	n/a
Block Cracking	**L, M, H**
Corrugation	L, M, H
Depression	L, M, H
Jet Blast Erosion	**n/a**
Joint Reflection Cracking	**L, M, H**
Longitudinal and Transverse Cracking	**L, M, H**
Oil Spillage	**n/a**
Patching	**L, M, H**
Polished Aggregate	n/a
Raveling and Weathering	**L, M, H**
Rutting	L, M, H
Shoving	**L, M, H**
Slippage Cracking	**n/a**
Swelling	L, M, H

Figure 3-25. Distress List for ACC Pavements.

Distress Type	Severity Levels (L = Low, M = Medium, H = High)
Blow Up	**L, M, H**
Corner Break	**L, M, H**
Durability Cracking	**L, M, H**
Linear Cracking	**L, M, H**
Joint Seal Damage (modification factor: 4.0)	**L, M, H**
Small Patching	**L, M, H**
Large Patching	**L, M, H**
Popouts	**n/a**
Pumping	**n/a**
Scaling	**L, M, H**
Settlement	L, M, H
Shattered Slab	**L, M, H**
Shrinkage Cracking	n/a
Joint Spalling	**L, M, H**
Corner Spalling	**L, M, H**

Figure 3-26. Distress List for PCC Pavements.

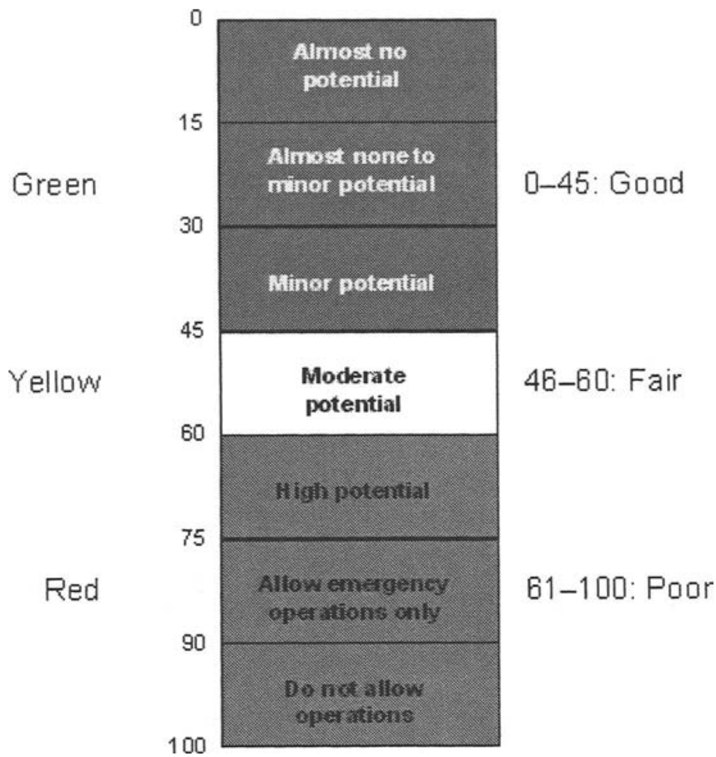


Figure 3-27. FOD Potential Rating Scale.

Standard Aircraft	Use FOD Index/FOD Potential Rating Relationship Curve for Standard Aircraft for Aircraft Listed Below in the Same Row
F-16	A-37, F-4, F-15, F-22, F-117, C-38, T-37, T-38, U-2
KC-135	A-300, A-310, AN-124, B-1, B-2, B-52, B-707, B-737, B-747, B-757, B-767, C-21, C-32, C-38, C-40, C-135, C-141, DC-8, DC-10, E-3, E-4, E-8, EC-18, EC-135, IL-76, KC-10, L-1011, T-1A, T-43, VC-25, VC-137
C-17	A-10, B-727, C-5, C-9, C-12*, C-20, C-22, C-23*, C-130*, DC-9, OV-10*, T-6*, V-22*

Figure 3-28. Recommended FOD Curve Applicability for Various Aircraft.

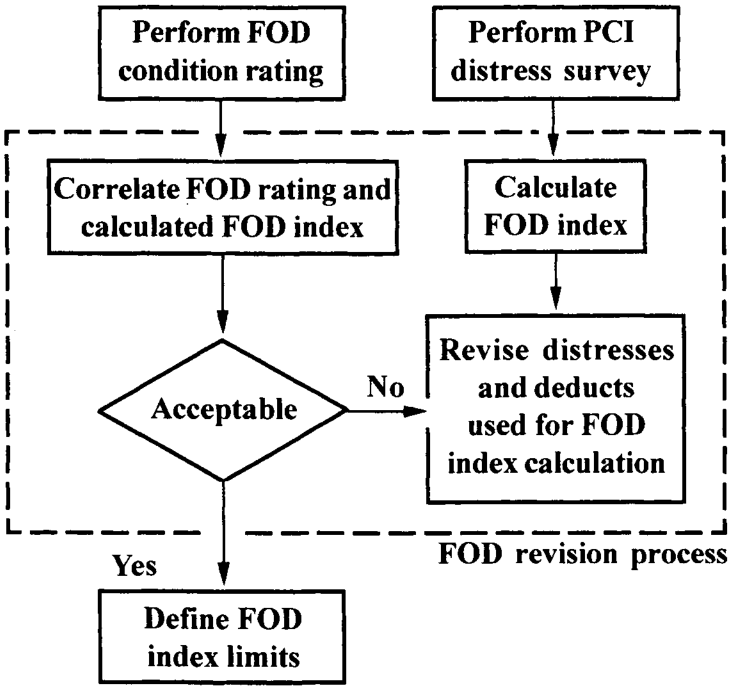


Figure 3-29. Development Process.

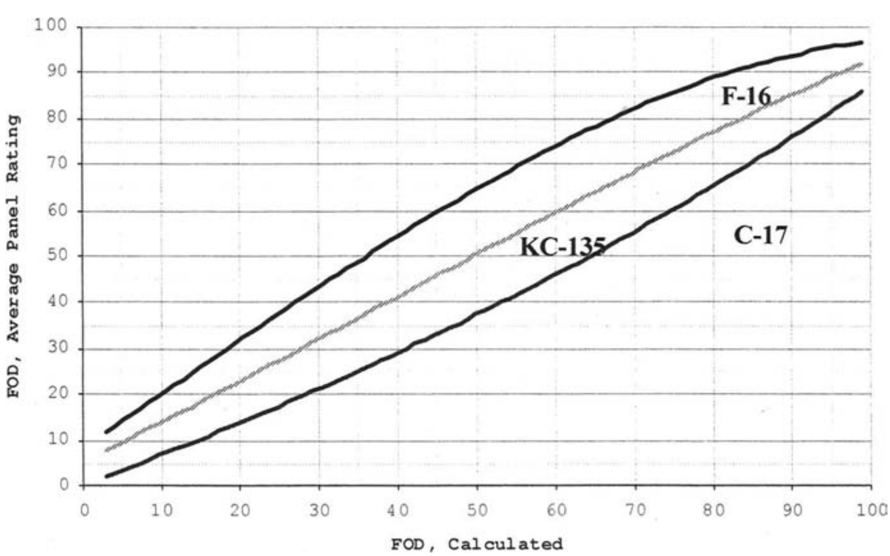


Figure 3-30. Relationship Between FOD Index and FOD Potential Rating for Asphalt Pavements.

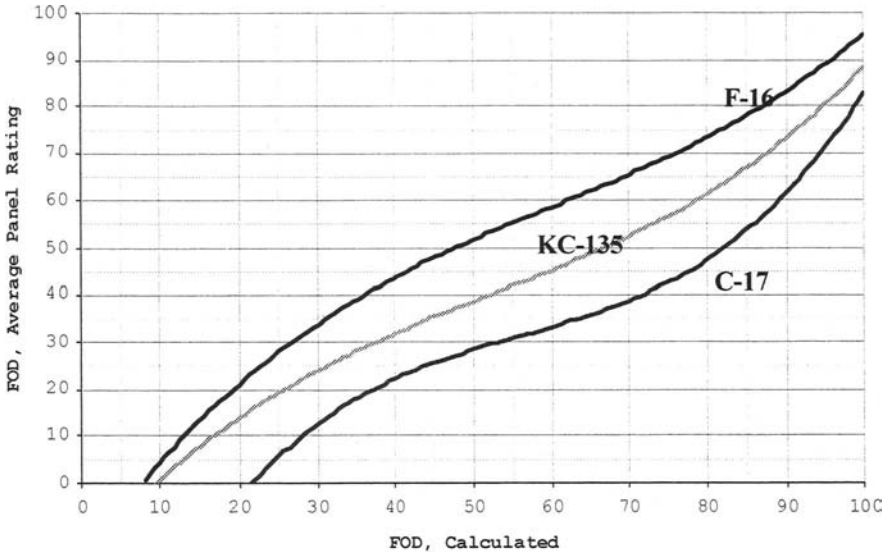


Figure 3-31. Relationship Between FOD Index and FOD Potential Rating for Concrete Pavements.

FOD Potential Rating	FOD Index					
	F-16		KC-135		C-17	
	ACC	PCC	ACC	PCC	ACC	PCC
Good: 0–45	0–32	0–41	0–44	0–60	0–59	0–77
Fair: 46–60	33–45	42–62	45–60	61–78	60–75	78–89
Poor: 61–100	46–100	63–100	61–100	79–100	76–100	90–100

Figure 3-32. FOD Index Limits.

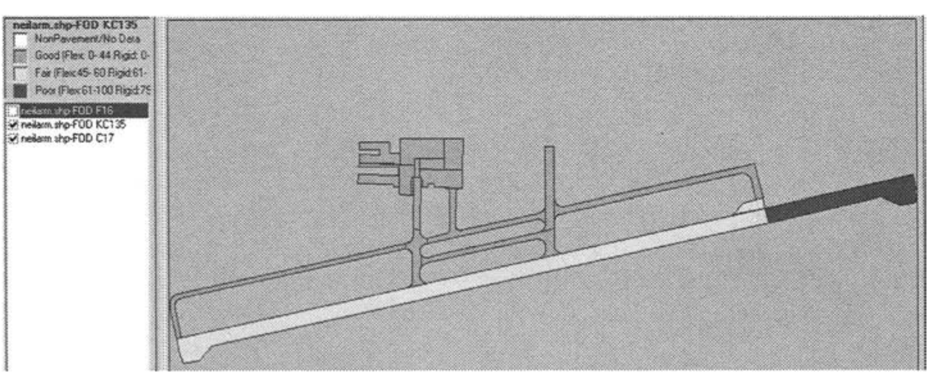


Figure 3-33. Example FOD Potential Rating

3.6 Automated Distress Data Collection

There are currently several types of technologies for use in distress data collection. These technologies include: 35-mm analog continuous film, digital camera, and digital line scan imaging. The technologies are usually vehicle-mounted along with profiling equipment for determining roughness, rutting, and faulting.

Frames are used for automated distress data collection in place of samples. Figure 3–34 illustrates the use of frames in the distress survey of asphalt-surfaced roads. Frames are normally one lane wide when roadways are being surveyed. The collected data is viewed on a workstation in the office and the distresses are manually interpreted by trained personnel. Research efforts are currently underway to automatically interpret distress types and severities. When one is interpreting distresses, it is recommended that the frame length be a minimum of 20 ft long to ensure accurate distress identification. The following paragraphs provide a brief description of the currently used three technologies for capturing pavement images.

3.6.1 35-mm Analog Continuous Film

The pavement surface images are collected using continuous strip photography, (Cline et al, 2001). The images cover a width of 16 ft (4.9 m) and cracks as fine as 0.04 in. (1 mm) in width can be recorded at speeds up to 60 mph (100 km/hr).

This system consists of a boom-mounted, 35 mm slit camera, electronic controller, and custom illumination system. The data collection vehicle is shown in Figure 3–35. The electronic controller synchronizes the film speed to the speed of the vehicle so that there is no loss in resolution with changes in vehicle speed. The illumination system consists of an array of halogen lights mounted in a custom bumper. This system allows control of the angle and degree of illumination on the pavement surface for maximum resolution. The images collected are at a 1:200 scale. These images are analyzed to determine the types, severity, and extent of pavement distress using workstations shown in Figure 3–36.

This system also uses a pulse camera, which photographs the transverse profile of the roadway at selected intervals to determine rut depth and shoulder drop-off. The pulse camera is synchronized with a hairline strobe projector mounted on the rear bumper; it maintains a precision of + 2 mm and is synchronized to the actual vehicle speed.

3.6.2 Digital Camera

The technology uses a digital camera with strobe lighting, (Cline et al, 2001) The imaging system collects full pavement width of 14 ft (4.3 m) at the resolution of 1,300 x 1,024 pixel images. Images are generally collected at night using strobe lighting. Data collection ranges from 1 to 60 mph (1 to 100 km/hr) and all images are stored in an onboard computer system. Images are then stitched together (computer software package) to form a continuous image of the pavement surface. Image location is closely controlled through a highly accurate distance measuring instrument (DMI) and global positioning. The data collection vehicle is shown in Figure 3–37.

Data processing is completed in the office using the continuous image viewer as illustrated in Figure 3–38. The pavement images can be scrolled forward or backward at varying speeds. A scaled grid is laid over the image to assist in quantifying the distresses.

3.6.3 Digital Line Scan Imaging

The technology uses state-of-the-art digital imaging to collect continuous, high-resolution images of the pavement surface, (Cline et al, 2001). The digital imaging system consists of a 2,000-pixel digital line scan camera, illumination system, and computerized controller. The line scan camera is set to cover a width of 14.5 ft (4.4 m). Data collection speed ranges from 1 to 60 mph (1 to 100 km/hr) and all images are stored in an onboard computer system. Images are generally collected at night using an illumination system, which consists of an array of halogen lights mounted in a custom bumper. The data collection vehicle is shown in Figure 3–39. Data processing is completed in the office. Images are analyzed to determine the types, severity, and extent of pavement distress at the workstations shown in Figure 3–40.

This system also uses a pulse camera, which photographs the transverse profile of the roadway at selected intervals to determine rut depth and shoulder drop-off. The pulse camera is synchronized with a hairline strobe projector mounted on the rear bumper; it maintains a precision of + 2 mm and is synchronized to the actual vehicle speed.

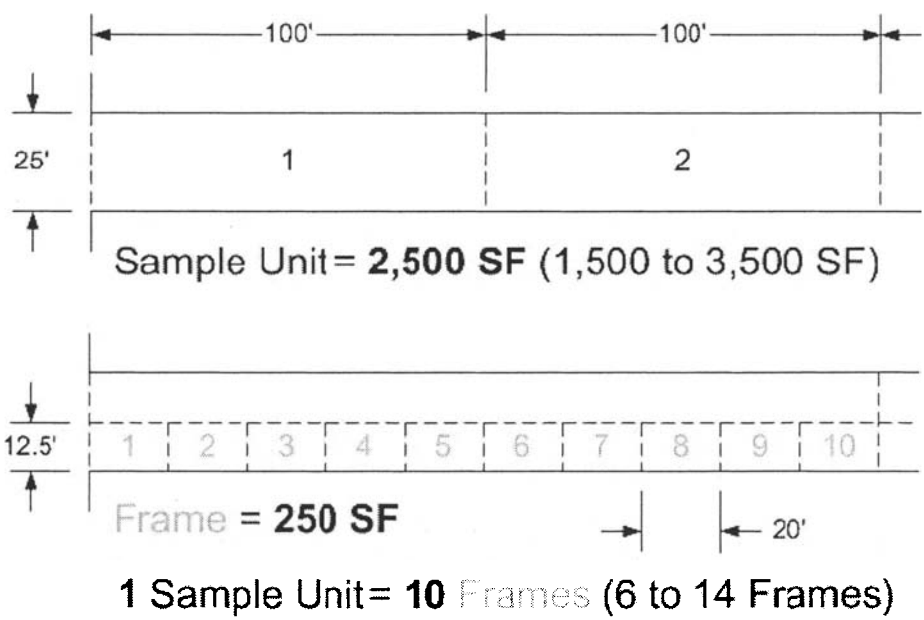


Figure 3-34. Asphalt Surfaced Roads Survey Using Frames.

Figure 3-35. 35-mm Analog Continuous Film Data Collection Vehicle (Cline et al., 2001)

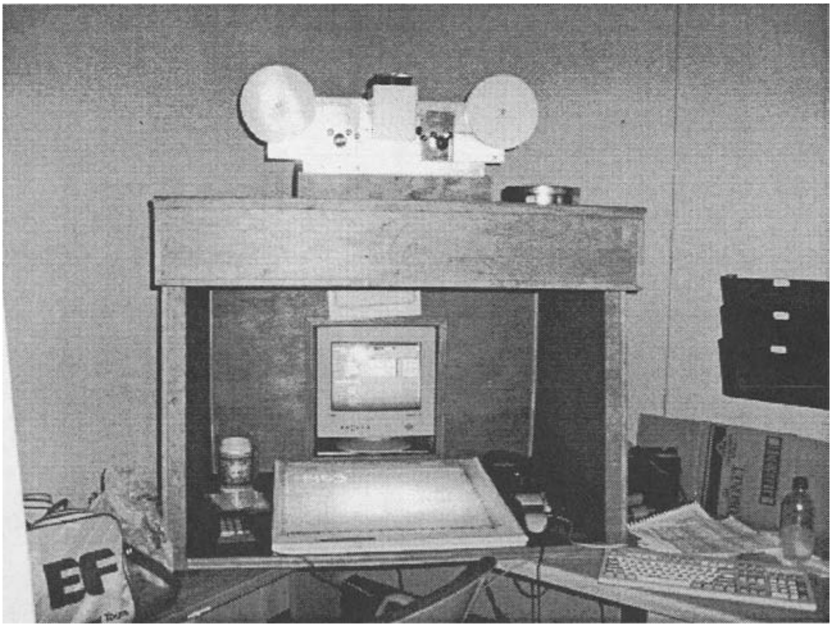


Figure 3-36. Workstation Used to Analyze 35-mm Film Images (Cline et al., 2001)

Figure 3-37. Digital Camera Data Collection Vehicle (Cline et al., 2001)

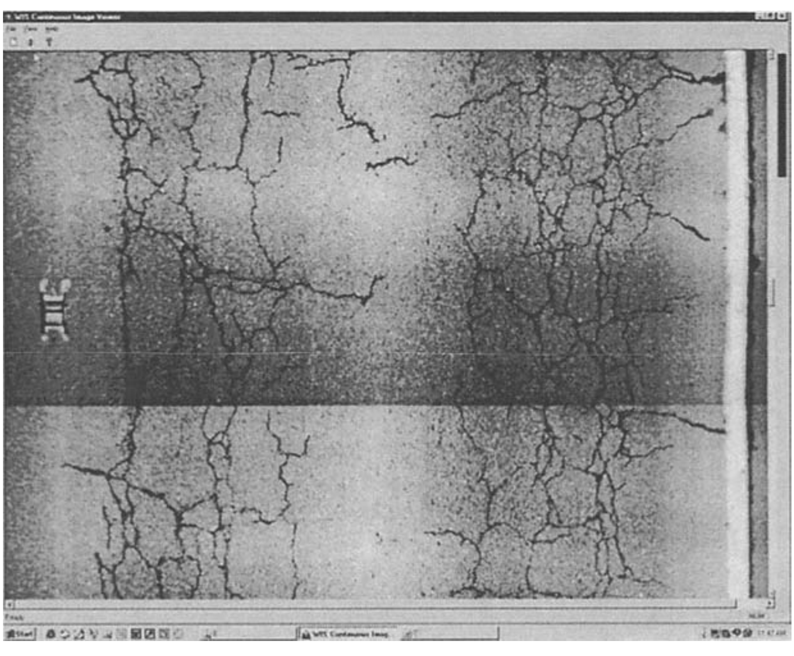


Figure 3-38. Image Viewer for Data Processing of Digital Camera Data (Cline et al., 2001)

3.7 Comparison of Manual and Automated Distress Data Collection Results

This section presents the results of a study sponsored by the U.S. Navy (Cline et al,2001) to compare the results of distress surveys conducted both manually and by using automated equipment. The study concluded that distress measurements taken manually or captured from automated images are consistent. The study also concluded that the PCI calculated from either procedure is consistent.

3.7.1 Comparison When Manual Distress Survey is Performed Using Frames

Several sections were marked so comparisons could be made by using the exact locations by both manual and automated techniques. These sections were marked so they would be compatible with the automated data collection procedure. The pavements were marked at 0 ft, 20 ft, 40 ft etc. Data collected in these sections were used to compare distresses measured by each technique and determine PCI by use of "frames" using both techniques. The manual PCI was conducted by measuring distresses in each of the marked "frames" and combining those to make one sample for each 2,000 to 2,500 sq ft (186 to 233 m^2) of area surveyed.

Results indicate that in general, distress type and quantity are consistent between techniques and the severity is somewhat inconsistent. However, severity appears to be

Figure 3-39. Data Collection Vehicle for Digital Line Scan Imaging (Cline et al. 2001)

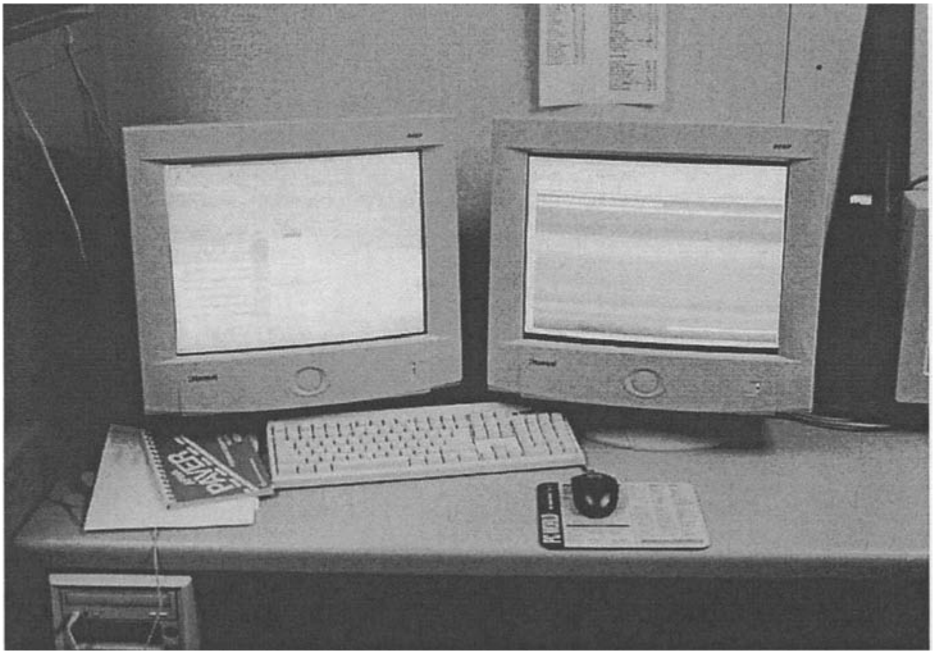


Figure 3-40. Data Processing Workstation for Digital Line Scan Images (Cline et al., 2001)

typically lower by the automated system. Consistent distress type and quantity between techniques indicate that both field procedures produce similar results. The inconsistency of the severity indicates similar concerns that are present during any survey, which is interpretation of the level of severity from person to person. Reviewing the pavement at a workstation where distress can be magnified may produce more accurate measurements and better consistency from person to person.

The following PCI values indicate both survey techniques, using the same pavement areas to determine PCI, result in similar values. The results on SINGLETON 06 indicate the inconsistency of severity, as discussed above, and when the severity is consistent, the PCI is consistent.

	Manual PCI	Automated PCI
TARAWA 01	83	81
TARAWA 02	38	39
TARAWA 03	67	72
WASP 04	84	85
SINGLETON 06	59	79 (61)[1]
TICONDEROGA 01	40 (38)[2]	35

[1]PCI if severity of distresses were M as they were for the manual survey.
[2]PCI when an additional sample unit was added.

3.7.2 Comparison When the Manual Distress Survey Is Performed Using Sample Units

Several additional sections were selected to perform a complete manual survey using ASTM standards. These sections were surveyed manually using samples and were not disclosed to the contractor as to which were conducted manually.

The following PCI values indicate that both survey techniques result in similar values. Typically, results within 5 points of one another, using the same technique, would be considered satisfactory.

	Manual PCI	Automated PCI	Difference
INTREPID 01	87	85	-2
INTREPID 02	64	60	-4
INTREPID 03	30	34	+4
ORISKANY 01	33	35	+2
ORISKANY 02	89	86	-3
ORISKANY 03	94	89	-5
KEARSARGE 01	27	86	*
KEARSARGE 02	53	52	-1
KEARSARGE 03	55	60	+5
CLUB DRIVE 01	16	13	-3

**Incorrect part of section used. KEARSARGE 01 is part of a parking area and based on the results, it is apparent that the two procedures followed a different centerline. Based on the existing conditions, shifting of the centerline along this section would in fact change the PCI significantly. Therefore, when this occurs, closer attention needs to be given to where the road is actually centered.*

3.8 Effect of Sample Unit Size on PCI Accuracy

The effect of sample unit size on PCI accuracy was investigated for asphalt roadways, (Shahin et al. 1996). The study was conducted by employing the 35-mm film automated distress data collection technique. Twenty-four asphalt pavement sections were surveyed. The film from each section was divided into image units (frames) each one lane wide (12.5 ft) and 10 ft long (125 sq ft). A regular sample unit was defined as one lane wide by 200 ft long (2,500 sq ft) or 20 frames. Figure 3–41 shows a comparison between the PCI calculated using a 250-sq ft sample size (10% of regular size) and the PCI calculated using a 2,500 sq ft sample size (regular size). Similar figures can be plotted for the other sizes, however, the closer the size to the regular size the lower the error. Figure 3–42 shows a plot between the relative sample unit size to regular size and the expected amount of error in the PCI. As can be seen from the figure, as long as the size is within 40% from the regular size, the error is limited to about 2 points.

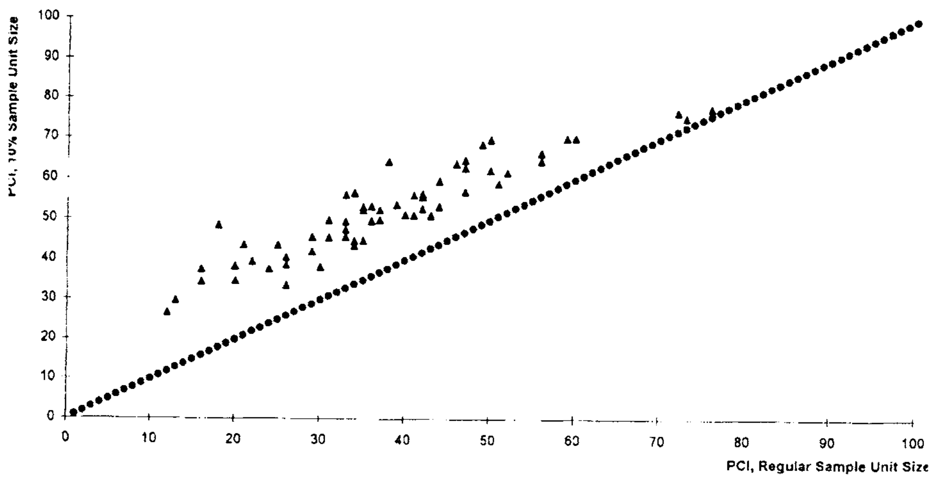


Figure 3-41. Comparision of the Effect of Sample Unit Size on PCI Accuracy (Shahin et al. 1996).

3.9 PCI Calculation Using Micro PAVER

Computing the PCI manually is not a tedious operation for a single sample unit, but the volume of data generated from a survey is generally quite large, and calculations involving these data are time-consuming. Once distress information has been entered into PAVER, the program automatically calculates the PCI of each sample unit surveyed and determines an overall PCI for a section, as well as extrapolated distress quantities. The program can also determine the percentage of deduct values based on distress mechanism (i.e., load, climate, and other) for a section. The percentage of deduct values attributed to each distress mechanism is the basis for determining the primary causes of pavement deterioration. Figure 3–43 shows an example of an automated PCI calculation from the PAVER system.

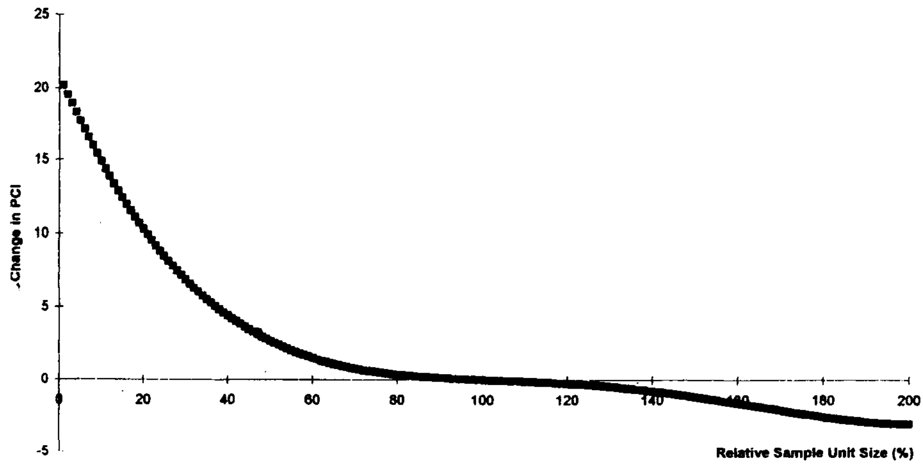


Figure 3-42. Relative Sample Unit Size and Expected Amount of Error in PCI Calculation (Shahin et al. 1996).

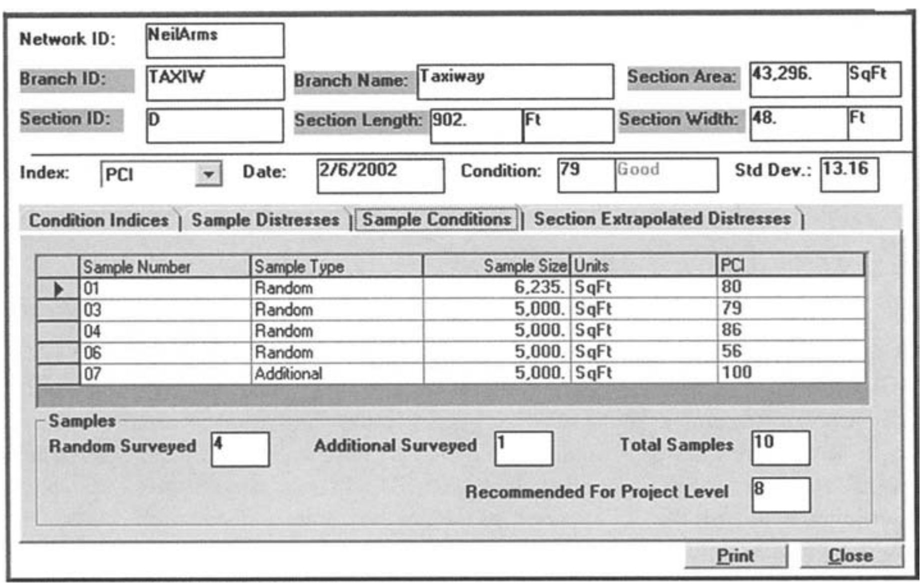


Figure 3-43. Example Automated PCI Calculation from the Micro PAVER System.

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4

Nondestructive Deflection Testing (NDT)

4.1 Introduction

One of the most reliable methods for determining the structural condition of an in-service pavement is to use nondestructive deflection testing (NDT). NDT has two major advantages over destructive testing. First, destructive testing by definition disturbs the underlying paving layers or requires removal of the pavement materials to a laboratory for testing, whereas NDT is truly an *in situ* test that evaluates the pavement without any material disturbance or modification. The second advantage of NDT is that the tests are relatively quick and inexpensive, allowing more tests to be completed while causing less disruption to traffic than destructive testing. It is recommended practice that some coring be performed in association with NDT to verify layer thicknesses for accurate back-calculation of the layer moduli. In general, however, the amount of destructive testing needed to evaluate a pavement in conjunction with NDT is minimal.

NDT equipment operates by applying a load to the pavement and measuring the resulting maximum surface deflection or the surface deflection basin. NDT results are used to determine the following:

1. Asphalt Pavements
   a. Elastic modulus of each of the structural layers
   b. Allowable loads for a specified number of load applications
   c. Overlay thickness design
2. Concrete Pavements
   a. Concrete elastic modulus and subgrade modulus of reaction
   b. Load transfer across joints
   c. Void detection
   d. Allowable loads for a specified number of load applications
   e. Rehabilitation design

NDT data can be used in conjunction with the information from the distress survey to select the best maintenance and rehabilitation (M&R) alternative. It is recommended that NDT be conducted prior to destructive testing to better select the location for coring and material sampling, if required. For airfield pavements, NDT results can be used to determine the Aircraft Classification Number/ Pavement Classification Number ratio (ACN/PCN). The determination of ACN/ PCN is presented in Section 4.7.

4.2 Pavement Deflection Measurement Devices

At present, there are many different commercially available deflection testing devices. The devices are grouped based on loading mode as: impulse, steady-state dynamic, and static. The impulse NDT devices are the most recently developed. They better simulate the load from a moving tire and are currently used by highway and airport agencies more than any of the other devices . Impulse deflection devices are presented in Section 4.2.1. The other deflection device types, steady-state dynamic and static, are presented in Sections 4.2.2 and 4.2.3, respectively.

4.2.1 Impulse Deflection Devices

The Falling Weight Deflectometer (FWD) is the most commonly used impulse deflection device. FWD is based on the concept of dropping a weight (W) from a known height (H), thus producing a kinetic energy that is equal to W x H. The resulting force pulse transmitted to the pavement approximates the shape of a half-sine wave. The load pulse shape and duration have a significant impact on measured deflection as discussed in Section 4.3.

The integration of the force applied to the pavement (F) multiplied by the composite compression of the falling weight and the pavement (δ) is equal to the produced kinetic energy as follows:

$$\int_{\delta=0}^{\delta=\Delta} F \cdot d\delta = W \cdot H \qquad (4\text{-}1)$$

Where Δ = total compression

A rough approximation of Equation 4–1 is

$$0.5 \cdot F \cdot \Delta \cong W \cdot H \quad \text{, or}$$

$$F \cong \frac{2W \cdot H}{\Delta}$$

For example dropping a weight of 400 lbf from a height of 2 ft that produces total compression of 0.5 in. will produce a force on the pavement of approximately 38,400 lbf.

An FWD device can apply loads from 3,000 to over 50,000 lbf based on the device used. FWD devices have relatively low static preloads. The preload will vary from few hundred to few thousand pounds based on the device. Thus, the negative effects of a high preload are avoided. Following is a description of two of the commercially available FWD devices. Other FWD devices that are available but not presented in this book include the JILS (Foundation Mechanics 2004), and the Carl Bro (Carl Bro 2004).

4.2.1.1 Dynatest FWD

Dynatest offers two FWD models (Dynatest 2004). One of the models, (Fig. 4–1), can apply peak impact loads in the range of 1,500 lbf to 27,000 lbf (7 KN to 120 KN). The second model (Fig. 4–2), also known as the heavy FWD (or HWD) can apply peak impact loads in the range of 6,500 lbf to 54,000 lbf (30 KN to 240 KN).

The Dynatest FWD is a trailer-mounted system and the operations control computer is usually located in the tow vehicle. The impulse load is produced by a single-mass falling weight striking a buffer system which, in turn, transfers the energy to the loading plate. The automated operation control performs several functions including the lowering and raising of the loading plate and deflection sensor bar to and from the surface of

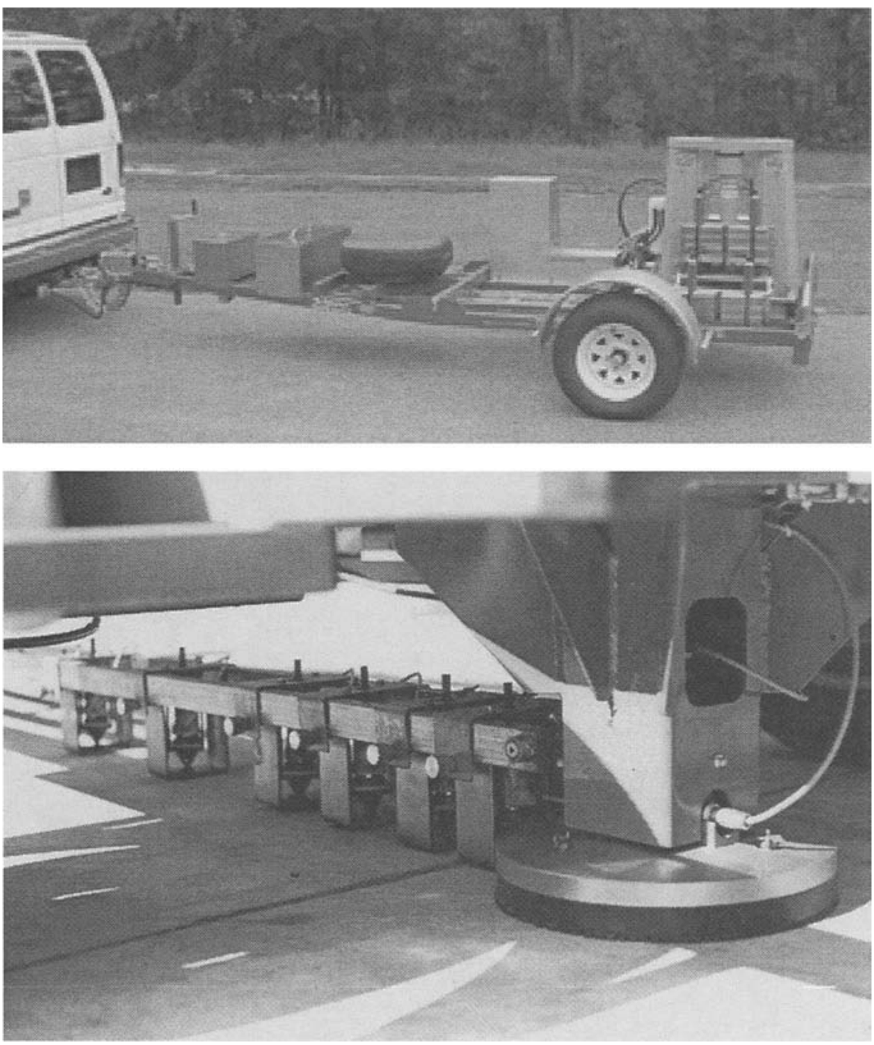

Figure 4-1. Dynatest FWD.

Figure 4-2. Heavy Dynatest FWD (HWD).

the pavement being tested. The automated controls, after lowering the loading plate, can run any selected test sequence of up to sixty-four (64) drops (or more with optional software) before raising the loading plate. The drops can be performed automatically from any combination of four pre-selected heights. Additional backup operation controls are also provided for manually carrying out each step in the test.

The buffer system that the weight strikes is changed along with the size of the weight to produce an approximate half-sine wave load pulse with a loading time of 25 – 30 msec for any falling weight. The minimum rise time is 10 – 15 msec for all loads.

Two different sizes of loading plates are provided, one that is 300 mm in diameter and one that is 450 mm in diameter. A split 300 mm diameter plate assembly (2 segments) is optionally available for improved surface contact on rutted surfaces.

Deflections are measured using velocity transducers (normally seven or optionally nine). One of the transducers measures the deflection of the pavement surface through the center of the loading plate. The remainder of the transducers can be positioned along the raise/lower bar up to a distance of 3 m (10 ft) from the center of the loading plate. A "rear deflector extension bar" and/or "rear/transverse deflector extension bar" is optionally available which can accommodate at least two of the deflectors.

Other optional equipment for the Dynatest FWD include:

Video System—designed for operator assistance in Portland Cement Concrete (PCC) joint testing.

Automated Differential Global Positioning System (GPS) Unit—to determine "absolute" location from satellite transmitted GPS data.

Automated Air Temperature Probe—directly feeding ambient (air) temperature data into the Field Program. The probe may be used for manually taking the asphalt temperature.

Automated, Non-contact Infra-Red Temperature Transmitter—to monitor and store the pavement SURFACE temperature.

4.2.1.2 KUAB FWD

KUAB manufactures two types of FWDs (KUAB 2004). One type uses a single mass system for load generation similar to the Dynatest described in 4.2.1.1. The other type uses a KUAB patented two-mass force generating system.

In a single-mass system, a mass is dropped on rubber buffers which transmits the load to the plate. In a two-mass system (Fig. 4–3 and 4–4), a mass is dropped on rubber buffers, which transmits the force to a second mass (intermediate mass). The intermediate mass transmits the force to another set of rubber buffers, which in turn transmits the force to the plate. The two-mass system creates a smoother load pulse than that produced by a single-mass system (Section 4.3).

Both KUAB FWD types come in a variety of models where peak load, load rise time, number of deflection sensors, and load plate size can be selected by the users. The peak load can be varied from 3,000 to 66,000 lbf. (a special model was also produced with a peak load of 130,000 lbf). The load rise time is 10 to 30 msec. The load pulse is measured with a load cell. Two loading plates are available that are 11.8 in. and 17.7 in. diameter. The load plates are segmented (Fig. 4–5). Each load plate is divided into four quarter-circle segments that are terminated in a common hydraulic pressure chamber. Each segment of the load plate is free to conform to the shape of the pavement surface being tested.

Deflections are measured using absolute seismic displacement transducers (seismometers) or velocity transducers. One sensor is placed at the center of the plate. Other sensors are mounted on a bar and automatically lowered to the pavement surface with the loading plate. The most common number of deflection sensors is 7; however, up to 16 deflection sensors can be delivered in various configurations.

Figure 4-3. KUAB FWD.

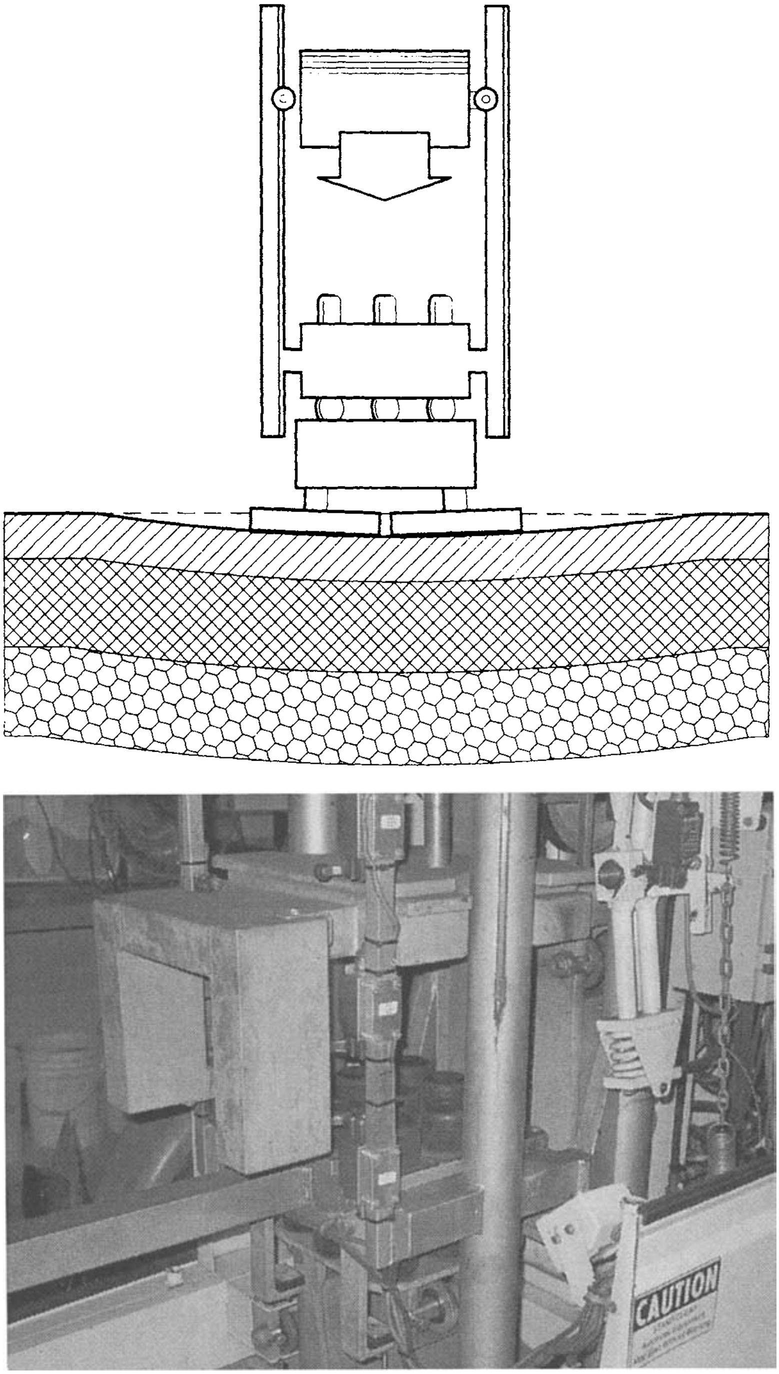

Figure 4-4. Two Mass System of KUAB FWD.

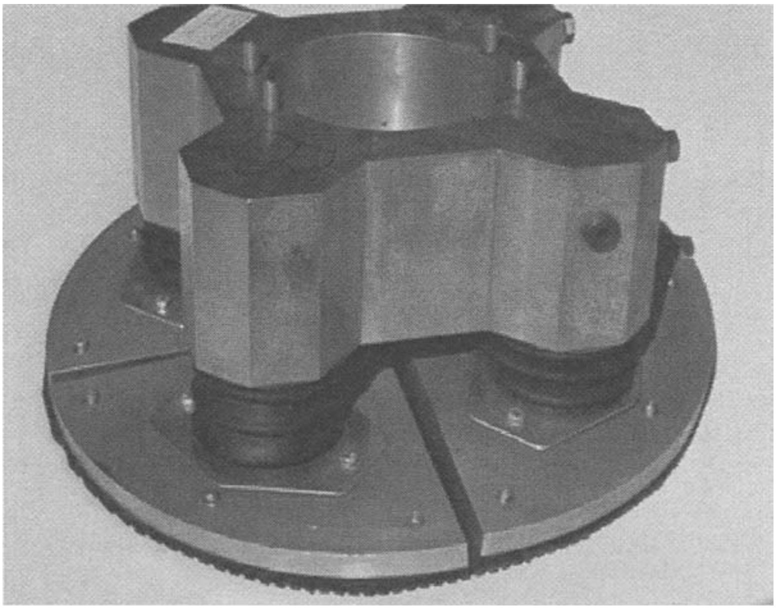

Figure 4-5. Segmented Load Plate.

The KUAB models are available as both trailer or van-mounted. In the trailer-mounted models, the signals from the load cell and deflection sensors are processed by a computer placed in the tow vehicle, which also controls the device's complete operation.

There are several options available with each of the models, including temperature measuring gauges, distance measuring systems, core drill, video camera, and GPS device.

4.2.2 Steady-State Dynamic Deflection Devices

Steady-state dynamic deflection devices all use a similar mode of operation. A relatively large static preload is applied to the pavement and a sinusoidal vibration is created by the dynamic force generator. Figure 4–6 shows a typical loading series. The amplitude of the peak-to-peak dynamic force must be less than the static force; otherwise, the device will bounce off the pavement surface. It is important to note that a substantial applied static load is always present. This may adversely affect the accuracy of the test. Most paving materials are stress-sensitive, meaning that their stiffness changes depending on the stress levels. A high preload will therefore change the stiffness of the materials, producing deflection data that may not be representative of how the pavement would respond under a moving wheel load.

Some steady-state vibration devices allow the amplitude and frequency of the wave to change, producing different load magnitudes. When testing a pavement section with this equipment, deflection data can be obtained for a number of different load magnitudes. In general, pavements do not exhibit a linear load vs. deflection relationship. By varying the load, a better characterization of a pavement's response to load can be

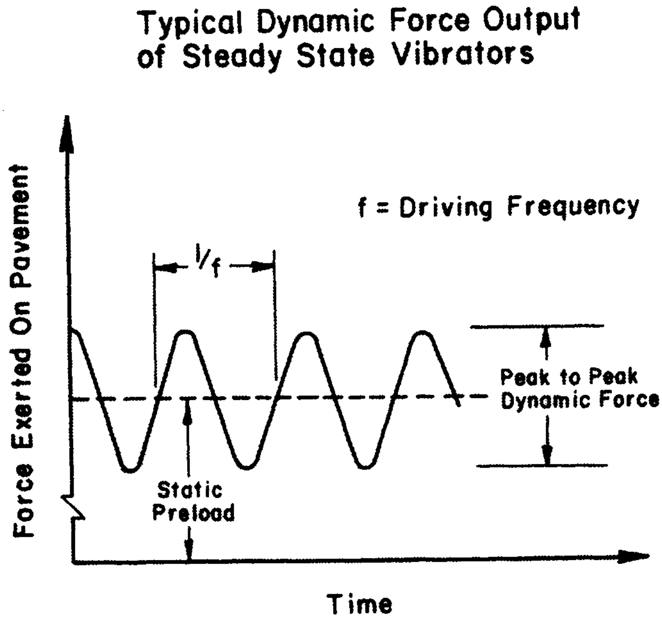

Figure 4-6. Typical Dynamic Force Output of Steady-State Vibrators.

obtained. Many agencies have replaced their steady-state deflection equipment with FWD type devices. One steady-state deflection device still used by some agencies is the Dynaflect.

4.2.2.1 Dynaflect Equipment

The Dynaflect, shown in Figure 4–7, was one of the first commercially available steady-state dynamic deflection devices. It is trailer-mounted and can be towed by a standard automobile. The Dynaflect is an electromechanical system. "The cyclic force generator utilizes a pair of unbalanced flywheels, rotating in opposite directions at a speed of 480 rpm or 8 cycles per second. The vertical component of the acceleration of the unbalanced mass produces the cyclic force" (Dynaflect 2004). The Dynaflect has a static weight of 2,000 lb and produces a 1,000 lb peak-to-peak dynamic force at a fixed frequency of 8 cycles/sec. The load is applied through two rigid steel wheels and the resulting deflections are recorded by five velocity transducers (geophones). The transducers are suspended from a placing bar and are normally positioned with one located between the two wheels and the remaining four placed at 1-ft intervals.

The unit is moved to the test site and the loading wheels and transducer are lowered to the surface. Once a test is complete, the sensors and rigid steel wheels are raised, and it is moved to the next test site. If only a short distance exists between test sites, the unit can be moved on the rigid steel wheels at a maximum speed of 6 mph. Most new models have a completely automated test sequence.

Technical limitations of this device include: (1) the maximum peak-to-peak force is 1,000 lb (a force many believe to be far too small for testing heavy highway or airfield pavements), and (2) neither the magnitude nor the frequency of the load can be varied. However, agencies that have used the Dynaflect for many years have developed a significant database of information to facilitate better use of the data.

4.2.3 Static Deflection Devices

Static deflection devices apply either a static or slow-moving load to the pavement surface and measures the resulting deflections. The most commonly used static deflection device is the Benkelman Beam

4.2.3.1 Benkelman Beam

The Benkelman Beam (Fig. 4–8) is a simple hand-operated deflection device. It consists of a support beam and a probe arm. The probe arm is 10 ft long and is pivoted at a point 8 ft from the probe which rests upon the pavement surface. It is used by placing the tip of the probe between the dual tires of a loaded truck, typically an 18,000-lb axle load. As the loaded vehicle moves away from the beam, the rebound or upward movement of the pavement is recorded.

Some problems encountered with this device include: (1) the need to ensure that the front supports are not in the deflection basin, and (2) the difficulty or inability to determine the shape and size of the deflection basin.

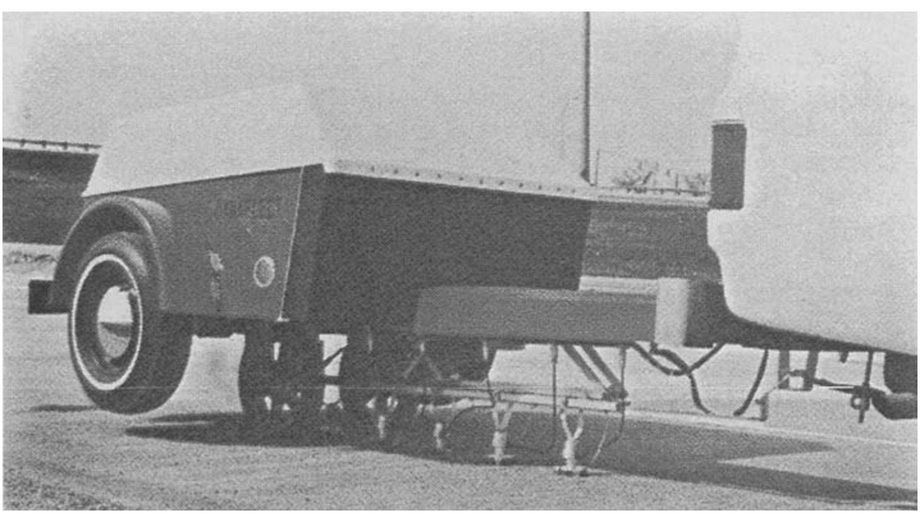

Figure 4-7. Dynaflect Steady-State Dynamic Deflection Device.

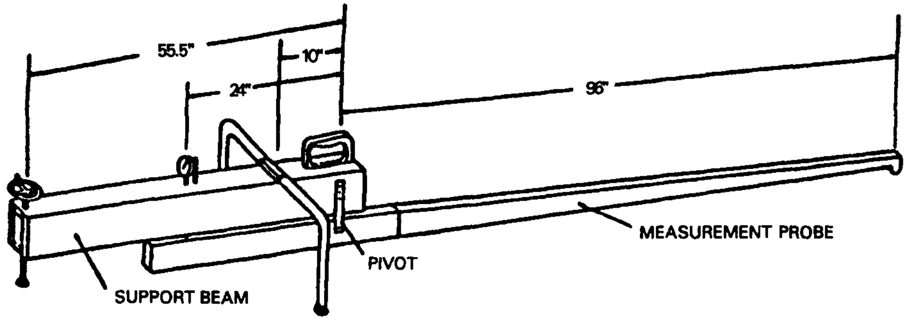

Figure 4-8. Benkelman Beam.

4.3 Factors Affecting Deflection Values

4.3.1 Pavement Structure

The deflection of a pavement in response to an applied load represents an overall system response. It is important to remember that the complete pavement system consists of all constructed layers (i.e., subbase, base, surfacing) plus the subgrade. The deflected surface profile is commonly referred to as the deflection "basin" or "bowl." The shape of the basin, including maximum deflection under the load and tapering deflection away from the load, is an important parameter in analyzing pavement systems. In general, a weaker system will deflect more than a stronger system under the same load; however, the exact shape of the basin is related to the strengths of the individual component layers. The deflection basin "area" is a calculated value based on surface deflections. Typically, deflection data is obtained directly beneath the load and at radial distances up to 72 in. (1.8 m) from the center of the pressure plate.

Figure 4–9 presents two deflection basins obtained from computer simulation of two pavement systems of equal thickness but differing component strengths. Although the maximum deflection is the same in both cases, the basin shape differs. For Case A ("Strong"), the maximum deflection is 23.1 mils and the basin area equals 21.73 in. For Case B ("Weak"), the maximum deflection is the same while the calculated basin area is 16.96 in. The area is computed as illustrated in Figure 4–10 for deflections measured at 0, 12, 24, and 36 in. from the center of the plate.

$$\text{AREA} = \frac{6}{D_0} \times \left(D_0 + 2 \times D_{12} + 2 \times D_{24} + D_{36}\right) \qquad (4\text{-}2)$$

where

AREA = deflection basin area in inches

D_i = surface deflection at radial distance i

The surface fatigue life of a pavement is directly proportional to the critical load induced strains in the asphalt, with higher strain values indicating shorter lives. The calculated values for Cases A and B are 254 and 363 micro units, respectively. This

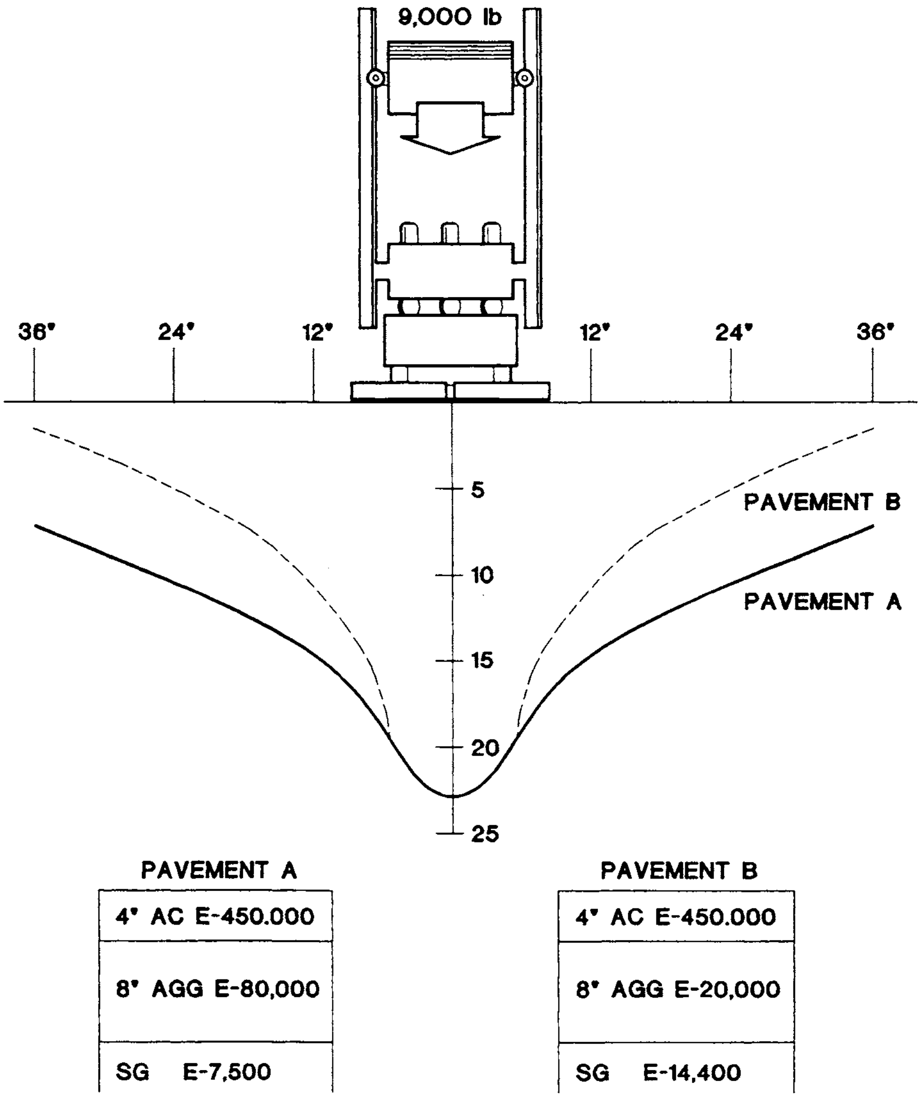

Figure 4-9. Comparison of Deflection Basin of Two Pavements.

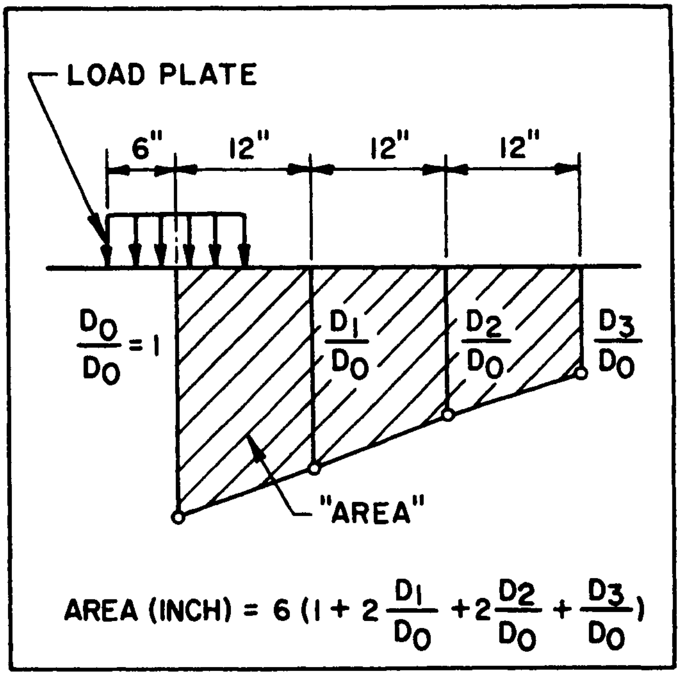

Figure 4-10. Calculation of Basin Area (From Hoffman and Thompson, 1981).

indicates that although the maximum deflections of both pavements are equal, pavement system B ("Weak") would be expected to fail sooner due to the higher strains.

Although maximum deflection based design procedures do provide a relatively sound basis for analysis, they are not without their limitations. Maximum deflection describes how the overall pavement system behaves under a load, but not necessarily how the individual layers are going to resist fatigue or permanent deformation.

4.3.2 Load Magnitude

A second factor that affects recorded deflection values is the magnitude of load. Load levels ranging from as little as 1,000 lbf to over 50,000 lbf are available. Some NDT units offer the potential to vary the applied load while others use a constant value. Many researchers have found that light loads do not sufficiently stress the underlying layers of heavy highway and airport pavements (Bush, Alexander, and Hall 1985; FAA 1976; Hall 1975; Ullidtz and Stubstad 1985). FAA Advisory Circular 150/5370-11 (FAA 1976) states that "the load deflection relationship of pavements is often nonlinear, and test results obtained by using small loads which have to be extrapolated over one or two orders of magnitude can result in serious errors." To accurately characterize pavement's response under design loads, the load level of the NDT device should be selected as closely as possible to those design load values. An example of the nonlinear relationship between load and deflection is shown in Figure 4–11. This means that characterizing pavement response to a heavy load through the use of a small load could be very misleading. Bush et al. (1985) state that under a light load the "force may not seat the

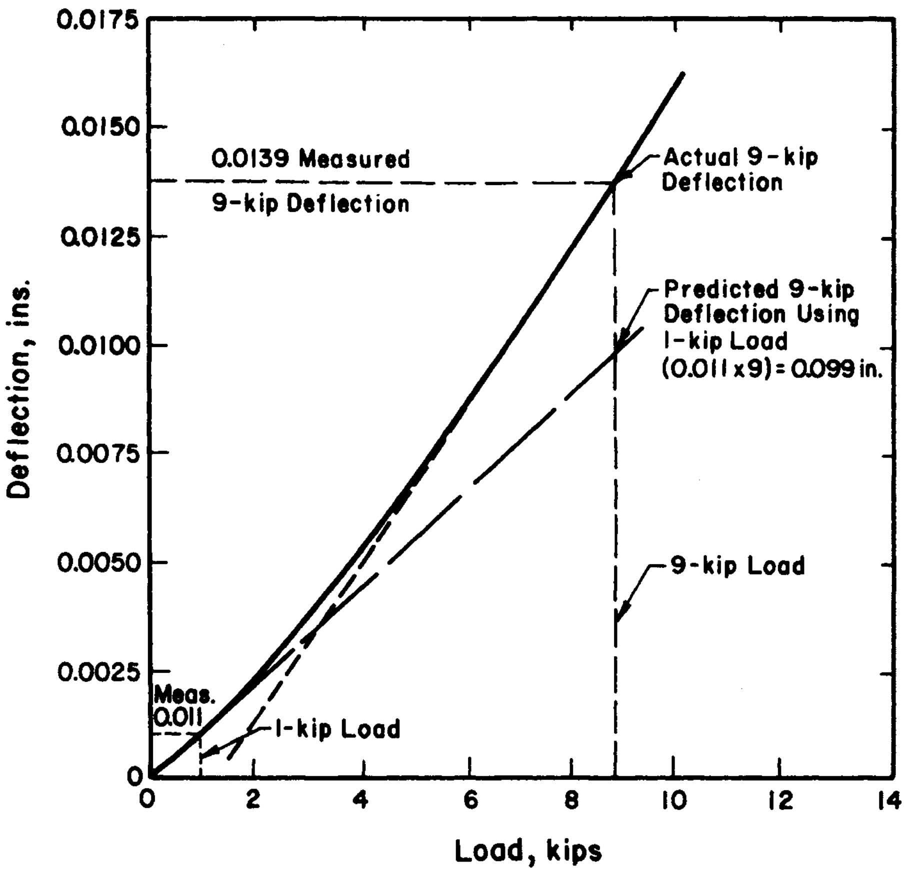

Figure 4-11. Illustration of Nonlinear Relationship Between Load and Deflection.

pavement and excite the full strength of the supporting subgrade." Furthermore, the small deflections produced by the light loading devices make it more difficult to record the deflection accurately.

As a general rule, the testing load should not be less than half the design load.

4.3.3 Loading Mode

Even if the magnitude of the load is held constant, the pavement response can vary greatly depending on the mode of the loading. As discussed in the section on equipment, there are three different modes of loading: static, steady-state dynamic, and impulse. Out of these three, the impulse mode simulates the load from a moving vehicle best. However, even within the general impulse type of loading, the shape of the produced loading pulse and the loading duration are of extreme importance.

Normally the load pulse is not symmetrical (see Fig. 4–12); therefore, using the total length of load pulse duration to describe loading time would be misleading. It is strongly recommended that the time from zero to peak load "rise time" be used instead, unless the pulse happens to be symmetric.

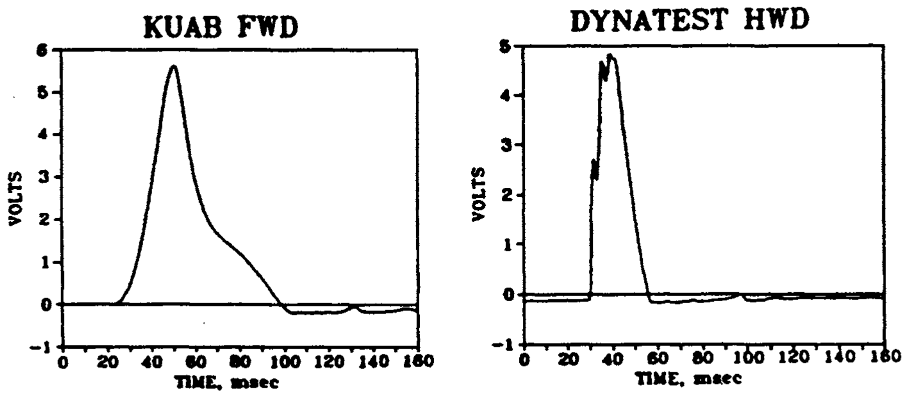

Figure 4-12. Typical Load Pulse Plots.

The selection of a rise time to simulate vehicle loading is rather a difficult task. There is a fundamental difference between the moving load of a vehicle and the stationary load of an FWD. When a vehicle travels a pavement, there is always a deflection bowl around the wheel. If the deflection bowl is frozen and examined at an instant in time, there is only one bowl. The deflection bowl caused by an FWD is different. It takes some time for the deflection to travel from the plate outward.

With the FWD, the peak deflection is measured in each sensor position when it occurs, and a deflection bowl is constructed using peak deflections even though they did not occur at the same time. Figure 4–13 shows the load pulse at the bottom, and above it are the deflection time histories from plate center to 1.8 m away. In spite of this, the data is often analyzed as if there was a stationary bowl, using the maximum values as if they existed at the same time. This error could be minimized by increasing the rise time. By using a sufficiently long rise time, the time lag between sensor peak deflection is minimized. This is demonstrated in Figure 4–14.

The effect of the load pulse shape and rise time cannot be overlooked because it can affect deflection peak values by as much as 10% to 20%.

4.3.4 Load Distribution

Most mechanistic analysis routines assume a full contact between the loading plate of the NDT device and the pavement being tested, thus assuming a circular uniform stress distribution under the loaded area.

Touma et al. conducted a field test where a pressure- sensitive film was placed under two FWD devices, one with a segmented load plate and the other with a solid load plate. The segmented plate was that described under the KUAB FWD and shown in Figure 4–15. The testing was conducted on three pavements: a smooth, newly paved asphalt pavement, a relatively strong asphalt pavement that had a rut depth of 1/8 in, and a relatively weak chip-seal pavement which had a flat profile under the loading plate. The measured pressure distribution under the segmented and solid plates is shown in Figure 4–15 for the three pavements. Mechanistic analysis using the field data showed that if full contact is assumed when in reality it did not occur, significant errors reaching 100% may result in the back-calculated layer moduli.

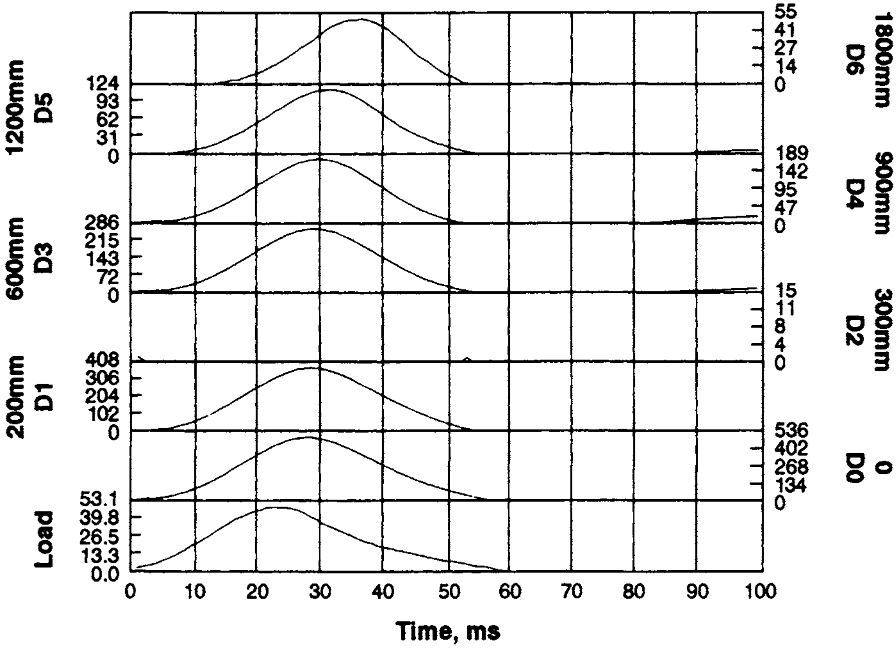

Figure 4-13. Time history for a FWD load pulse and deflections 0-1800 mm from load center.

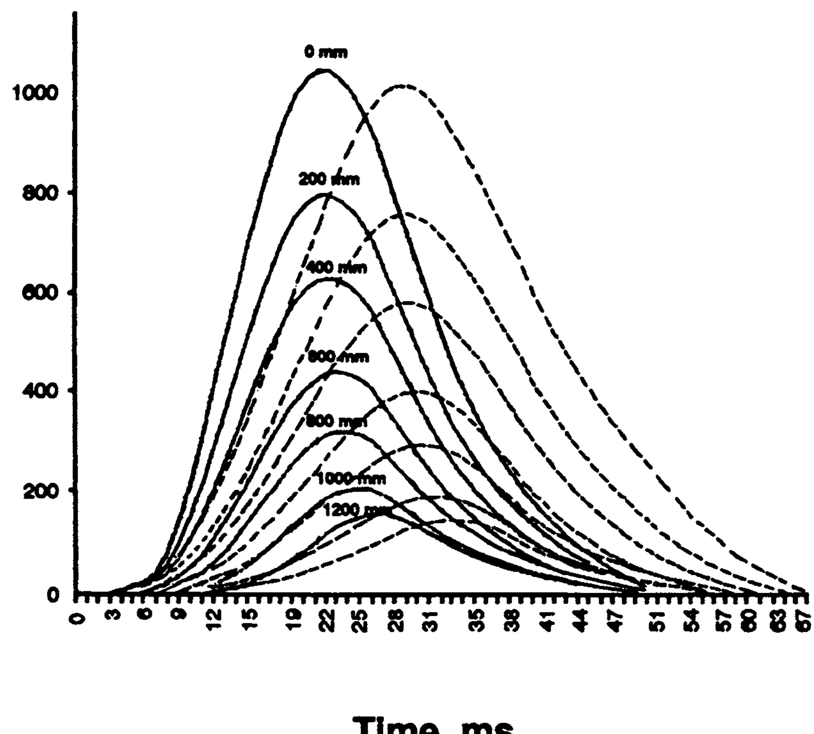

Figure 4-14. Time History for Deflection 0-1200 mm from Load Center for Different Rise Times.

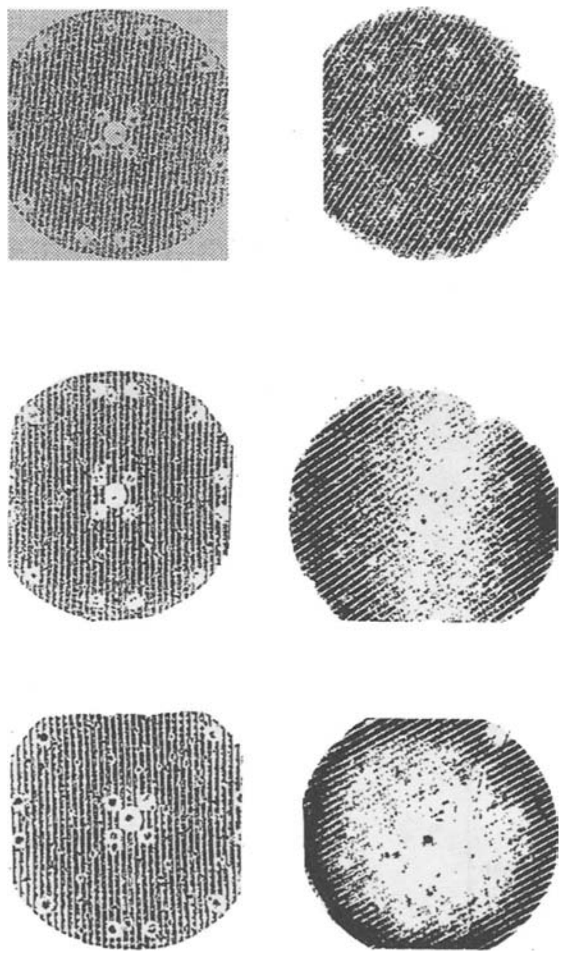

Figure 4-15. Actual Pressure Distribution Under Rigid and Segmented Plates (From Touma, Crovetti, and Shahin 1990).

Because it is difficult and impractical to measure the load distribution associated with each field test, it is important to use a loading plate of such design to ensure full contact with the pavement for all conditions.

4.3.5 Pavement Temperature

Another factor that must be closely monitored during testing is the pavement temperature. When testing asphalt pavements, the deflection changes as pavement temperature varies because the stiffness of the asphalt layer is a function of its temperature. At higher temperatures, the asphalt stiffness is reduced, thus increasing deflections.

Figure 4–16 shows an example of the relationship between asphalt concrete stiffness and temperature. Figure 4–17 shows an example impact of the asphalt concrete stiffness on surface deflection as measured at 0, 12, 26, and 36 in. from the center of the loaded plate. As can be seen from the figure, the highest impact of the asphalt concrete stiffness is at D0 and there is hardly any impact on D36. This is to be expected as the sensor located at 36 in. away from the center from the load will measure deflections occurring in the subgrade.

Deflections of plain jointed PCC pavements are also affected by temperature changes, particularly at joints and cracks. As the slab warms up, it will expand, causing the joints and cracks to become tighter. This will reduce the maximum deflection recorded.

It is therefore necessary to record the pavement temperature during testing. A relationship should be developed that will allow all deflections to be corrected to a standard temperature, such as 70° F. This can be done by repeatedly testing representative points throughout the day and recording the pavement temperature and deflection. This data can then be plotted to find a relationship to convert the deflections found at any temperature to a deflection at a standard temperature, provided this standard temperature falls within those recorded during testing.

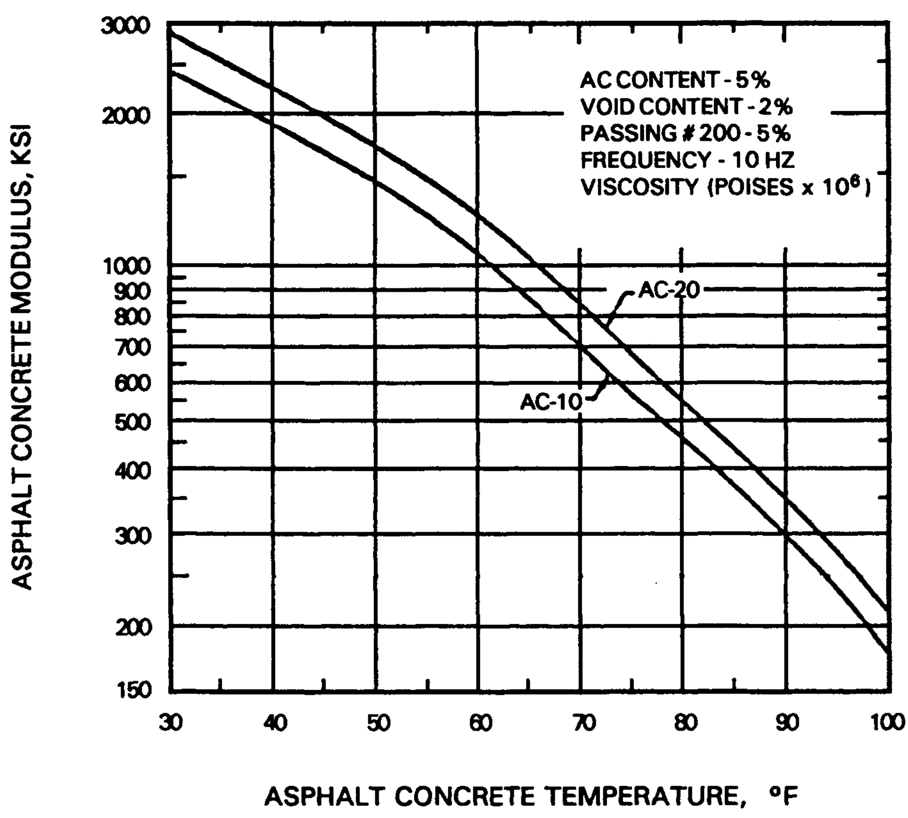

Figure 4-16. Typical Ashpalt Concrete Modulus–Temperature Relationship (From Thompson, M.R. 1984).

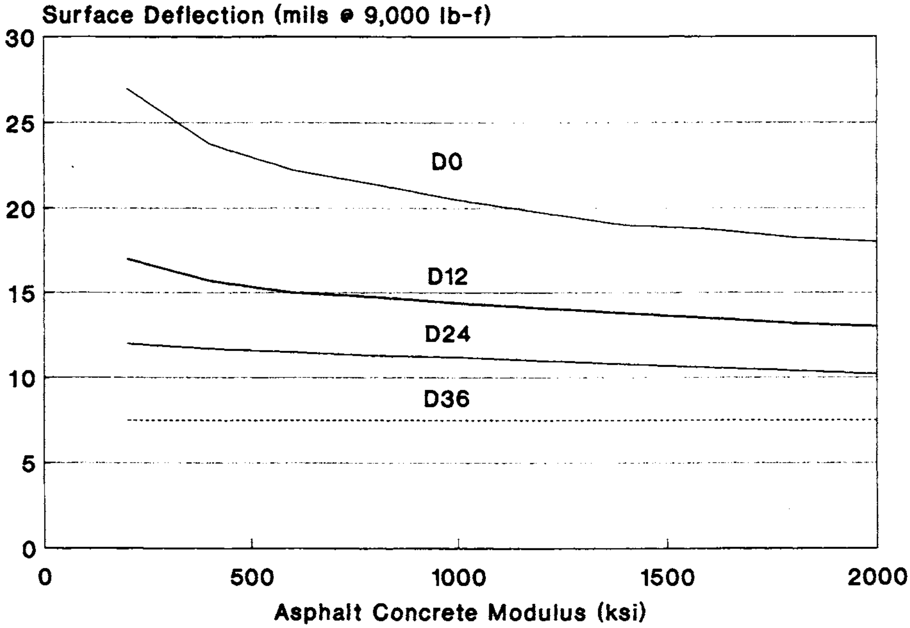

Figure 4-17. Surface deflection vs. asphalt concrete modulus.

4.3.6 Testing Season

The testing season is also an important factor in analyzing deflections. There are basically four distinct seasonal periods in cold climatic areas, as shown in Figure 4–18.

1. The period of deep frost when the pavement is very strong.
2. The period during which the frost is beginning to disappear from the pavement subgrade system and the deflection increases greatly due to saturated pavement layers.
3. The period during which the excess pore water from the melting frost leaves the pavement/subgrade and the soil begins to recover, and the deflection decreases rapidly.
4. The period during which the deflection levels off slowly as water content slowly decreases.

In areas that do not experience freeze-thaw, the deflections follow more of a sine curve, with the peak deflection occurring either in the spring when significant free moisture exists or in the hot summer in relatively dry areas.

Each agency must determine during what season the peak deflections are at a maximum. It is desirable that the deflection testing be conducted at this crucial season. If it is not possible, an adjustment factor should be applied to relate the measured deflection to the deflection that would be obtained during the critical season. This adjustment factor must account for both temperature and moisture variations. A deflection survey is not recommended when the subgrade is frozen.

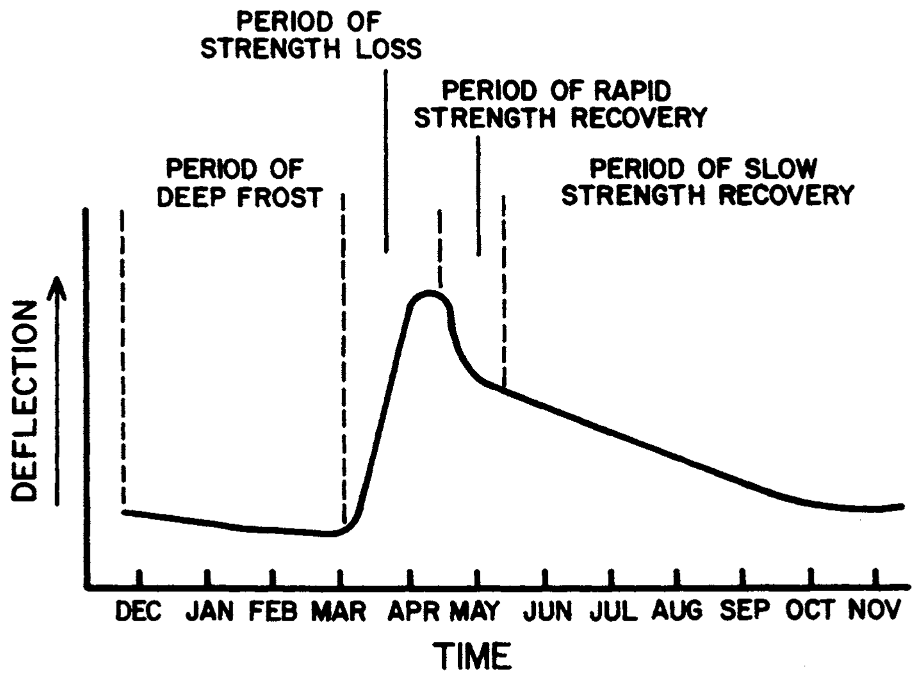

Figure 4-18. Seasonal Effects on Pavement Deflection (From the Asphalt Institute MS-1).

4.4 Uses of NDT at Different Levels of Pavement Management

The use of NDT is usually related to the level of pavement management. At the network level, NDT is used to identify the beginning and end of pavement management sections and to identify those pavement sections that should receive project testing and evaluation. Figures 2–1 and 2–3 are examples of how NDT is used to identify pavement sections for M&R management. For airfield pavements, NDT is performed at the network level every 5 to 10 years to calculate a structural index for each pavement section. A typical airfield structural index is the ACN/PCN which is described in Section 4.7.

At the project level, NDT is used to identify the location and cause of failure in flexible pavements, detect voids, and determine load transfer across joints or cracks in rigid pavements. Figure 4–19 is an example of how NDT is used for void detection in concrete pavements. The procedure shown in the example was developed by Crovetti and Darter (1986); in this method the slab corner deflection is measured at two load levels and the intercept is determined. If the intercept is greater than 2 to 3 mils (1 mil = 0.001 inch), then the pavement is suspected of having voids or poor support. Figure 4–20 shows how the load transfer is calculated across a joint for concrete pavement. The load transfer (LT) is calculated by placing the testing plate on one side of the joint and a sensor on the other side (see Fig. 4–1). The LT is calculated as the ratio of the deflection of the unloaded slab divided by the deflection of the loaded slab where the plate is placed.

NDT is also used to calculate layer moduli, establish load limits, determine remaining structural life, and determine overlay design. The layer moduli back-calculation is nor-

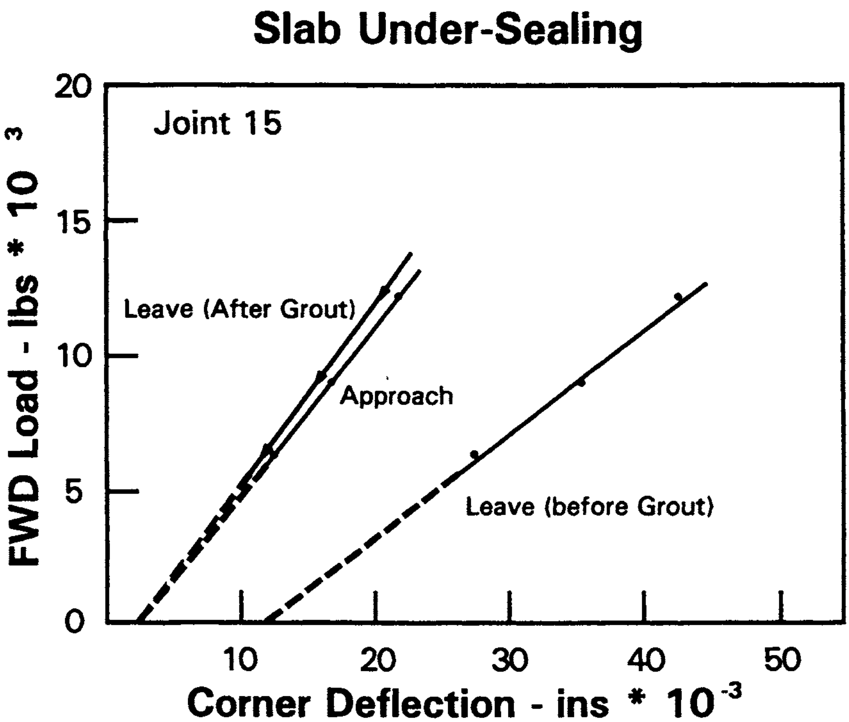

Figure 4-19. Use of NDT for Void Detection (From Crovetti and Darter 1986).

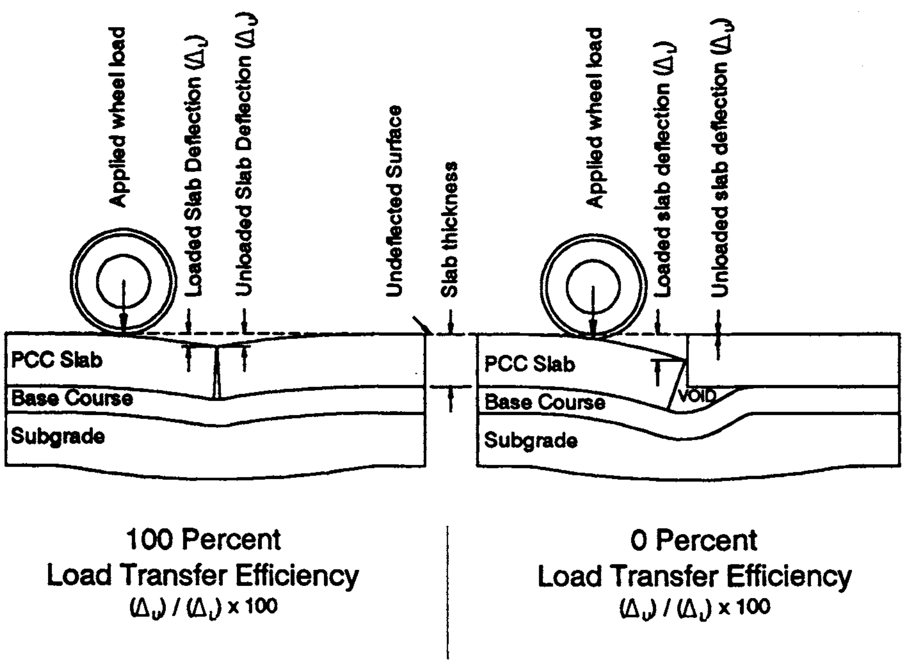

Figure 4-20. Joint Load Transfer Concept.

mally performed by mechanistically finding the layer moduli that will produce the same deflection bowl as measured in the field (Fig. 4–21). This process requires engineering judgment because there can be several combinations of layer moduli that will produce close approximation of the measured field deflections. The larger the number of layers, the more difficult the process becomes. Once the layer moduli have been estimated, stresses and strains under traffic are calculated and the pavement fatigue life is estimated. This data can be used further to estimate required overlay thickness or to establish load limits as illustrated in Figure 4–22.

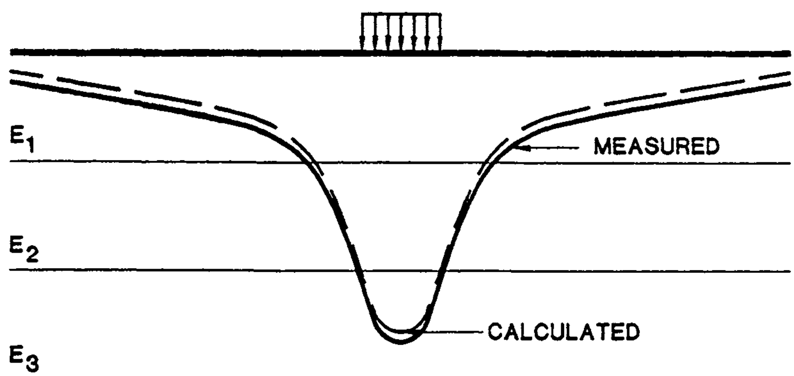

Figure 4-21. Layer Moduli Back-Calculation.

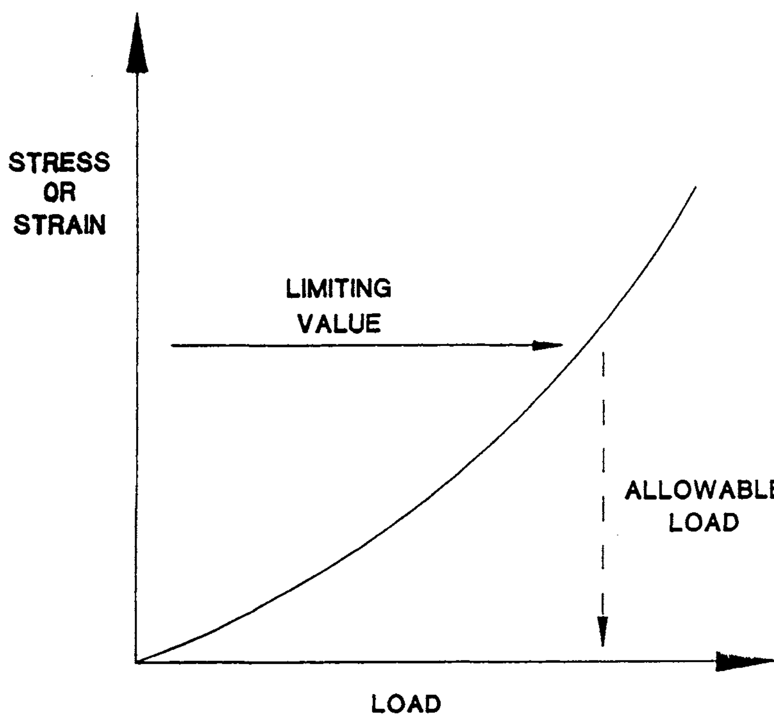

Figure 4-22. Establishing Load Limits.

4.5 Design of NDT Field Survey

The NDT field survey design includes determining the number of points to be tested (or the frequency of testing) and specifying the location of points to be tested.

4.5.1 Frequency of Testing

Even though NDT is used at the network and project levels, the frequency of testing at the network level is normally much less than at the project level. Three levels of testing frequencies have been recommended by ASTM (D4694):

Level I—A general overview of pavement condition for network analysis.

Level II—A routine analysis of the pavement for purposes such as overlay or rehabilitation design projects.

Level III—A detailed or specific analysis of the pavement, such as the evaluation of joint efficiency or foundation support for PCC slabs.

The recommended testing frequencies for the three levels are: 500 to 1,500 ft, 100 to 500 ft, and 10 to 100 ft, respectively.

4.5.2 Location of Testing

For flexible pavement, the testing is usually conducted in the outer wheel path. The testing may also be conducted in the inner wheel path based on the specific pavement condition and testing objectives. This will also determine whether each lane should be tested in multiple-lane highways.

Jointed concrete slabs are tested at the center of the slab to determine layer material properties, at the joints to determine load transfer, and at the corner to locate voids or weak foundation support (Fig. 4–23).

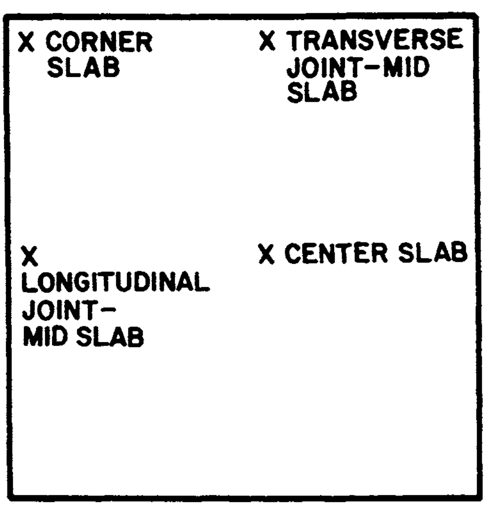

Figure 4-23. Recommended NDT Test Pattern for PCC Slab.

4.6 Airfield Pavement Structural Evaluation Using NDT

FWD is the primary equipment used for airfield pavement structural evaluation. Several computer programs are available to analyze the results. The U.S. Army Corps of Engineers PCASE (PCASE 2004) is presented here as an example. PCASE contains a set of programs that were developed and are continually being updated the by the Corps. Following is a brief description of the PCASE programs used in structural evaluation:

1. BASIN selects a representative deflection basin for each pavement section.
2. WESDEF determines the set of layer modulus values that provides the best fit between measured and computed deflection basins.
3. TRAFFIC determines the critical aircraft and computes equivalent passes of that aircraft based on the projected traffic mix. The critical aircraft is the aircraft from the mixture that requires the greatest pavement thickness to support its projected passes.
4. WESPAVE computes allowable aircraft loads at the design pass level, allowable passes of the design aircraft at maximum load, required overlay thickness, and Pavement Classification Number (PCN).

Output from PCASE includes:

- Allowable loads for a selected number of aircraft passes
- Allowable aircraft passes at a specified load
- PCN

Any of the above outputs can be used as a pavement structural index, but the one normally reported is the ACN/PCN where ACN is the Aircraft Classification Number. The ACN/PCN procedure is described in Section 4.7 below.

4.7 ACN/PCN Structural Index

The ACN/PCN method was developed by the International Civil Aviation Organization (ICAO) for determining the weight limitations on aircraft operating on an airport pavement. The weight limitations are obtained by comparing the Aircraft's Classification Number (ACN) with the airport's Pavement Classification Number (PCN). An aircraft having an ACN equal to or less than the PCN can operate on the pavement without weight restriction. The ACN/PCN system applies only to pavements with bearing strengths of 12,500 pounds (5700 kg) or greater. Reporting strength for pavements with bearing strengths less than 12,500 pounds is in terms of the maximum allowable aircraft weight and allowable tire pressure, if applicable.

The PCN for any pavement is reported by a code consisting of five elements as shown in Figure 4–24. Following is a brief description of each element:

1. PCN Numerical Value: Relative indicator of the load carrying capacity of the pavement in terms of a standard single wheel load at a tire pressure of 181 psi (1.25 MPA).
2. Pavement Type: R for rigid pavements and F for flexible pavements.
3. Subgrade Category: A, B, C, or D based on the CBR value for flexible pavements or subgrade reaction (K) value for rigid pavements. (See Chapter 11 for CBR and K determination procedures).
4. Tire Pressure Category: Has little effect on rigid pavements. Tire pressure may be restricted on asphaltic concrete depending on the quality of the asphalt mix and climatic conditions (Figure 4–25).
5. Evaluation Method: The symbol T is used if the evaluation is based on a technical study and U is used if it is based on experience.

PCN	Pavement Type	Subgrade Category	Tire Pressure Category	Evaluation Method
Numerical Value	R-rigid F-flexible	A-CBR>13 K>400pci	W-No limit	T-Technical
		B-CBR 8-13 K 201-400 pci	X-up to 217 psi	U-Experience
		C-CBR 4-8 K 100-200 pci	Y-up to 154 psi	
		D-CBR<4 K<100 pci	Z-up to 73 psi	

Figure 4-24. PCN Coding Elements.

Category	Allowable Tire Pressure Code	Marshall Stability Range (pounds)
High	W	2000 +
Medium	X	1250-2000
Low	Y	750-1250
Very Low	Z	< 750

Figure 4-25. Allowable Tire Pressure Based on Marshall Stability.

4.7.1 Mechanistic PCN Method

In the mechanistic method, the allowable design aircraft load is determined based on deflection measurements from the FWD. The allowable load is then converted to a PCN value. There are several mechanistic methods available; the one used here for illustration is based on the U.S. Army Corps of Engineers PCASE program (PCASE 2004). Following is a description of the procedure.

Determine the elastic modulus for each layer:

1. Analyze the measured deflection basins from the FWD for the pavement section under consideration. Using the layered elastic program, estimate the different layer moduli for each basin. Use the estimated moduli to determine the average modulus for each layer. Another approach is to select a representative deflection basin for the pavement section and use the representative basis to estimate the elastic modulus for each layer.

2. Determine the design aircraft and equivalent number of passes. In PCASE, the design aircraft is determined as the aircraft among the aircraft mixture using the pavement which requires the greatest pavement thickness to support its projected passes. The number of passes of the design aircraft required to produce an equivalent effect on the pavement as the mixture of traffic is the design pass level. The procedure for determining the design aircraft and equivalent number of passes is as follows:

    a. Determine the total thickness for each aircraft at its projected pass level. The aircraft requiring the greatest thickness is designated as the design aircraft.

    b. Determine the allowable number of passes for each aircraft for the maximum required thickness.

    c. Determine the design passes in terms of the design aircraft as follows:

$$\begin{matrix}\textit{Design Passes in terms} \\ \textit{of Design Aircraft}\end{matrix} = \sum_{i=1}^{n} \frac{\textit{Projected Passes for Aircraft}_i}{\textit{Allowable Passes for Aircraft}_i}$$

3. Compute allowable design aircraft load. The allowable design aircraft load is computed for the existing pavement structure using the aircraft gear configuration and design pass level. The PCASE procedures for flexible and rigid pavements are described separately as follows:

For Flexible Pavements

a. The design pass level is converted into coverages.

b. The coverages and the AC stiffness modulus are used to calculate the allowable strain at the bottom of the AC layer using Figure 4–26 or the following equation:

$$ALLOWABLE\ STRAIN_{AC} = 10^{-A}$$

where

$ALLOWABLE\ STRAIN_{AC}$ = allowable tensile strain at the bottom of the asphalt layer, inches/inches

$$A = \frac{N + 2.665 LOG_{10}\left(\dfrac{E_{AC}}{14.22}\right) + 0.392}{5}$$

$N = LOG_{10}$ (aircraft coverages)

E_{AC} = AC modulus, pounds per square inch

c. The allowable tensile strain at the bottom of the AC layer pavement structure and gear configuration are used in the elastic layer program to compute the allowable aircraft load.

d. The coverage and subgrade modulus (E_{SG}) are used to calculate the allowable vertical strain at the top of the subgrade using Figure 4–27 or the following equation:

$$ALLOWABLE\ STRAIN_{SG} = \left(\frac{10{,}000}{N}\right)^{1/B} \times A$$

where

$ALLOWABLE\ STRAIN_{SG}$ = allowable vertical strain at the top of the subgrade, inches/inches

N = aircraft repetitions (passes)

$A = 0.000247 + 0.000245\ LOG(E_{SG})$

$B = 0.0658\,(E_{SG})^{0.559}$

E_{SG} = subgrade modulus, pounds per square inch

e. The allowable vertical strain at the top of the subgrade, pavement structure, and gear configuration are used in the elastic layer program to compute the allowable aircraft load.

f. The lowest allowable aircraft load for steps ‘c’ and ‘e’ above is reported.

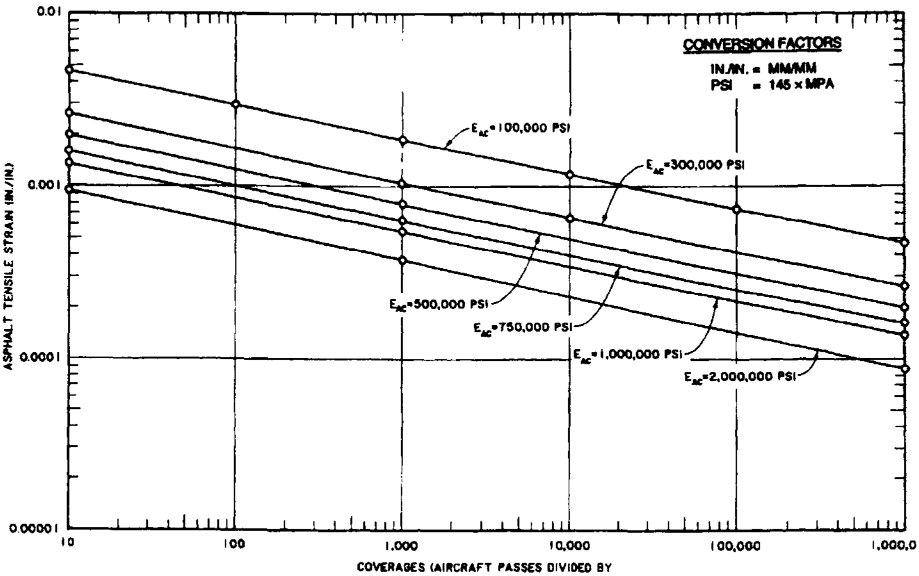

Figure 4-26. Allowable Tensile Strain at the Bottom of the Asphalt Layer (UFC 2001)

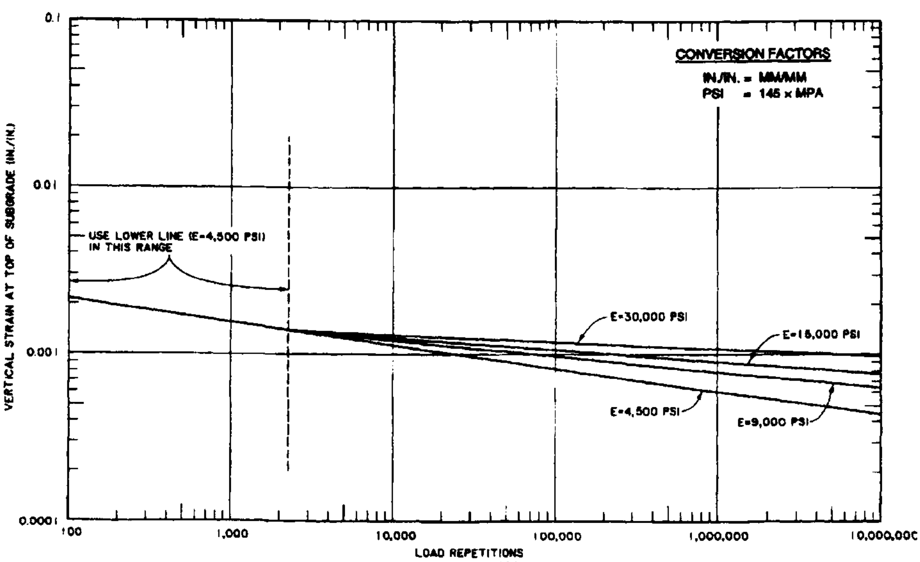

Figure 4-27. Allowable Vertical Strain at the Top of the Subgrade (UFC 2001)

For Rigid Pavements

a. The design pass level is converted into coverages.

b. A desirable (design) Structural Condition Index (SCI) is selected. The SCI is a distress index that is calculated similar to the PCI but with limited distresses as described in Chapter 3. An SCI of 80 corresponds to the formation of one or more cracks per slab in 50% of the trafficked slabs. However, experience has indicated that an SCI of 80 is somewhat conservative; a value of SCI of 50 is recommended.

c. The aircraft coverages and design SCI are used to calculate the Design Factor (DF) from Figure 4–28 or the following equation:

$$DF = A + B\ LOG\ C$$

where

DF = design factor

A = 0.2967 + 0.002267 (SCI)

B = 0.3881 + 0.000039 (SCI)

C = coverage level at selected SCI

SCI = Structural condition index

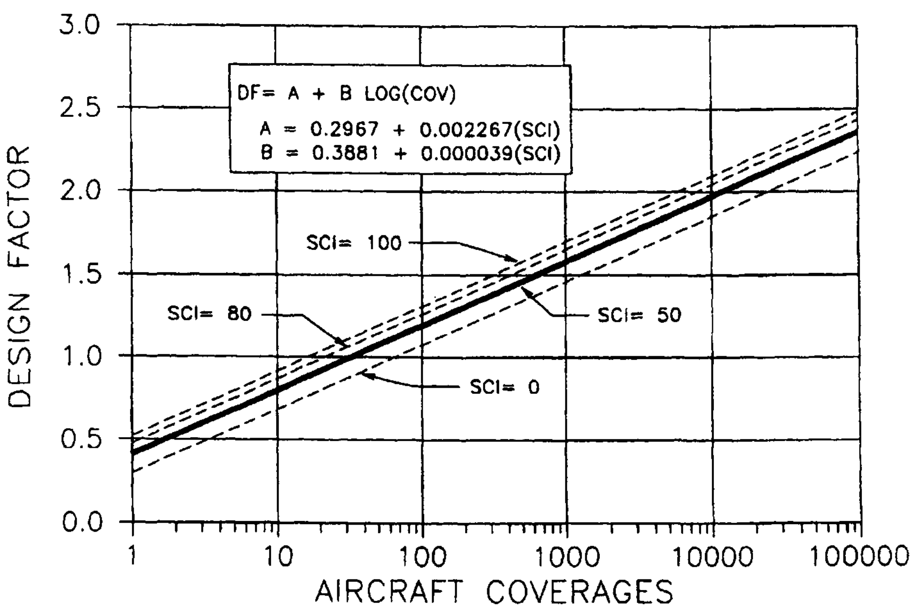

Figure 4-28. Design Factor, Based on Aircraft Coverages and Design SCI (UFC 2001)

d. The allowable tensile stress at the bottom of the slab is calculated from the following equation:

$$ALLOWABLE\ STRESS_{PCC} = \frac{R}{DF}$$

where

$ALLOWABLESTRESS_{PCC}$ = allowable tensile stress at the bottom of the slab in pounds per square inch

R = PCC flexural strength in pounds per square inch

e. The allowable tensile stress at the bottom of the slab, pavement structure, and aircraft gear configuration are used in the layer elastic program to compute the allowable aircraft load at the slab center.

f. The allowable load determined at the slab center is reduced for poor joint transfer using a load reduction factor as shown in Figure 4–29. The load reduction factor is function of the deflection ratio (Fig. 4–20) across the slab joints, and it ranges between 0.75 and 1.00.

g. The reported allowable aircraft load is the allowable load at the slab center (step 'e') multiplied by the load reduction factor (step 'f').

4. The allowable aircraft load is converted to PCN. It should be noted that the PCN is the load carrying capacity of the pavement in terms of a single wheel load at a tire pressure of 181 psi (1.25 MPa) reported in Kg and multiplied by 2.0. The conversion is a function of the pavement type, gear configuration, and subgrade strength category. PCASE calculates the PCN from the determined allowable load. Conversion charts are also available in the FAA Advisory Circular 150/5335-5 (FAA 1983).

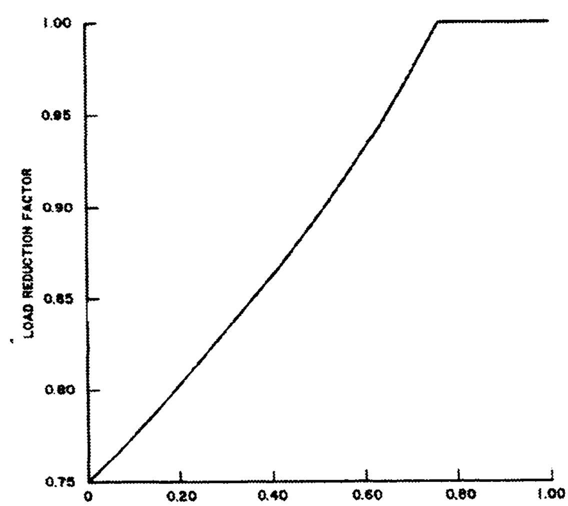

Figure 4-29. Load Reduction Factors for Load-Transfer Analyses (UFC 2001).

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5

Roughness Measurement and Analysis

5.1 Background and Definitions

Roughness is an important indicator of pavement riding comfort and safety. From an auto driver's point of view, rough roads mean discomfort, decreased speed, potential vehicle damage, and increased operating cost. From an aircraft pilot's viewpoint, airfield pavement roughness can cause discomfort, vibration of the instrument panel, and potential danger both to the aircraft and its passengers. Therefore, roughness is a condition indicator that should be carefully considered when evaluating primary pavements.

The use of roughness measurements in pavement management has been demonstrated at both the network and project levels. At the network level, roughness is used for dividing the network into uniform sections, establishing value limits for acceptable pavement condition, and setting maintenance and rehabilitation (M&R) priorities. Several agencies combine the roughness condition index with other pavement condition indices, such as distress, to formulate a composite index that is used for various management activities. It is important to realize, however, that roughness is a measure of user comfort and a safety indicator, but by itself it is not necessarily a good indicator of the overall need for M&R. Also, roughness indicators may not correspond to the best economic timing for performing major rehabilitation. For example, an asphalt pavement that is structurally deficient and showing low-severity alligator cracking can still provide a good level of riding comfort. Nevertheless, such a pavement should be scheduled for immediate detailed project evaluation and structural rehabilitation before the optimum time for rehabilitation has passed.

At the project level, roughness measurements are used to locate areas of critical roughness and to maintain construction quality control. Areas of critical roughness can be identified by examining a plot of the roughness measurement (index) against distance. Construction quality control is attained by specifying acceptable roughness limits. As will be shown later, the limits should be tied to specific analysis techniques. Roughness measurement is neither necessary nor economical for all project analysis. For example, when a pavement is to be reconstructed, roughness measurements are of no value except for record keeping.

5.1.1 Definition of Roughness

Pavement roughness consists of random multi-frequency waves of many wavelengths and amplitudes. Longitudinal roughness has been defined as "the longitudinal deviations of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality and dynamic pavement load" (Janoff et al. 1975). Pavement profiles, detailed recordings of surface characteristics, are frequently used to characterize roughness. Different wavelengths will have different effects on ride quality depending on vehicle characteristics and driving speed. For example, wavelengths over 100 ft on highways will have little effect on vehicle ride, but on runways, wavelengths up to 400 feet may be significant.

5.1.2 Factors Contributing to Roughness

There are several causes of pavement roughness: traffic loading, environmental effects, construction materials, and built-in construction irregularities. All pavements have irregularities built into the surface during construction, so even a new pavement that has not been opened to traffic can exhibit roughness. The roughness of a pavement normally increases with exposure to traffic loading and the environment.

Short-wavelength roughness is normally caused by localized pavement distresses, that is, depressions and cracking. Roughness may be further aggravated by traffic. For example, corrugation can cause an increase in dynamic wheel force which in turn can increase the severity and roughness of that corrugation.

Long-wavelength roughness is normally caused by environmental processes in combination with pavement layer properties. Poor drainage, swelling soils, freeze-thaw cycles, and nonuniform consolidation of subgrade may all contribute to surface roughness. Warping and curling of long concrete slabs will also cause roughness.

5.1.3 Roughness Measuring Systems

Several roughness indices are currently in use by highway and airport agencies. These indices are based either on pavement surface profile or on output from a roadmeter installed in a vehicle. The latter is known as a response type road roughness measuring system (RTRRMS). Response-type indices are vehicle dependent and are not repeatable—even when the same vehicle is used—due to change in the vehicle's characteristics over time. Profile-based indices, on the other hand, are repeatable because they are based on mathematical modeling of the measured profile. Response-type indices can be made more useful by calibrating them with a repeatable index—most likely a profile-based one. The use of profile indices has not been very popular in the past due to the historical cost of profile measuring devices. This is no longer the case, however, and more agencies are acquiring profile measuring systems.

5.2 Profile-Roughness Measuring Systems

A profile-roughness measuring system involves measuring the profile, filtering the profile to include only those waves of interest, and mathematically computing some type of a roughness index by modeling the response of a standard vehicle or by evaluating the amplitude properties of selected wave bands.

5.2.1 Profile Filtering

A profile consists of different wavelengths, varying from a few inches to hundreds of feet. To analyze a profile for roughness, it is important that the profile be filtered to include only those waves of interest. This can be achieved using a technique called the "moving average filter," (Sayers and Gillespie 1986). This technique smooths the profile at each point by averaging the elevation over a selected baselength as shown in Figure 5–1. The equation for calculating the smoothed elevation at each point is as follows:

$$y_s(i) = [1/(2k+1)]\sum_{j=i-k}^{i+k} y_r(j) \tag{5-1}$$

where

$y_r(j)$ = unfiltered "raw" vertical profile elevation for sample j

$y_s(i)$ = smoothed profile elevation for sample i

k = number of samples in half of the moving average baselength

b = z.k.dx = baselength of moving average

dx = distance between samples

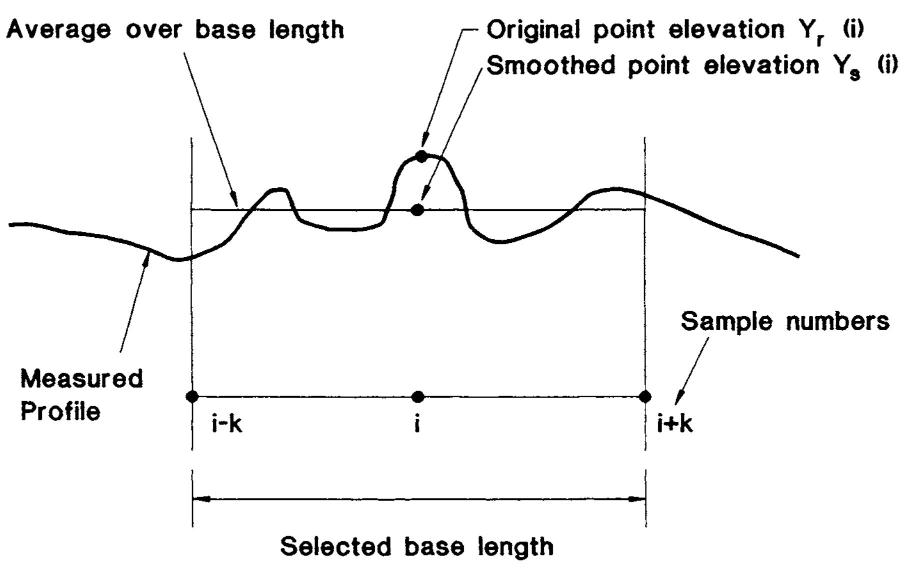


Figure 5-1. Moving Average Filter.

In order to have a true moving average, it is recommended that a minimum k value of 4 be used. The longer the baselength, the higher the value of k should be. Equation 5–1 can be expressed differently to facilitate computations as follows:

$$y_s(i) = y_s(i-1) + [1/(2k+1)][y_r(i+k) - y_r(i-k-1)] \qquad (5\text{-}2)$$

Since Eq. 5–2 is recursive, the first value must be calculated using Eq. 5–1, but the remainder of the values are more efficiently calculated using Eq. 5–2.

A moving average filters out short wavelengths (high frequencies), leaving the longer wavelengths (low frequencies), and thereby smooths the signal A filter that removes high frequencies is called a low-pass filter. For example, to smooth out waves shorter than 1 ft, a 1-ft baselength is selected. A low-pass filter is applied using Equations 5–1 and 5–2, and a smooth signal is produced.

When analyzing road profile, it is usually desirable to remove the long wavelengths, leaving the roughness associated with the shorter waves. In other words, a "high-pass filter" is needed. The moving average is converted from a low-pass filter to a high-pass filter by subtracting the smoothed signal from the original signal (Sayers and Gillespie 1986), as shown below:

$$y_h(i) = y_r(i) - y_s(i) \qquad (5\text{-}3)$$

where

y_h = "high-pass filter" elevation, with y_r and y_s as defined in Equation 5–1.

For example, to eliminate waves larger than 100 ft, a low-pass filter is first applied using a baselength of 100 feet. The smoothed signal is then subtracted from the original signal using Equation 5–3. Figure 5–2 illustrates unfiltered and filtered profiles.

5.2.2 Profile-Based Indices

5.2.2.1 International Roughness Index (IRI)

In 1982, the International Road Roughness Experiment (IRRE) was held in Brazil (Sayers, Gillespie, and Queiroz 1986). The purpose of this experiment was to develop an international roughness index (IRI) for exchanging data, and to publish guidelines for measuring roughness on a standard scale.

The IRI is a standard roughness measurement related to measurements obtained by road meters installed on vehicles or trailers. The IRI is a mathematical model applied to a measured profile. The model simulates a quarter-car system (QCS), shown in Figure 5–3, traveling at a constant speed of 80 km/hr. The IRI is computed as the cumulative movement of the suspension of the QCS divided by the traveled distance. The IRI scale is shown in Figure 5–4. The IRI is sensitive to wavenumbers of between 0.01 and 0.23 cycles per feet (0.03 and 0.75 cycles per meter, Fig. 5–5), which corresponds to wavelengths of between 100 and 4 ft (wavelength = 1/wavenumber). However, there is still some response for wavelengths outside this range.

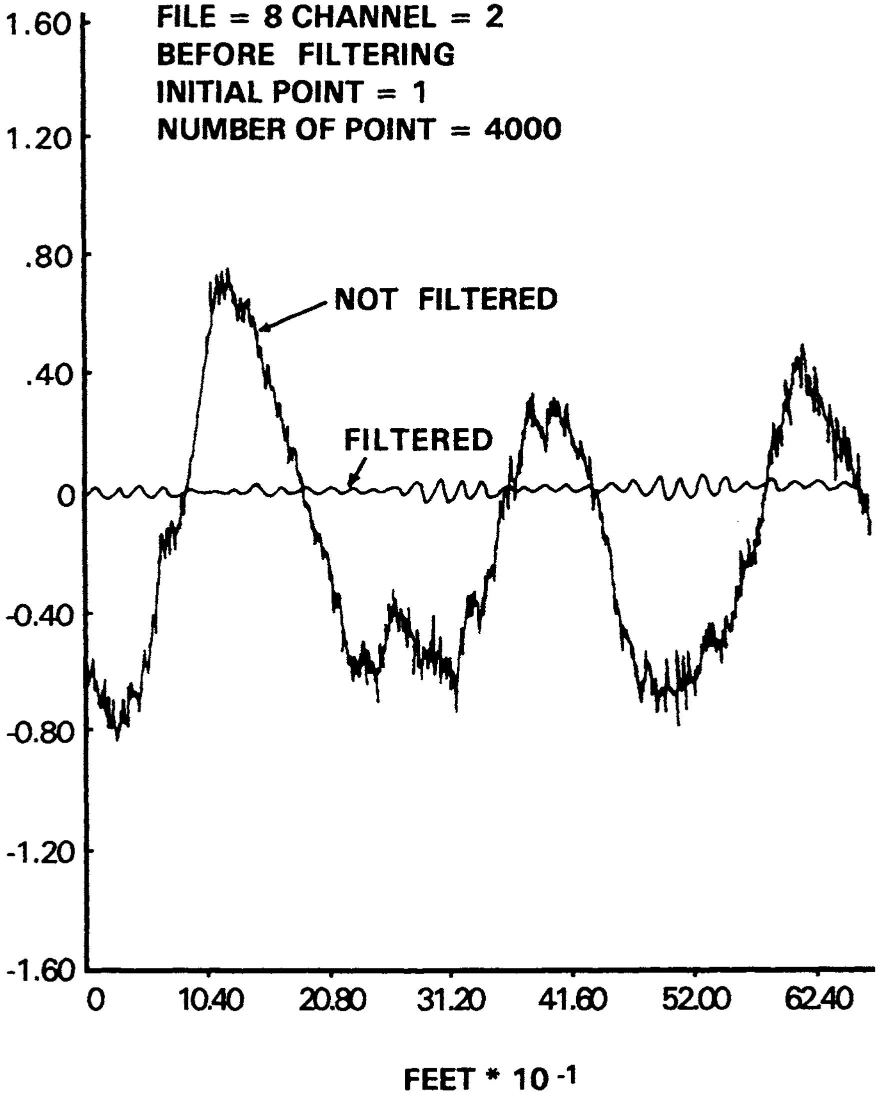


Figure 5-2. Illustration of Effect of Digital Filtering of a Profile.

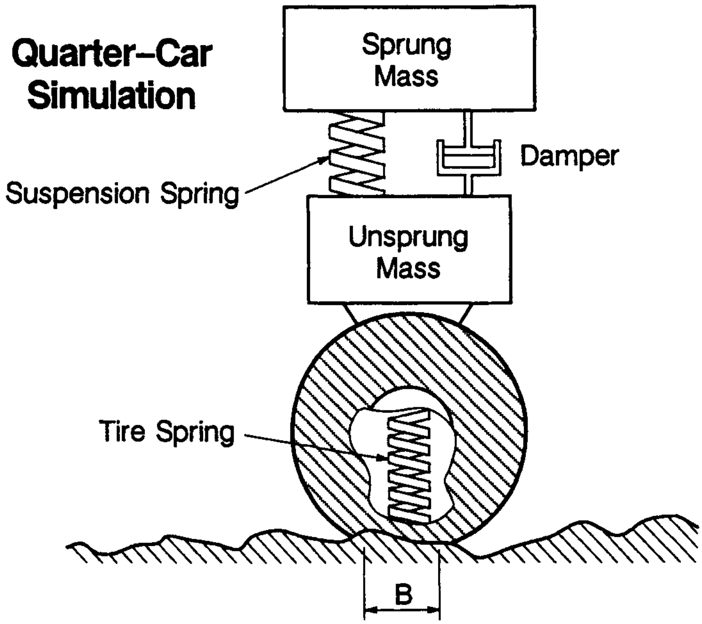


Figure 5-3. Quarter-Car Model Used as the Basis of the International Roughness Index.

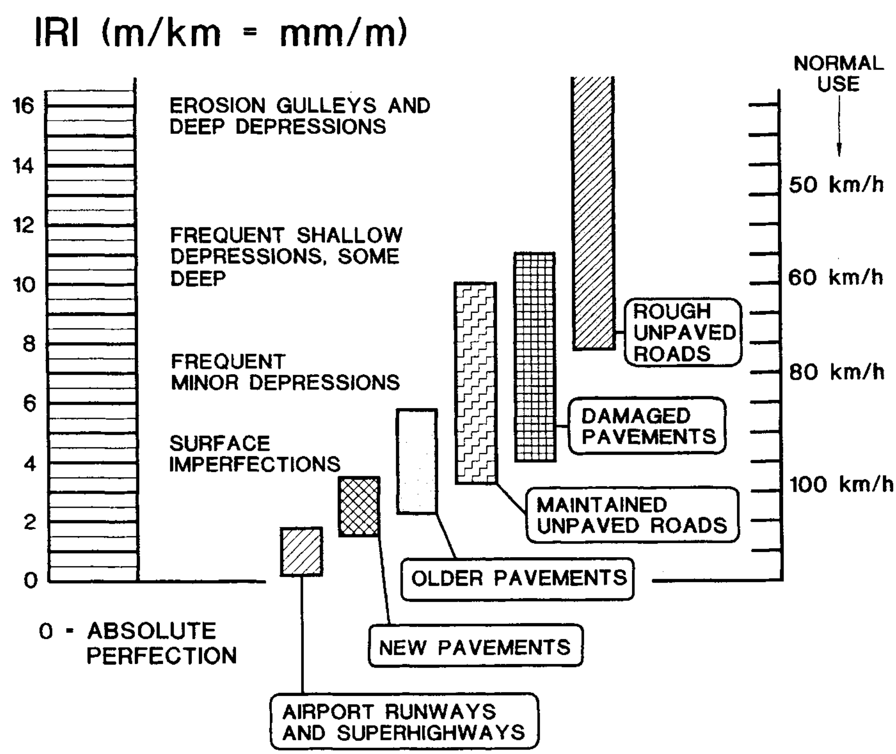


Figure 5-4. The IRI Roughness Scale.

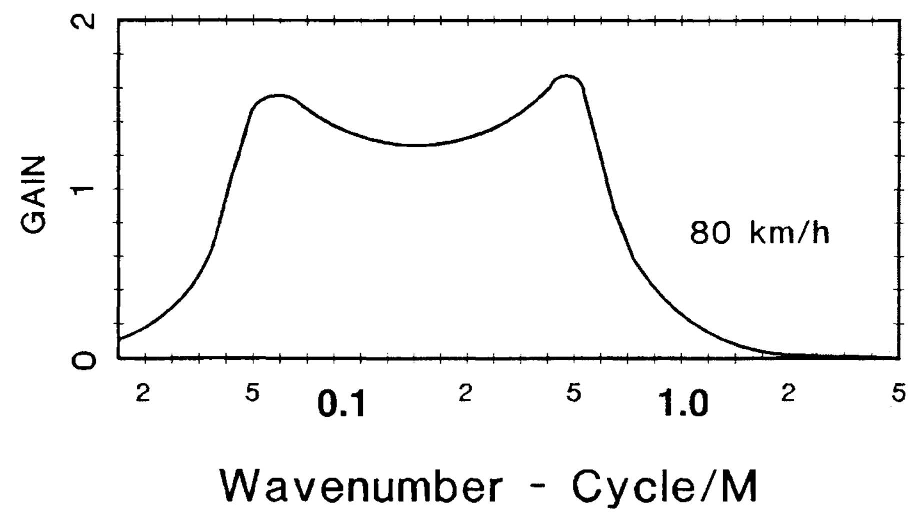


Figure 5-5. Frequency and Wavenumber Sensitivities of the IRI Analysis.

5.2.2.2 Half-Car Roughness Index (HRI)

The HRI is the roughness index obtained when the IRI is calculated for the average of the left and right wheel path profiles. This is different than first calculating the IRI for each wheel path and then averaging it. The HRI was shown to be highly correlated to the average IRI of both wheel paths (HRI = 0.89 IRI), (NHI 2001). There is little or no additional information provided by the HRI over the IRI.

5.2.2.3 Ride Number (RN)

The RN was developed (NHI 2001) to agree with mean panel ratings (MPR) of road roughness using a scale from 0 to 5 where 5 is a perfect ride. This scale was selected to be similar to the present serviceability index (PSI) that was introduced during the American Association of State Highway & Transportation Officials (AASHTO) road test, (Highway Research Board 1962). The RN is a nonlinear transform of a longitudinal profile statistic called PIC (profile index), using the equation below:

$$RN = 5e^{-160(PI)} \qquad (5\text{-}4)$$

Figure 5–6 shows the sensitivity of the PI for a slope sinusoid (NHI 2001). If given a sinusoid as input, the PI filter produces a sinusoid output. The amplitude of the output sinusoid is the amplitude of the input, multiplied by the "gain" shown in the figure. The maximum sensitivity is for a wavelength of about 6.1 meters (20 ft). This is different from the IRI which has a sensitivity to the wavelengths of 2.4 m (7.9 ft) and 15.4 m (50.5 ft).

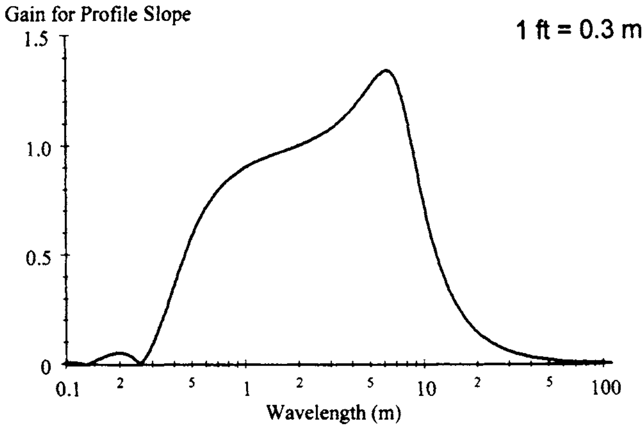


Figure 5-6. PI Fora Slope Sinusoid.

Therefore it is important to recognize that the content of a road profile that affects RN is different than the content that affects IRI, (NHI 2001). The RN can be calculated from either left or right wheel paths or both. If both wheel paths are used, each profile is processed independently to calculate the PI, and the results are combined as follows:

$$PI = \sqrt{\frac{PI_L^{\,2} + PI_R^{\,2}}{2}} \qquad (5\text{-}5)$$

The RN is then computed using Equation 5–4.

5.2.2.4 Root Mean Square of Vertical Acceleration (RMSVA)

The RMSVA is the root mean square (RMS) of a variable called vertical acceleration (VA) associated with the rate of change of slope. This variable is shown in Figure 5–7 and calculated as follows:

$$VA = [y_{x-b} + y_{x+b} - 2y_x]/b^2 \qquad (5\text{-}6)$$

where

y_x = profile elevation at point x

b = baselength

The VA variable is effectively equivalent to the mid-cord deviation (MCD) shown in Figure 5–8, which is determined as follows:

$$\begin{aligned} MCD &= [(y_{x-b} + y_{x+b})/2 - y_x] \qquad (5\text{-}7) \\ &= [y_{x-b} + y_{x+b} - 2y_x]/2 \end{aligned}$$

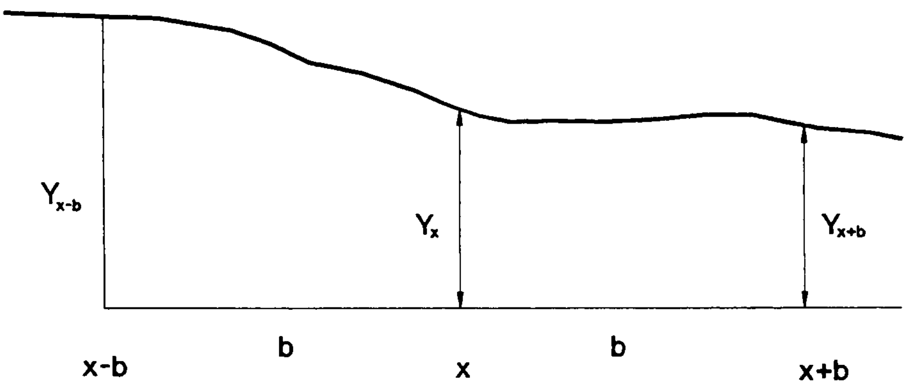


Figure 5-7. Root Mean Square of Vehicle Acceleration (RMSVA).

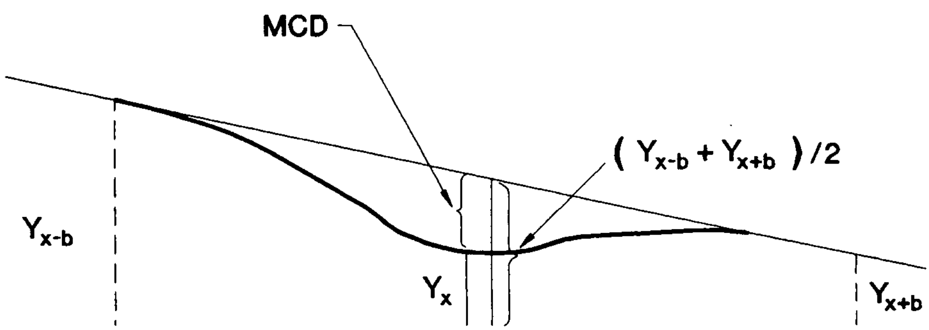


Figure 5-8. Mid-core Deviation.

By comparing Equations 5–6 and 5–7, it is noted that VA equals an MCD multiplied by $2/b^2$. Because digital profiles are normally measured at a constant interval dx, Equation 5–6 can be rewritten using sample number rather than longitudinal distance (Sayers and Gillespie 1986) as follows:

$$\mathrm{VA}(i) = [y_{i-k} + y_{i+k} - 2y_i]/(k \cdot dx)^2 \qquad (5\text{-}8)$$

where

i = sample number

k = integer used to define baselength

dx = distance between samples

b = baselength, calculated as $K \cdot dx$

Because the VA value at point *i* is calculated using profile elevation at distance *b* before *i* and distance *b* after *i*, the VA cannot be calculated for the first and last *b* distances of a measured profile.

The RMSVA is calculated as follows:

$$\text{RMSVA} = [1/(n-2k)] \cdot \left[\sum_{i=k+1}^{n-k} \text{VA}_{(i)}^{2} \right]^{1/2} \tag{5-9}$$

The RMSVA has the unit of 1/length, which is appropriate for spatial acceleration (i.e., ft/ft^2). To convert the RMSVA to time acceleration units (such as ft/sec^2), the RMSVA is multiplied by the velocity squared (assuming that the velocity is constant) as follows:

$$\text{RMSVA}(\text{time}) = v^2\ \text{RMSVA}(\text{distance}) \tag{5-10}$$

where

v = travel velocity

The value of the RMSVA depends on the value of the baselength, *b*, in relation to the roughness wavelength. As shown in Figure 5–9, the value is maximum for a wavelength of 2*b* and 0 for a wavelength of *b*.

A single baselength, therefore, will not produce an RMSVA that correlates well with road meter measurements. Typically, the RMSVA for at least two baselengths are combined.

In Texas, baselengths of 4 ft and 16 ft are combined to arrive at an index known as "MO" (Sayers and Gillespie 1986):

$$\text{MO} = -20 + 23 \times \text{RMSVA}_4(\text{time}) + 58 \times \text{RMSVA}_{16}(\text{time}) \tag{5-11}$$

Equation 5–11 assumes that RMSVA are in units of ft/sec^2. The MO index was developed to correlate with measurements from the Mays meter in inches per mile.

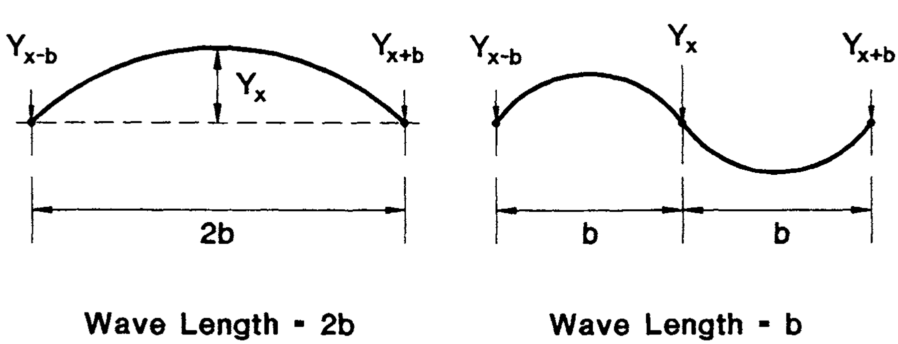


Figure 5-9. RMSVA Relationship to Baselength and Roughness Wavelength.

5.2.2.5 Waveband Indices

Waveband analysis is used to reduce a road profile to several indices, each quantifying roughness over a given waveband (range of wavelengths). Roughness can be analyzed over short, medium, and long wavelengths.

A function commonly used for this type of analysis is known as the power spectral density (PSD). The PSD of a variable is the mean square of that variable per wavenumber (wavenumber = 1/wavelength = cycles/distance).

Figure 5–10 (Sayers and Gillespie 1986) shows the PSD for profiles of typical roads defined in terms of elevation, slope, and acceleration. The figures show the amplitude of the three variables plotted against wave number in cycles per meter. Among the three, variable slope is preferred as an indicator of the relative contribution of different wavelengths to roughness because of the more narrow range of amplitudes over the frequency range of interest. In the case of elevation, amplitudes are normally larger at low wave numbers, making it difficult to detect the contribution of different wavelength to roughness. This makes it necessary for measuring instruments to cover a wide dynamic range (4 orders of magnitude.)

5.2.2.6 Slope Variance

The slope variance (SV) is a roughness index based on the changes in the slope of a pavement profile. It is described by the following equation:

$$\mathrm{SV} = \sum\left(X_i - \overline{X}\right)^2 / (n-1) \tag{5-12}$$

where

SV = slope variance

X_i = i^{th} slope measurement

n = number of slope measurements

$\overline{X}$ = mean slope measurement

The slopes of the lines between consecutive points are determined over the section of pavement to be analyzed. The spacing of these points will affect the slope variance with closer spacing producing a higher slope variable. This indicator was first determined at the AASHTO Road Test with the profilometer device shown in Figure 5–11. The profilometer recorded the angle (A) formed by the line of the support wheels (G and H) and the line CD connecting the two wheels (E) spaced at 9 in. Slope variance calculation yields only a single statistic to characterize a pavement section. Long-wavelength roughness cannot be detected readily by this method, so it is considered not acceptable for analyzing airport pavements.

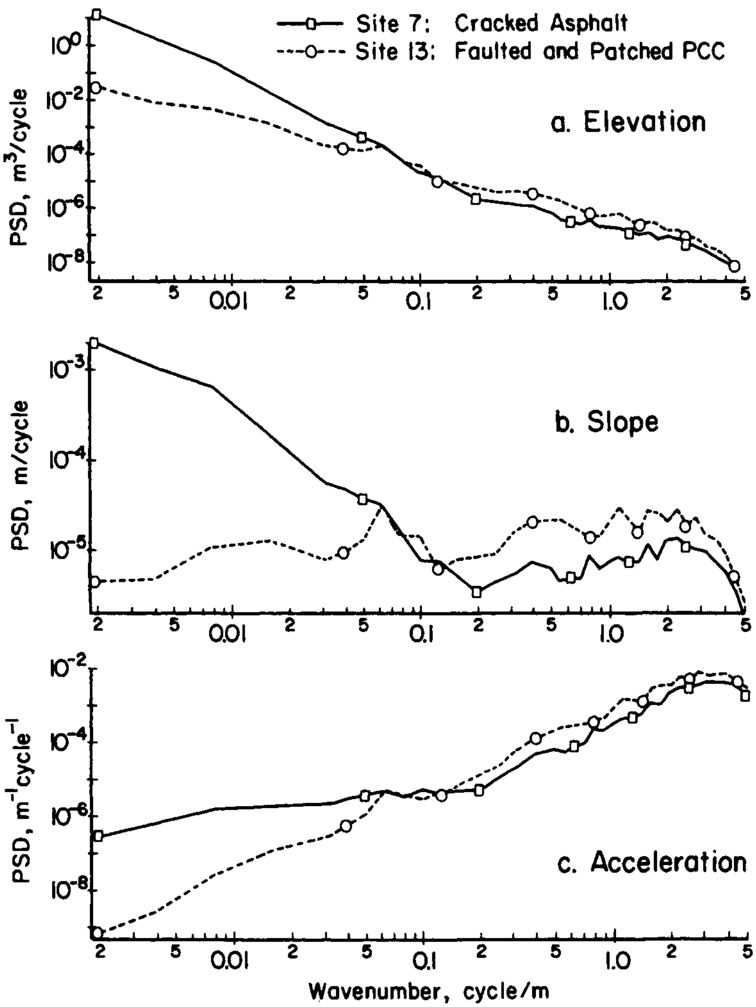


Figure 5-10. Power Spectral Density (PSD) Functions for Two Sites.

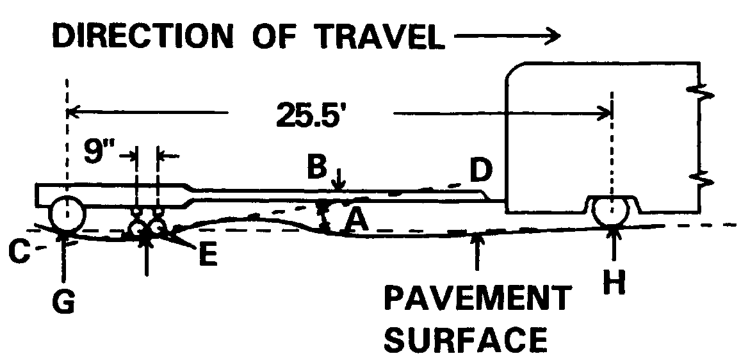


Figure 5-11. Schematic of Profilometer Used to Determine Slope at AASHTO Road Test.

5.2.2.7 Acceleration

Acceleration is another indicator of pavement roughness. Accelerations impose forces on motor vehicle passengers that contribute to discomfort. (Frequency, exposure time, etc., also contribute to discomfort). Accelerations can also cause structural and vehicle instrumentation damage. Gerardi (1991) reported that for aircraft, an acceleration of 0.4 g can be used as a tolerance limit. Figure 5–12 shows a sample vertical acceleration plot for an aircraft developed using the computer program TAXIM developed by T. Gerardi.

An acceleration-versus-distance plot along a pavement can help locate areas that cause extreme acceleration, so they can be smoothed out. This technique has been used with increasing success in measuring airport pavement roughness. Several computer programs have been developed to model the aircraft/runway profile interaction.

5.2.3 Profile Measuring Equipment

5.2.3.1 High Speed Inertial Profilers

High Speed Inertial profilers record the true profile of a pavement at highway speeds. A schematic diagram of an inertial profiler is shown in Figure 5–13. The principal components of an inertial profiler are;

1. *Height Sensor* - records the vehicle height from the pavement surface.
2. *Accelerometer* - records vertical acceleration of the vehicle. The acceleration is then double integrated with respect to time to obtain the vehicle vertical displacement.
3. *Distance Measuring System* - measures distance from a reference starting point.
4. *Computer Hardware/Software* - computes profile at each sampling point using information from the height sensor, accelerometer, and distance measuring system. The difference between the reading from the height sensor and the vehicle critical displacement provides the profile elevation. The profile is then processed to computer profile indices such as the IRI and RN.

The height sensor types that are commonly used in profilers are either laser or infrared. Previously used height sensors (in the 1980's and 1990's) include ultrasonic and optical types, but are no longer used due to associated problems.

The first high-speed inertial profiler was developed by Elson Spangler and William Kelley at the General Motors Research Corporation. The first inertial profiler was commercially manufactured by K.J. Law Engineers. (NHI 2001).

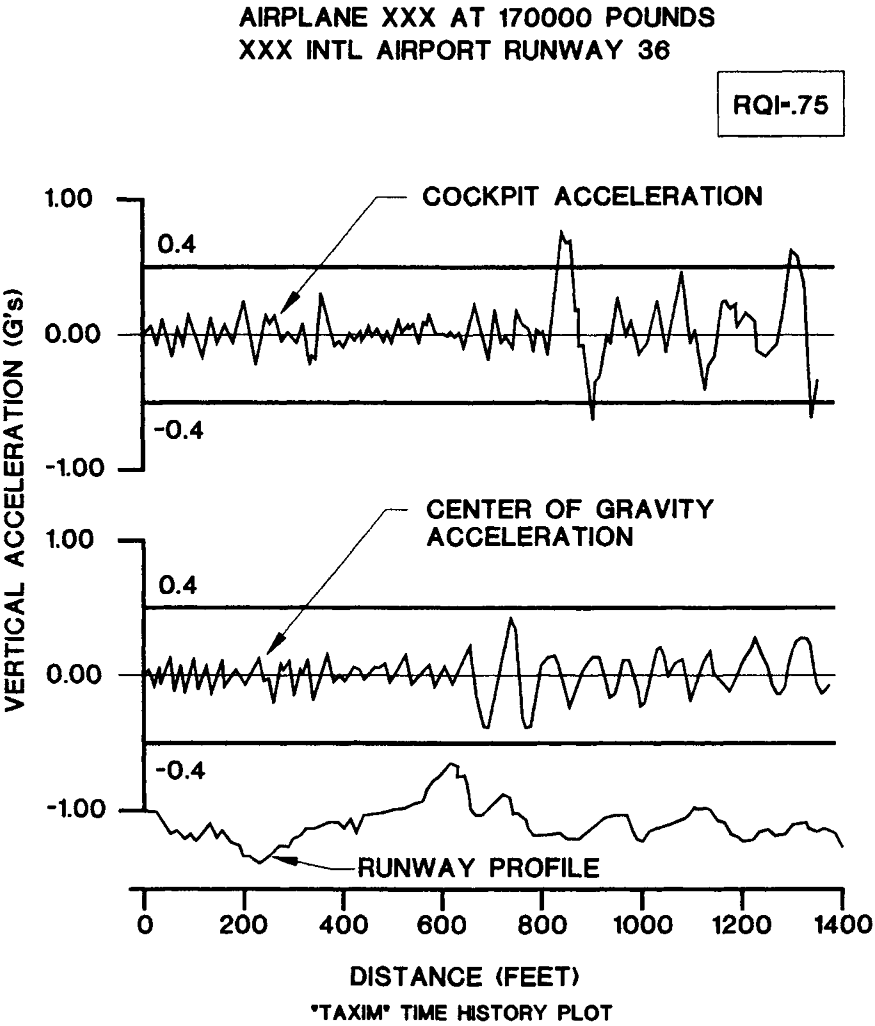


Figure 5-12. TAXIM Time History Plot (from APR Consultants, Inc., 1994).

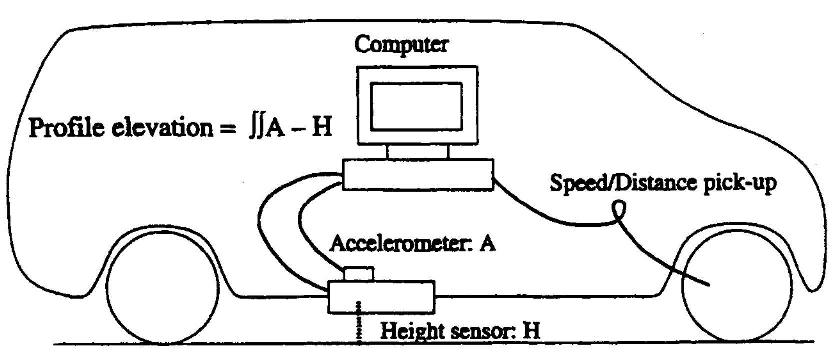


Figure 5-13. A Schematic Diagram of an Inertial Profiler.

In the 1980's, the South Dakota Department of Transportation designed and built its own high-speed inertial profiler that used ultrasonic height sensor and was adopted by several highway agencies. Profiler Manufactures include Ames Engineering (Ames 2004), Dynatest (Dynatest 2004), International Cybernetics Corporation (ICC 2004), and Surface Systems & Instruments (SSI 2004). Dynatest has acquired the Transportation Testing Equipment Division (TTE) of K.J. Law Engineers (KJ Law 2004). Figure 5–14 shows a bottom view of the Dynatest Road Surface Profiler (RSP). Figure 5–15 shows the data collection screen.

5.2.3.2 Lightweight Profilers

This system is similar to the high-speed inertial profilog system except the equipment is installed in a lightweight utility vehicle. It is used primarily for profiling new pavements. Its light weight (Fig 5–16) and low tire pressure makes it ideal for newly constructed concrete pavements. The profile recorded can be used for calculating profile-based roughness indices such as IRI, Profilograph PI, and determination of must grind quantities and location. Most of the vendors who manufacture high-speed profilers also manufacture lightweight profilers.

5.2.3.3 Reference Profilers

5.2.3.3a Rod and Level

This is conventional surveying equipment consisting of a precision rod, a level for establishing the horizontal datum, and a tape to mark the longitudinal distance for elevation measurement.

5.2.3.3b Dipstick Auto-Read Road Profiler

The Dipstick Profiler is shown in Figure 5–17. The operator walks the Dipstick along a survey line, alternately pivoting the instrument about each of its supporting legs. The elevation difference between the Dipstick's two legs is displayed and automatically recorded. The Dipstick has an accuracy of 0.0015 in (or 0.15 mm for the metric model) per reading. Data analysis for IRI computations is computerized and a continuous scaled plot of surface profile can be printed.

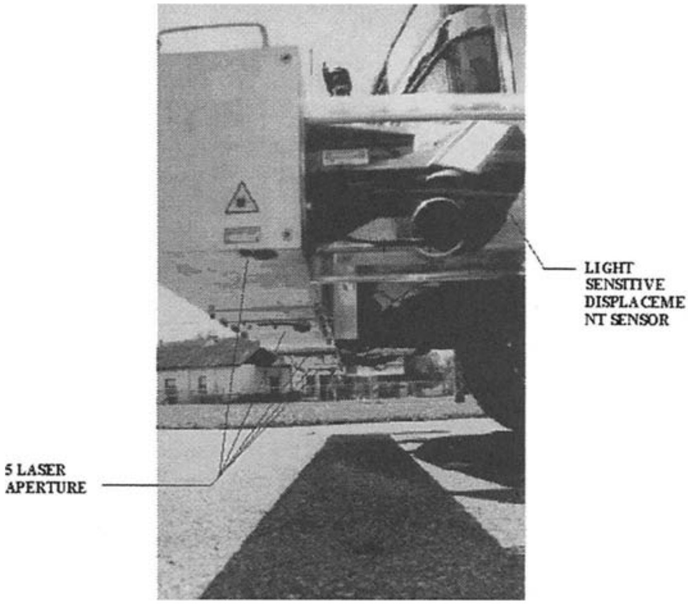


Figure 5-14. Bottom view of Dynatest Road Surface Profiler (RSP) (Dynatest 2004).

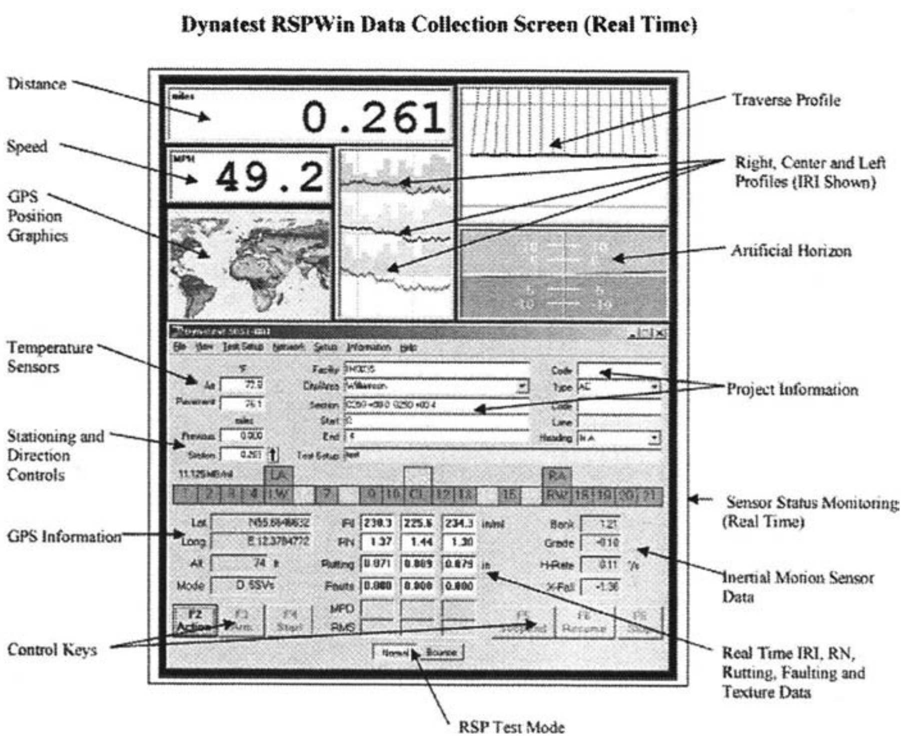


Figure 5-15. Data Collection Screen (Dynatest 2004).

Figure 5-16. Lightweight Profiler (Dynatest 2004).

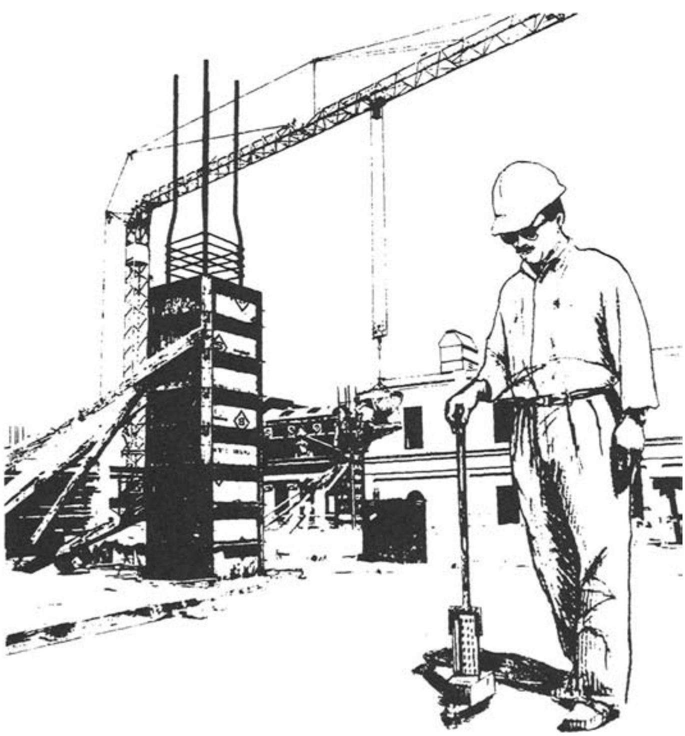


Figure 5-17. Dipstick Auto-read Profiler (FACE 2004).

5.3 Untrue Profile Measuring Systems

5.3.1 Vehicular Response-Roughness Measuring Systems

5.3.1.1 System Descriptions

These systems consist of a vehicle (or trailer), the driver and the contents of the vehicle, and a device called the road meter, which measures vehicle response to roughness. The road meters are either mounted on the body of a passenger car above the center of the rear axle, or on the frame of a single-wheel trailer. Most road meters measure the relative movement of the axle with respect to the vehicle body, but some measure the vertical acceleration of the vehicle body or axle. Measurements reported from these systems are normally in units of inches per mile or meters per kilometer, which is the cumulative movement of the suspension between the axle and the body divided by the traveled distance. These systems are also referred to as RTRRMSs, which stands for response-type road roughness measurement systems (Sayers, Gillespie, and Paterson 1986; Sayers and Gillespie 1986; Sayers, Gillespie, and Queiroz 1986).

5.3.1.2 Response Type Device Examples

One example of a car-mounted system, the Mays' Ride Meter (MRM), has received more acceptance by highway agencies than any other device in the response type category. Manufactured by the Rainhart Company of Austin, Texas, the MRM was initially developed by Ivan Mays to provide a simple method of obtaining roughness data. The MRM consists of a transducer mounted in the rear of the vehicle and a strip-chart that is usually placed in the front seat as shown in Figure 5–18. Figure 5–19 shows a typical MRM output chart. The top trace (distance) automatically zigs for 0.05 miles and zags for 0.05 miles, recording information generated by an odometer. This is independent of the chart feed, which advances at the rate of 1/64 in. for every 0.1 in. of rear axle/body excursion. Hence, the length of paper produced when divided by the known distance traveled is a meaningful index of roughness. The center trace (profile) follows the rear axle excursions in the same direction, but at half the magnitude. The bottom trace (landmarks) alternately zigs and zags at the touch of a button to mark an event such as the beginning or end of a bridge.

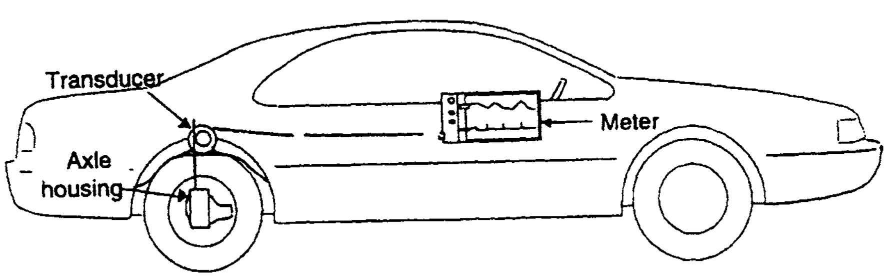
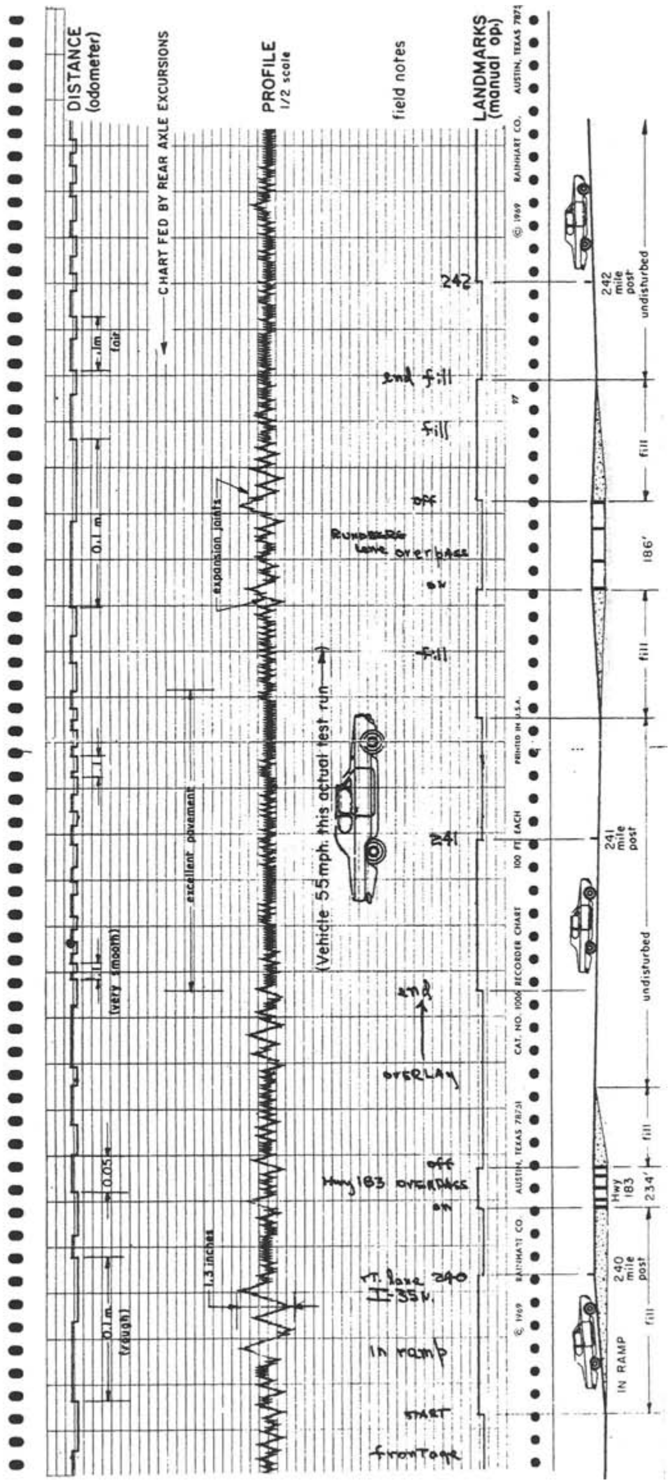


Figure 5-18. Mays' Ride Meter (MRM).


DISTANCE (odometer)
CHART FED BY REAR AXLE EXCURSIONS
PROFILE 1/2 scale
field notes
LANDMARKS (manual op.)
0.1 m (rough)
0.05
(very smooth)
.1m fair
0.1 m
1.3 inches
excellent pavement
expansion joints
(Vehicle 55mph. this actual test run)
IN RAMP
fill
240 mile post
Hwy 183
234'
undisturbed
241 mile post
186'
242 mile post

Two examples of trailer-mounted systems are the U.S. Bureau of Public Roads Roughometer (BRP) shown in Figure 5–20 and the TRRL roughometer which has undergone a great deal of development to achieve better standardization and ruggedness (Sayers, Gillespie, and Queiroz 1986). Both devices output a roughness index in units of slope (i.e., inches per mile or millimeter per kilometer). They operate at a speed of 20 mph (32 km/h).

5.3.1.3 Response Type System Calibration

Measurements obtained from vehicular response systems depend primarily on the type of vehicle, vehicle load, vehicle condition, and testing speed. Since the numbers obtained from these systems are not reproducible over time, or when any of the above conditions change, the numbers should be calibrated to a reproducible standard roughness index in order to be useful for pavement management.

Calibration should be conducted on representative test sites—roads that cover the range of expected roughness. It is recommended that each site have a minimum length of 0.2 miles and that the roughness be as uniform as possible. The longitudinal profiles of the sites are measured using a static or dynamic profiling device. The profile is used to compute a standard roughness index such as the IRI. When computing the IRI, the simulated speed should be 80 km/h (50 mph) regardless of the speed of the system being calibrated.

If the response system is operated at different speeds, then a different calibration is conducted for each speed. Repeated measurements should be made for each site, and the average considered as the response system number for that site. Using the results from all tests sites, a relationship is established between the Standard Roughness Index and the response system number.

In order to obtain good accuracy from the response system, it is recommended that the test vehicle be equipped with very stiff shock absorbers. It is also recommended that the developed relationship with the Standard Roughness Index be verified at least monthly when the system is in use.

5.3.2 Profilographs

A profilograph is primarily used for construction acceptance purposes. A profilograph consists of a rigid beam or frame with a system of support wheels at either end, and a center wheel (NHI 2001). The support wheels at the ends establish a datum from which the deviations of the center wheel can be evaluated. The center wheel is linked to a strip chart recorder or a computer that records the movement of the center wheel from the established datum. The profilograph is pushed along the pavement, and 3 to 5 km (2 to 3 miles) can be measured in an hour. Figure 5–21 is a photograph of a truss type California profilograph.

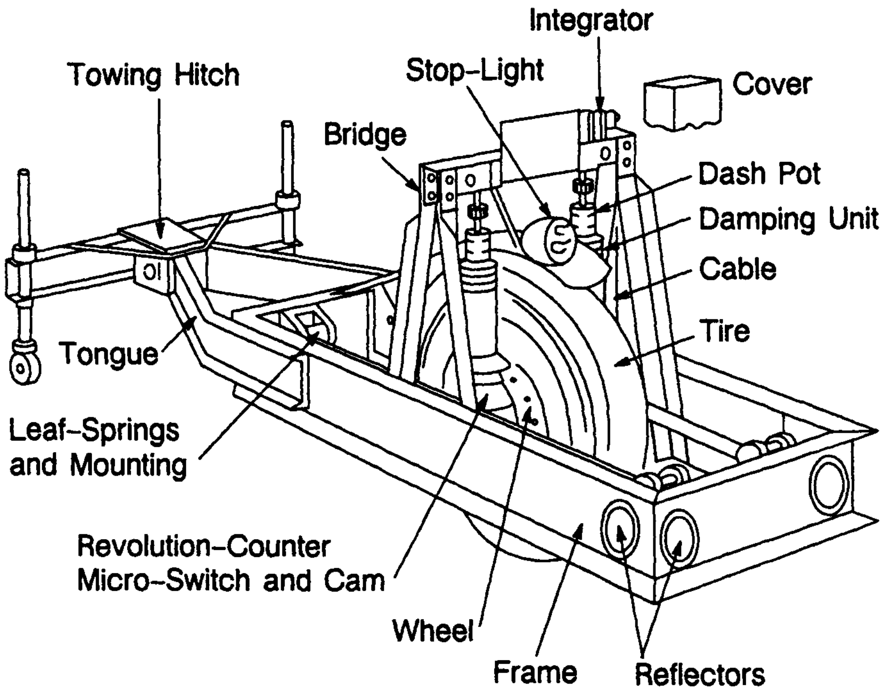


Figure 5-20. Roughometer.

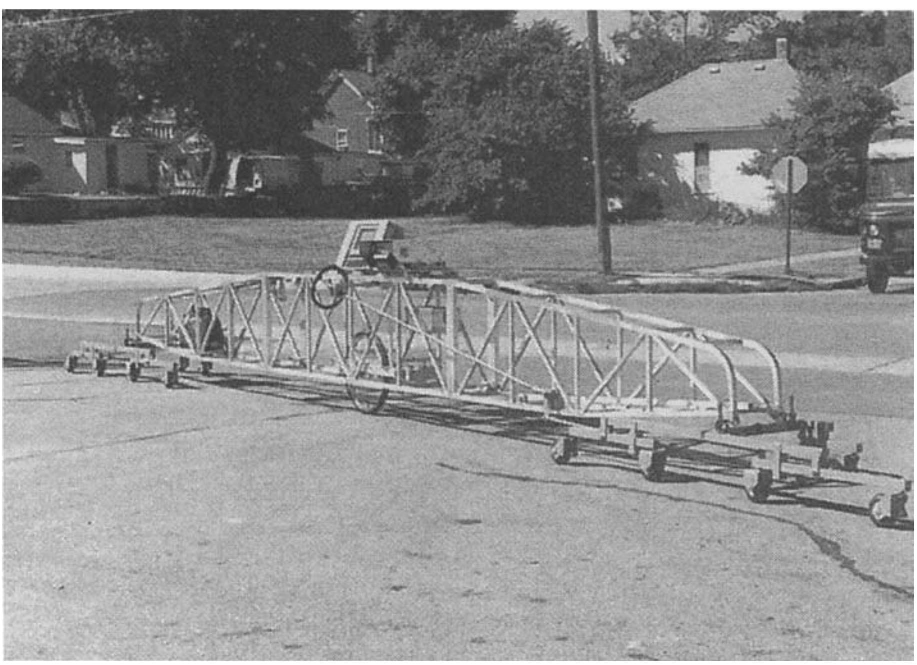


Figure 5-21. Truss Type California Profilograph.

The trace obtained from a profilograph is used to compute what is called a profile index (PI) of the pavement and to determine bump locations for grinding. The evaluation of the output from the strip chart recorder can be done either manually or electronically. In the manual method, a technician evaluates the profilograph output to determine smoothness results and bump locations. In the electronic method, the output of the strip chart recorder is scanned, the data reduction is performed by a computer program, and the results can be printed.

It should be noted that this PI is not related to the PI used in the calculation of the RN. Also there is no universal standard to calculating the PI from profilograph traces.

5.3.3 Straightedge

The straightedge is used primarily for construction acceptance purposes. Typical smoothness specifications indicate a maximum deviation of 3 mm (1/8 in.) for a 3 m (10 ft) straightedge. Figure 5–22 shows a schematic diagram of a rolling straightedge.

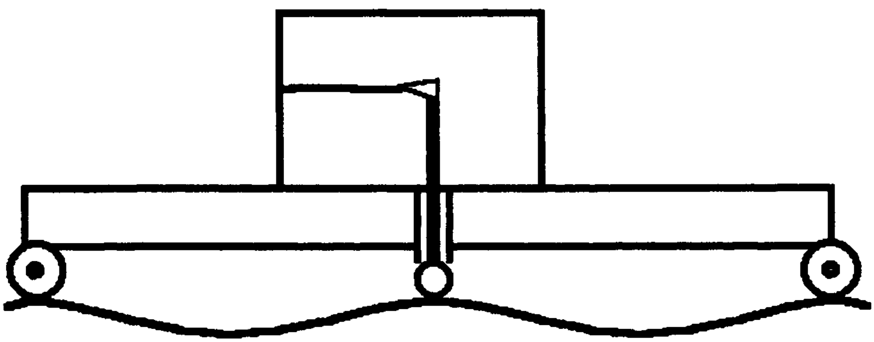


Figure 5-22. Rolling Straightedge Schematic Diagram.

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6

Skid Data Collection and Analysis

6.1 Introduction and Definitions

Vehicle control and aircraft landing safety is highly dependent on pavement surface characteristics. When pavements are dry, the friction generated between the tires and the pavement is normally high. During inclement weather, water can create a critical situation by increasing potential for hydroplaning or skidding—particularly when the skid resistance of a pavement is low. Without adequate skid and hydroplaning resistance, the driver or pilot may not be able to retain directional control and stopping ability on wet pavement. The major reason for collecting skid resistance data is to prevent or reduce accidents; the data are used to identify pavement sections with low or rapidly deteriorating levels of skid resistance. This information can then used as a management tool to help prioritize pavement maintenance and rehabilitation and to select the appropriate maintenance and rehabilitation (M&R) alternative.

Skid resistance is defined as the force that resists the sliding of tires on a pavement when the tires are prevented from rotating. Although skid resistance is considered a pavement property, various conditions, other than those of the pavement itself affect the force developed between a tire and pavement—conditions such as tire pressure, tire tread, the presence of water, temperature, load, and vehicle speed.

Hydroplaning occurs when the tire and the pavement surface are separated by water or contaminants. The two types of hydroplaning are dynamic and viscous.

Dynamic hydroplaning is a phenomenon that occurs with high water depth or vehicle speed on the pavement. Although water depth is the most significant variable, the speed at which hydroplaning occurs for a given water depth also depends on the variables mentioned earlier.

Viscous hydroplaning occurs when the surface is contaminated with a thin film of water, oil, or other slippery material. This phenomenon does not depend on the water depth and can be minimized by keeping the surface clean.

6.2 Factors Affecting Skid Resistance and Hydroplaning

As mentioned earlier, skid resistance and hydroplaning evaluation is complicated by the many variables that contribute to the friction value. The following sections summarize studies (Shahin and Darter, 1975) on the effects on skid resistance based on traffic and seasonal variations, vehicle factors (speed, tire pressure, wheel load, and tire tread), and pavement factors (surface characteristics and drainage).

6.2.1 Traffic and Seasonal Variations

Two pavement sections built at the same time may have different friction coefficients because they have been subjected to different traffic. As traffic rolls over the pavement, the tires polish the surface microtexture. Wear, dislocation, or reorientation of aggregates may also occur—especially under heavy traffic. In general, skid resistance deteriorates with increasing traffic until it reaches a level of equilibrium. There is no specific value at which it levels off. Due to seasonal variations of skid resistance (Fig. 6–1) there can be only a mean equilibrium value, which is a function of traffic and surface characteristics. Figure 6–2 shows the sideways friction factor for six different pavement sections and illustrates the effect of traffic on skid resistance. Although all six pavement sections had the same type of surface course and were installed at the same time, they were subjected to different traffic volumes. The daily traffic shown for each location is the average over a 3-year period. The study reported that skid resistance deterioration had stabilized at all locations after 2 years. The figure indicates there is a better correlation with the number of trucks (heavy traffic) than with the total number of vehicles. Figure 6–3 (from a different study reported in the same reference) illustrates the fact that skid resistance reaches a mean equilibrium value after many applications of traffic.

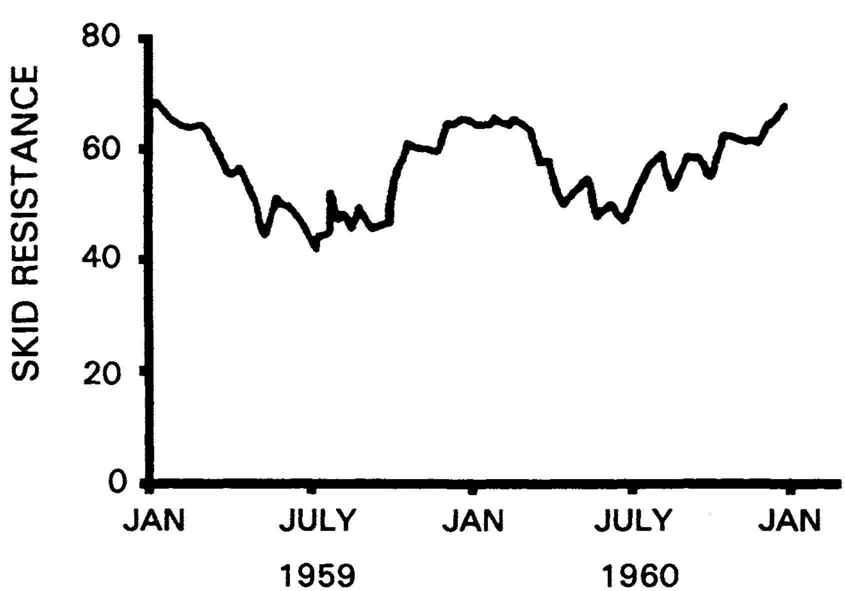


Figure 6-1. Seasonal Change of Skid Resistance (Federal Aviation Administration 1971).

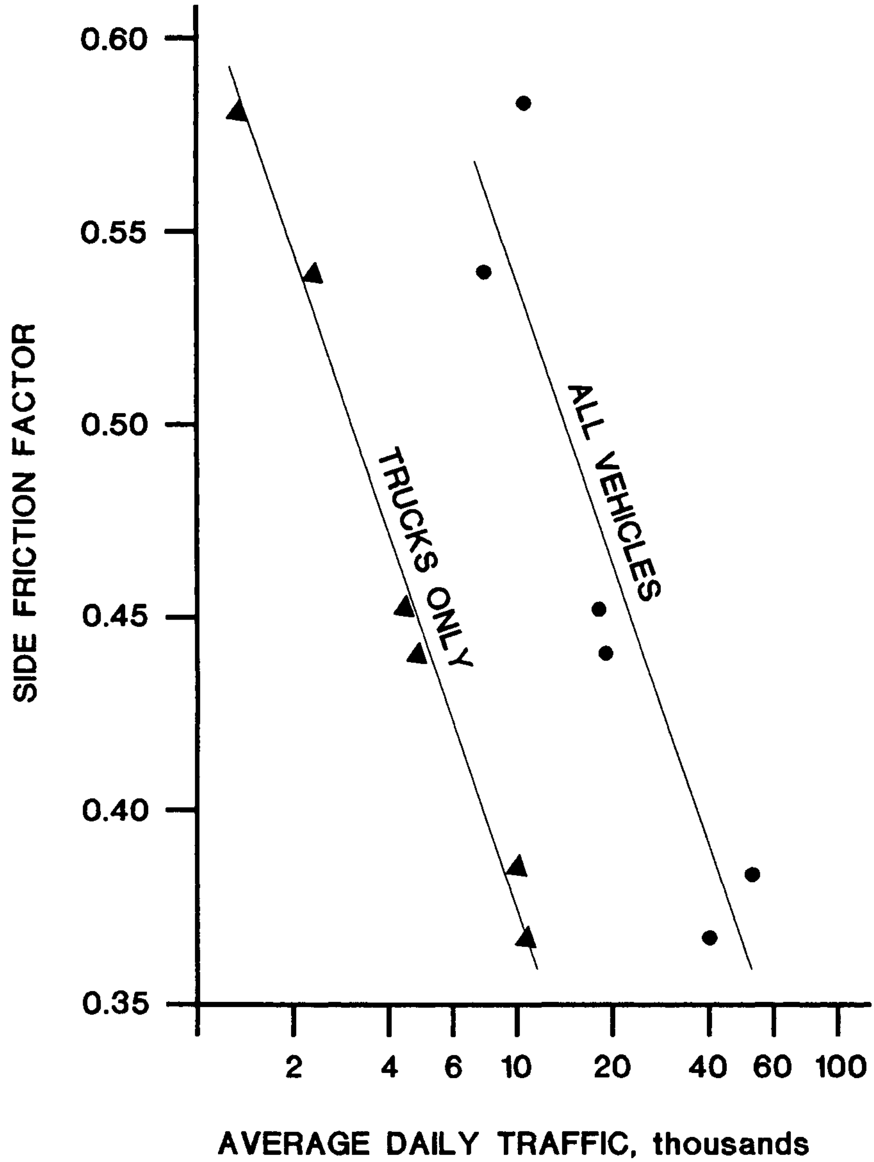


Figure 6-2. Deterioration of Skid Resistance with Exposure to Traffic (Federal Aviation Administration 1971).

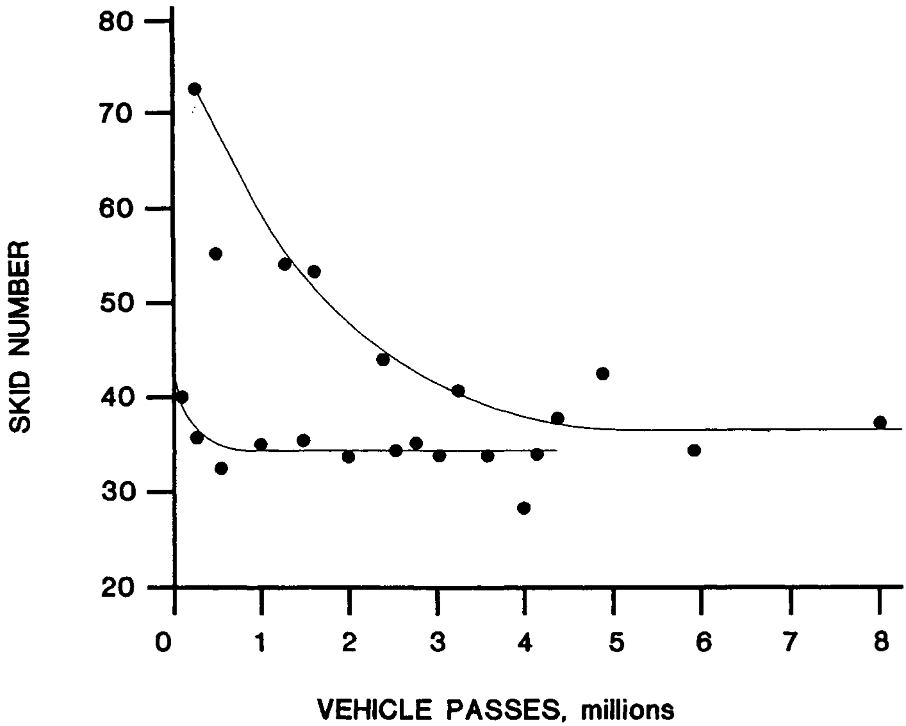


Figure 6-3. Loss of Skid Resistance of Two Pavements as a Function of Traffic Exposure (Federal Aviation Administration 1971).

6.2.2 Vehicle Factors

6.2.2.1 Speed

In general, the friction coefficient decreases with increase in speed. It has been determined that on dry pavement, the friction factor changes very little with change in speed; however, on wet pavement the decrease is significant.

Figures 6–4 and 6–5 show the change in friction with speed for concrete and asphalt pavements under dry and wet conditions. As mentioned earlier, on dry surfaces the friction factor changes only slightly with speed. This may not be the case, however, for asphalt surfaces if bleeding occurs (Fig. 6–6).

6.2.2.2 Tire Pressure

Experiments have shown that for a given wheel load, an increase in tire pressure will cause a decrease in friction coefficient. This can be attributed to the increased area of contact at low inflation pressure—the heat created by skidding or deceleration is distributed over a large area, which results in a cooler tire and a high friction coefficient.

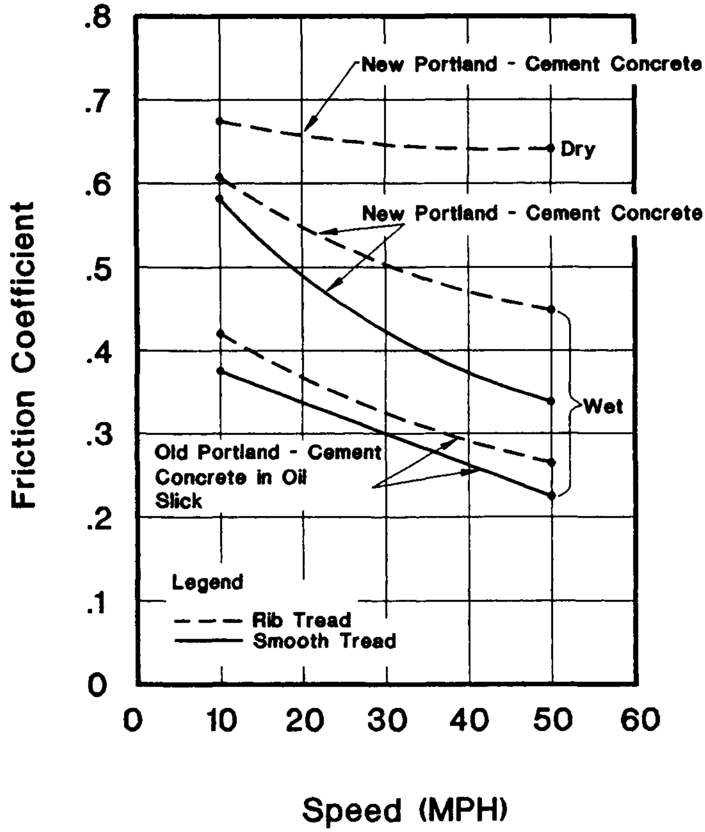


Figure 6-4. Friction Values on New Portland Cement Concrete and in Heavy Oil Slick on Old Portland Cement. Average Daily Traffic, 9000 Vehicles (Tomita 1964).

6.2.2.3 Wheel Load

Studies using varying wheel loads have shown that the friction coefficient decreases as the wheel load increases (Fig. 6–7). One of the explanations for such a phenomenon is that the increase in wheel load causes a decrease in the tire contact area per unit load and therefore a decrease in the friction coefficient. In contrast, it was also reported that a slight increase in friction coefficient occurred on ice as the rear axle static load was increased.

6.2.2.4 Tire Tread

Tread design has a significant influence on braking effectiveness. Tire grooves provide channels through which water at the tire-pavement interface can be displaced. At high speeds or in the presence of thick water films, there is not enough time for the water to be displaced and hydroplaning may occur. Figure 6–8 shows a comparison between braking effectiveness of smooth and five-groove tires for the 990A aircraft. NASA has reported that calculations using the results of this experiment showed that

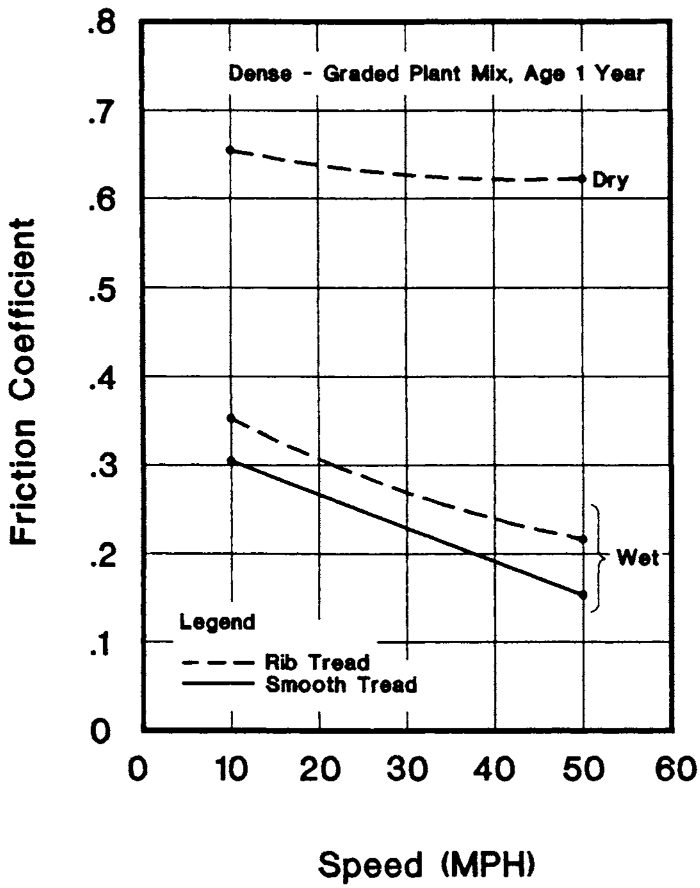


Figure 6-5. Friction Values on a Dense-graded Plant-mix Asphaltic Surface Constructed with Partly Crushed Gravel Aggregate. Average Daily Traffic, 15,000 Vehicles (Tomita 1964).

the stopping distance of the 990A aircraft with the smooth tires on the wet, ungrooved concrete runway was approximately 1,500 ft more than that required for the aircraft with unworn five-groove tires.

6.2.3 Pavement Factors

Surface texture can be defined in terms of microtexture and macrotexture (see Fig.6–9). Microtexture is what makes an aggregate smooth or rough to the touch. Its contribution to friction is through adhesion with the tire. The macrotexture is the result of the shape, size, and arrangement of the aggregates (for flexible pavements), or the surface finish (for concrete surfaces). Macrotexture's contribution to skid resistance is through developed hysteresis due to the tire deformation; the hysteresis reflects a loss in the vehicle's kinetic energy and thus helps it to stop. Figure 6–10 is a schematic diagram of the contribution of microtexture and macrotexture to the friction factor. At low speed, friction is due mainly to adhesion (microtexture). On the other hand, at high speed, the

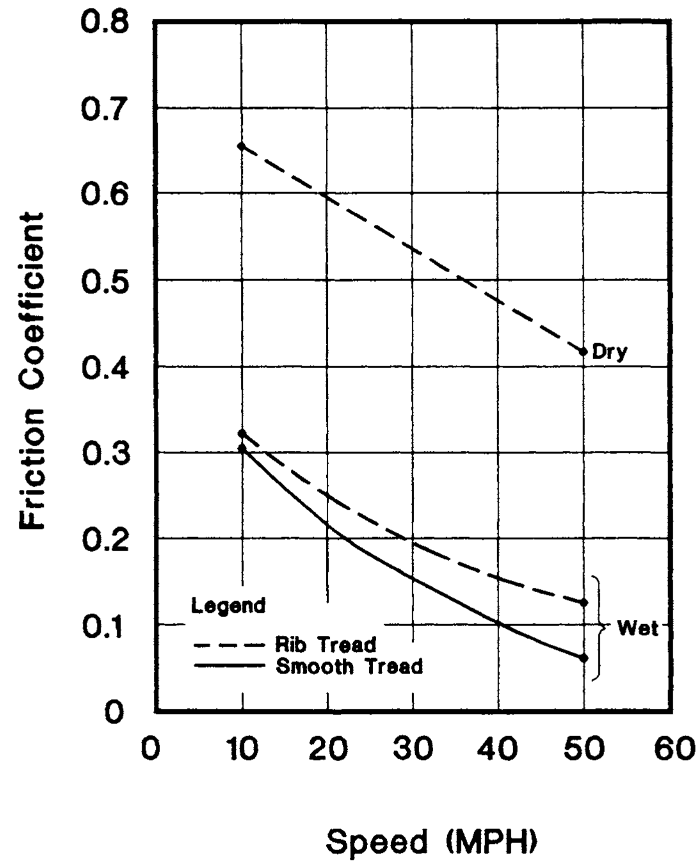


Figure 6-6. Friction Values on Asphalt Seal-coat Surface with Excess Asphalt Contributing to Bleeding in Hot Weather (Tomita 1964).

contribution of hysteresis becomes more significant. A pavement that is covered with a thin film of lubricant would provide only hysteresis.

Drainage is another significant characteristic of the pavement surface. A good drainage system provides channels for the water to escape, allowing contact between the tire and the pavement. The effectiveness of a drainage system can be evaluated by measuring the friction factor immediately after applying water to the surfaces and at intervals afterward to determine the increase in friction.

6.3 Friction Measurement Methods

Friction is a force that always opposes motion. The coefficient of friction is defined as the ratio between the frictional force in the plane of interface and the force normal to the plane. For pavements, the coefficient of friction is referred to as the friction factor, $f = F/L$, where F is the frictional force and L is the normal load. Several methods for measuring the friction factor of a pavement are discussed in the following sections.

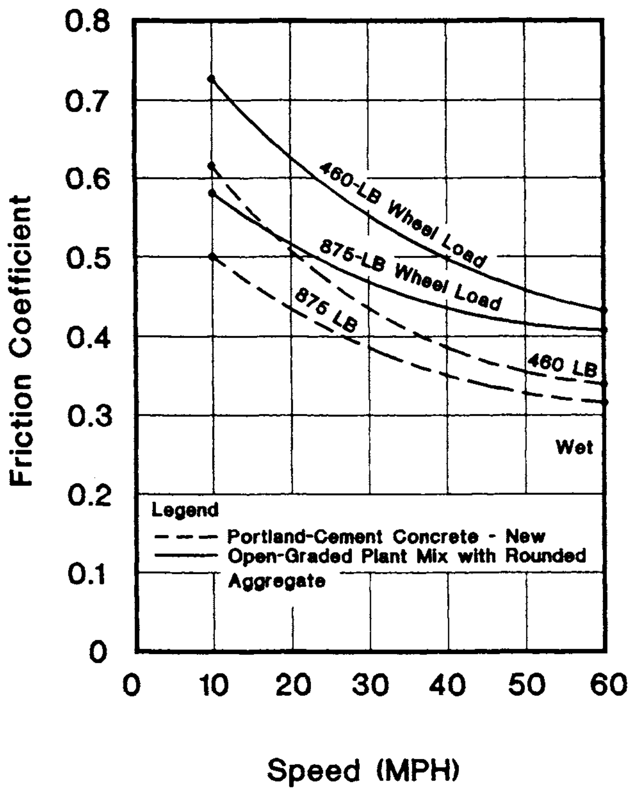


Figure 6-7. Effect of Wheel Load on Skid Resistance of Wet Portland Cement Concrete and Plant Mix Asphalt Surfaces (Tomita 1964).

6.3.1 Locked-Wheel Mode

In this method, trailers with one or two wheels are towed at a given speed. The test wheel is then locked and water is applied in front of it. After the test wheel has been sliding on the pavement for a certain distance to stabilize the temperature, the friction force in the tire contact patch is recorded for a specified period of time. The results are reported as Skid Number (*SN*), where:

$$SN = 100 \times \text{friction factor} \tag{6-1}$$

To minimize the variability of the results, a standard tire specified in ASTM Method E274 is used. Figure 6–11 is a photograph of typical locked-wheel skid trailers.

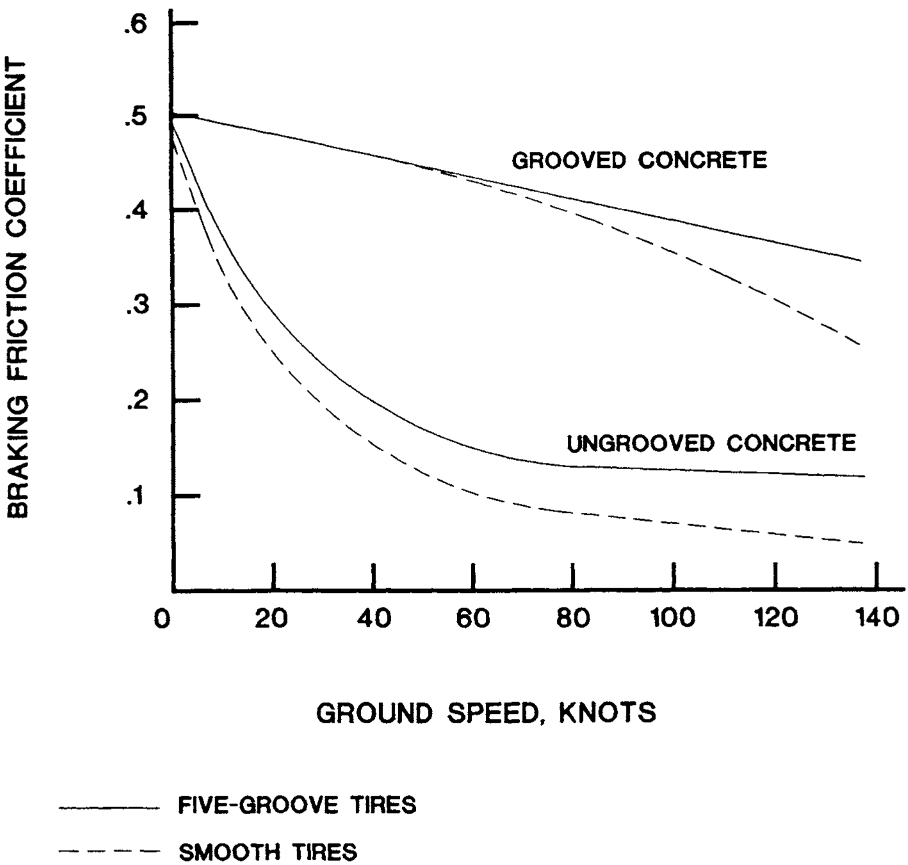


Figure 6-8. Tire-tread Effects on Wet and Puddled Runways for Twin-tandem Bogie Arrangements (NASA 1970).

6.3.2 Slip Mode

Slip (S) is defined as:

$$S = 100 \times \frac{W_0 - W}{W_0} \tag{6-2}$$

where

W_0 = angular wheel speed at free rolling

W = angular wheel speed at the time of measurement.

If the brake is applied on a straight-moving wheel, the slip increases until it reaches 100% when the wheel is locked. The friction factor increases with increasing slip until it reaches a maximum value f_{max}, at the "critical slip," and starts to decrease until the wheels are locked (Fig. 6-12). The critical slip and the ratio f_{max}/f_{lock} are functions of the surface texture and temperature; therefore they can be obtained only by appropriate measurement. Figure 6–13 shows the effect of surface texture on the ratio f_{max}/f_{lock}.

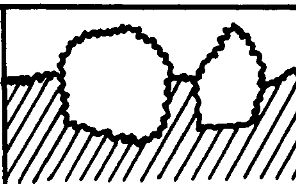
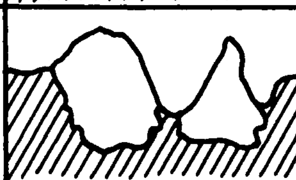
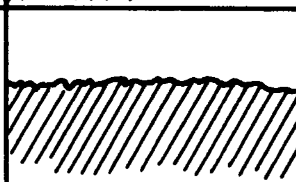
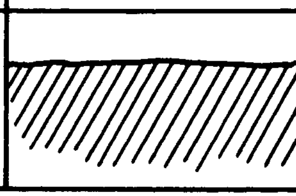
SURFACE		Scale of Texture	
		Macro (Large)	Micro (Fine)
A		Rough	Harsh
B		Rough	Polished
C		Smooth	Harsh
D		Smooth	Polished

Figure 6-9. Terms Used to Describe the Texture of a Road Surface.

The critical slip phenomenon is very important because it indicates the maximum friction does not occur when the wheels are locked, but rather in the range of 10% to 15% slip. This knowledge has brought about the development of the automatic brake control systems used on most aircraft.

More than one type of equipment is available for measuring skid resistance in the slip mode. Figures 6–14, 6–15, and 6–16 are examples of this equipment (GripTester, K.J. Law, SAAB).

6.3.3 Yaw Mode

The yaw mode measures the sideways friction factor by turning the test wheel (unbraked) to an angle with the direction of motion (yaw angle). Since the sideways friction factor varies with the magnitude of the yaw angle as shown in Figure 6–17, it is desirable to perform the testing at a yaw angle at which the friction factor becomes insensitive to small changes. A commercially available trailer for yaw mode testing is the

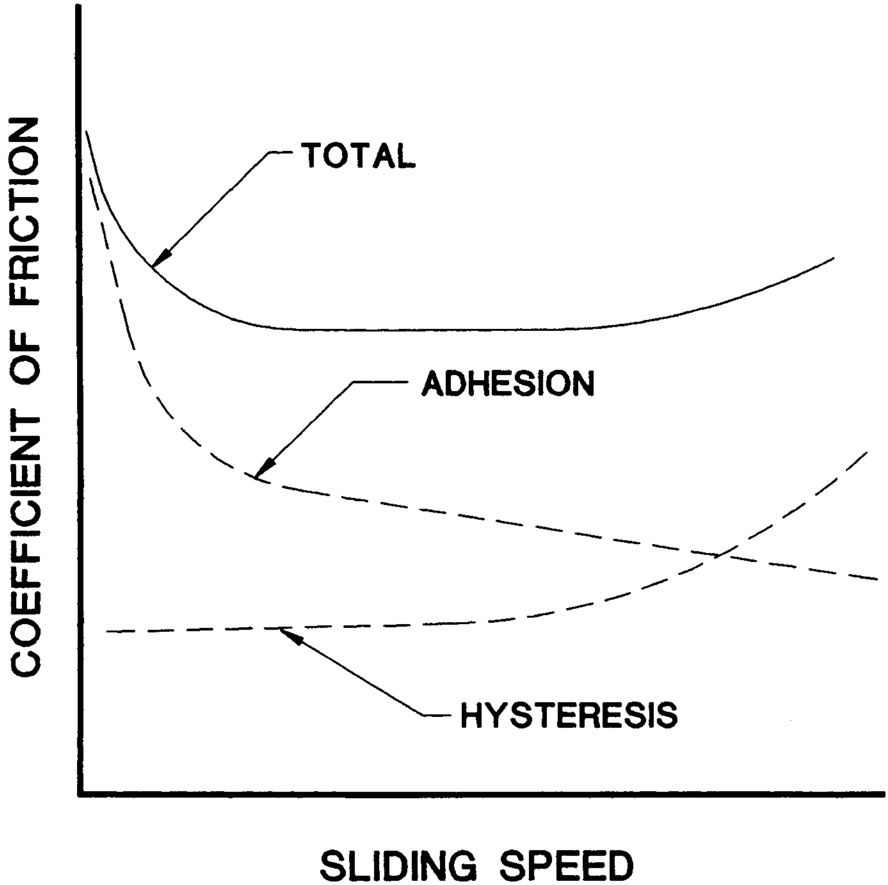


Figure 6-10. Generalized Representation of Coefficient of Friction Between Steel Sphere and Rubber as a Function of Sliding Speed (Federal Aviation Administration 1971).

Mu-Meter, which was developed in England and uses two smooth tires yawed at equal and opposite angles (7 1/2 degrees) (Fig. 6–18).

Another machine for measuring sideways friction is the SCRIM (Sideways-Force Coefficient Routine Investigation Machine), which was developed by Britain's Transport and Road Research Laboratory and manufactured under license by W.D.M., Ltd (Fig. 6–19). The vehicle carries the necessary water supply, which is spread in advance of the test wheel. The test wheel is mounted 20 degrees to the direction of motion of the vehicle and can be lifted clear of the road when not in use. The machine measures the sideways-force coefficient (SFC), which is expressed as follows:

$$SFC = \frac{\text{sideways force}}{\text{vertical reaction between tire and road surface}} \qquad (6\text{-}3)$$

SCRIM can provide continuous recording and can operate at high speeds (>40 mph).

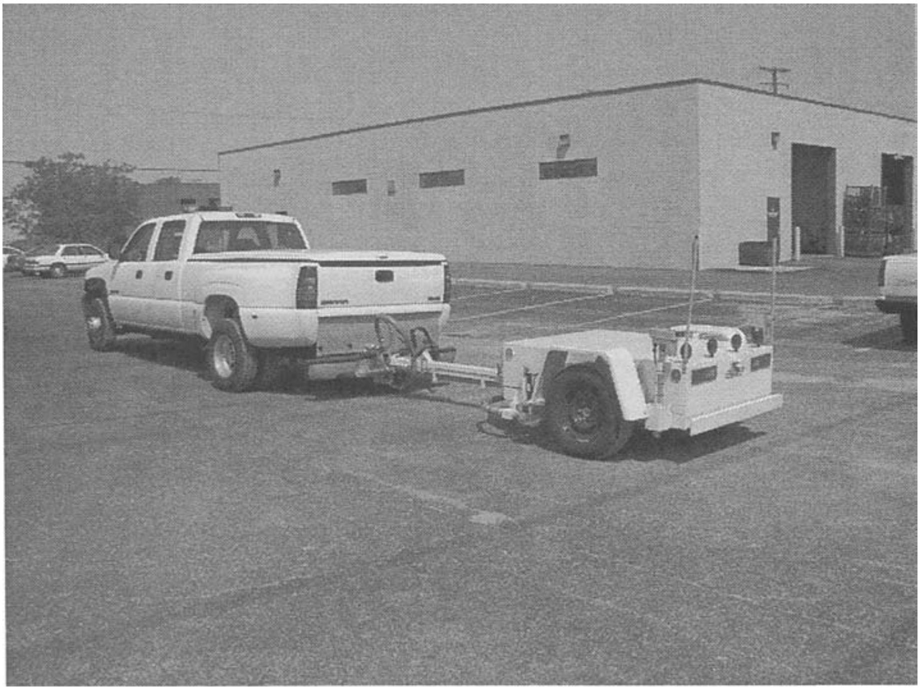


Figure 6-11. Locked-wheel Skid Trailers (Dynatest 2004).

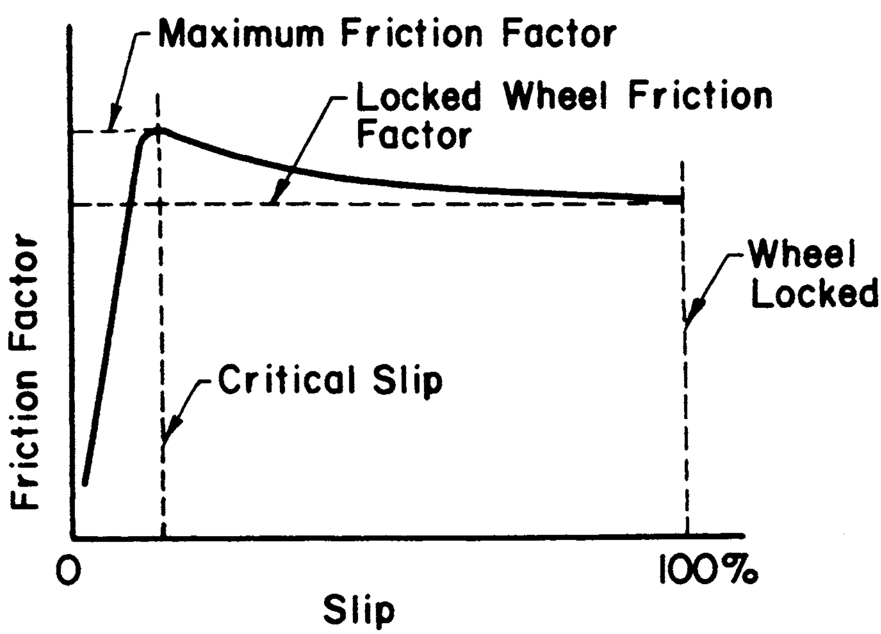


Figure 6-12. Friction Factor as a Function of Slip (Wheel Moving in Direction of Wheel Plane Being Braked) (From Federal Aviation Administration 1971).

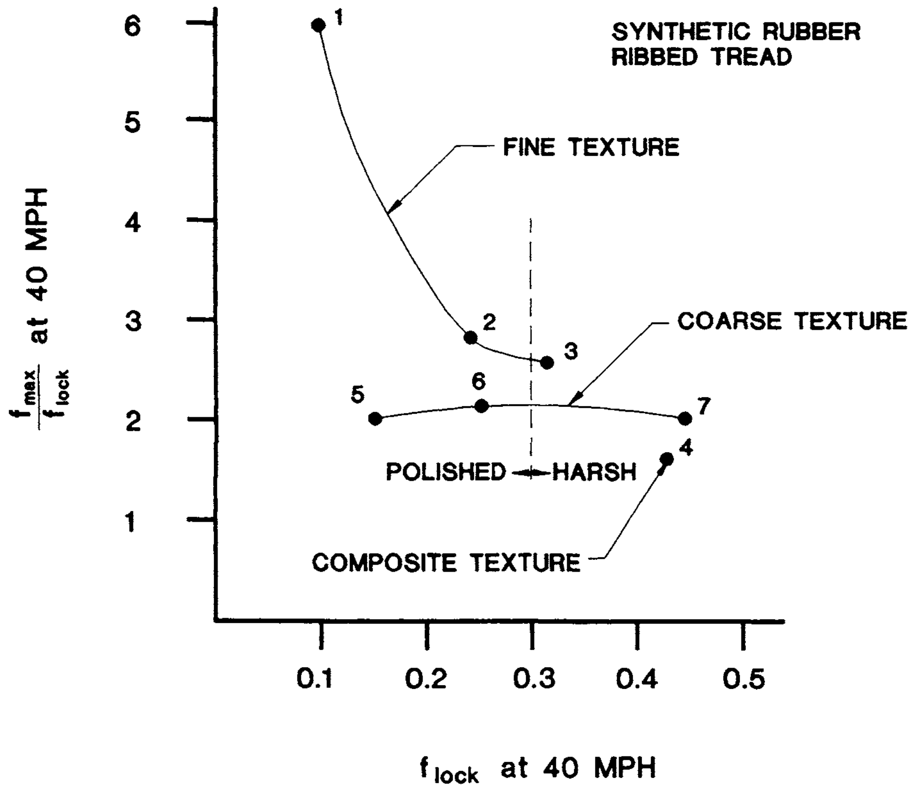


Figure 6-13. Ratio of Maximum and Locked-wheel Friction Factors at 40 mph on Various Wet Surfaces (From Federal Aviation Administration 1971.)

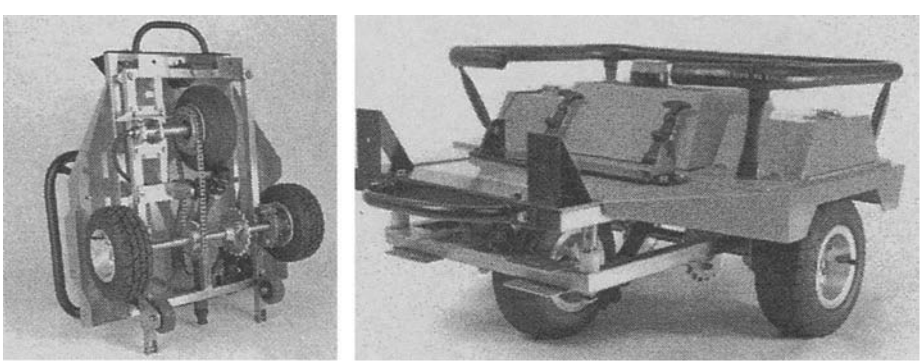


Figure 6-14. GripTester Surface Friction Tester (Findlay, Irvine Ltd 2004).

Figure 6-15. Dynatest Friction Tester. (Dynatest 2004)

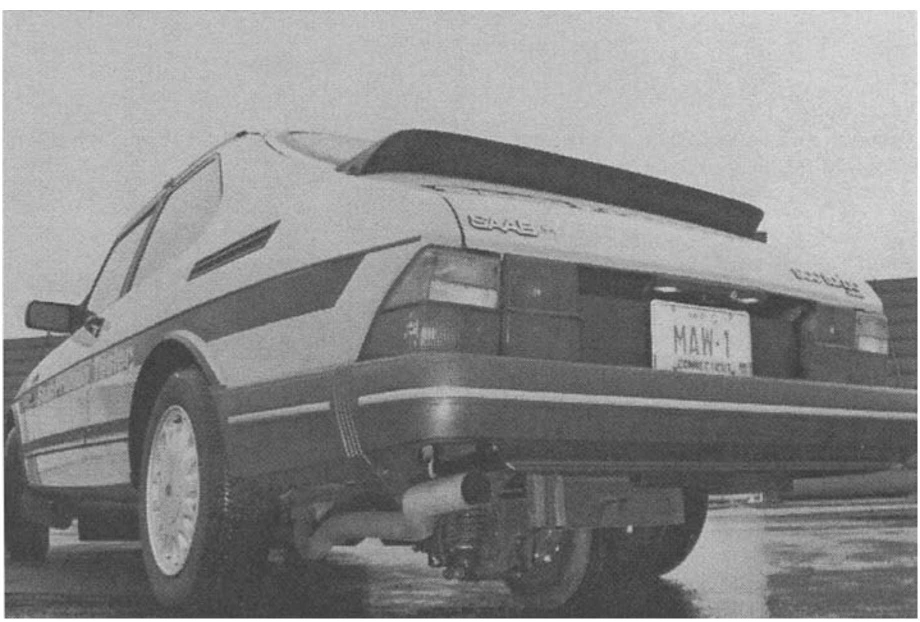


Figure 6-16. SAAB Friction Tester.

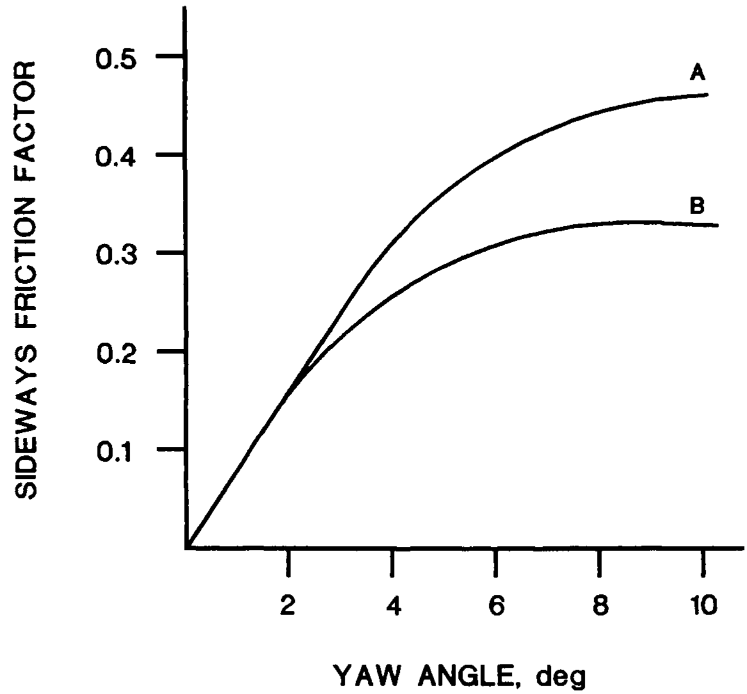


Figure 6-17. Typical Sideway Friction Factor vs. Yaw Angle Relationships for Two Wet Pavements (A and B) (From Federal Aviation Administration 1971).

6.3.4 Laboratory and Texture Measurement Methods

One of the most common laboratory devices is the British Portable Tester developed by the British Road Research Laboratory (Fig. 6–20). It consists of a rubber shoe attached to a pendulum, which slides over a sample of the surface under study. The method of testing is described in ASTM Method E303. The results are reported as British Pendulum Numbers (BPN).

Several methods are available for measuring the texture of a pavement surface, but no successful correlation has been developed between any of these individual measurements and skid resistance.

The following surface texture method was presented in FAA Advisory Circular AC No. 150/5320-12C. The procedure is effective for measuring the macrotextural depth but not the microtextural properties of the pavement surface. For runways, the average macrotextural depth should be at least 0.015 in. for forced skid resistance. The procedure consists of spreading grease of a known volume. The area covered by the grease is then measured. The texture depth is computed as the volume of grease/area covered by grease. The average texture depth is the sum of individual tests/total tests.

Texture can also be measured using portable devices such as the TRRL texture meter (Fig. 6–21). The device is manually propelled and a laser beam continually scans the surface of the pavement. The results are converted to surface texture information.

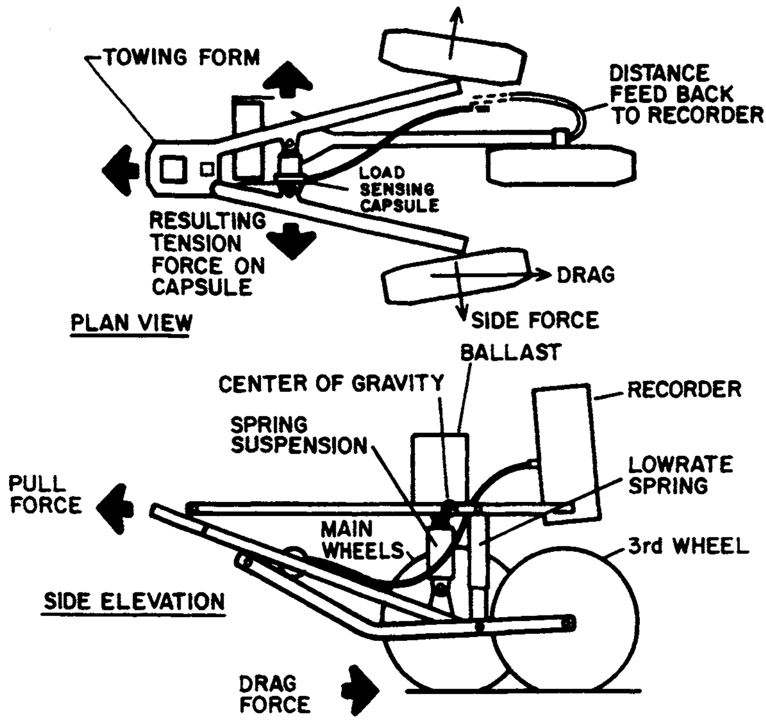


Figure 6-18. Diagrammatic Layout of Mu-Meter (Tomita 1964).

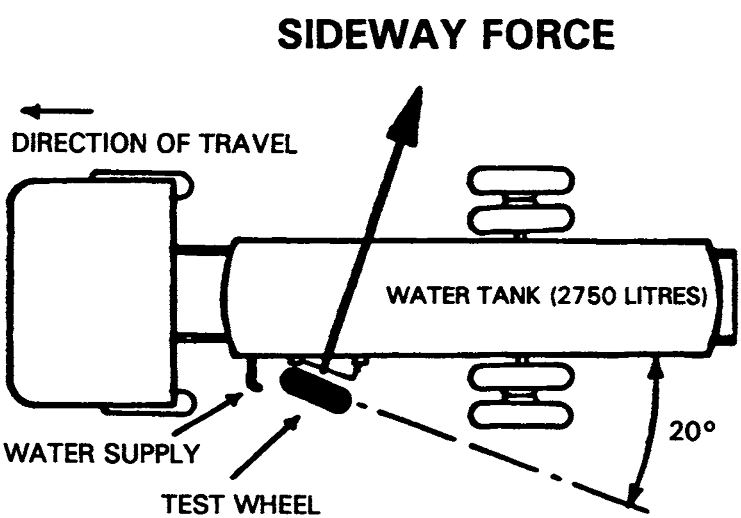


Figure 6-19. Sideways-force Coefficient Routine Investigation Machine. (SCRIM)

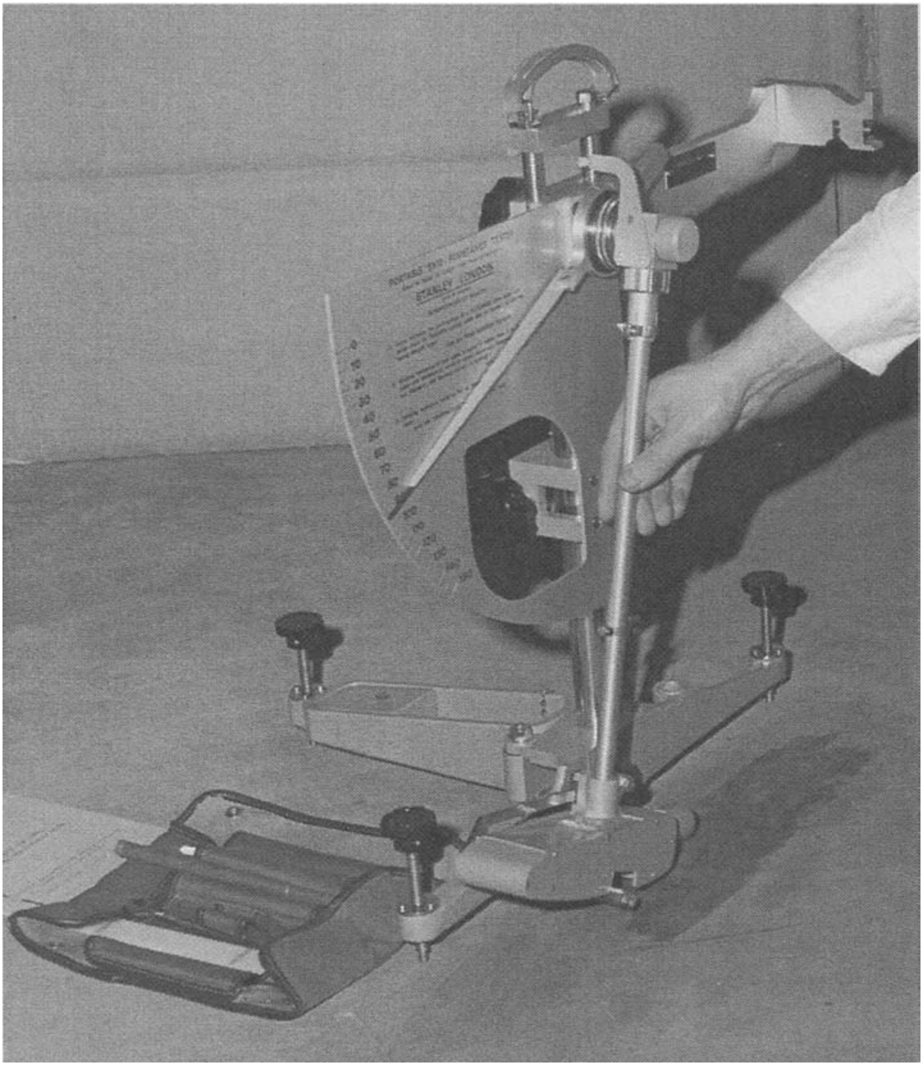


Figure 6-20. British Road Research Laboratory's Pendulum Friction Tester (British Portable Tester).

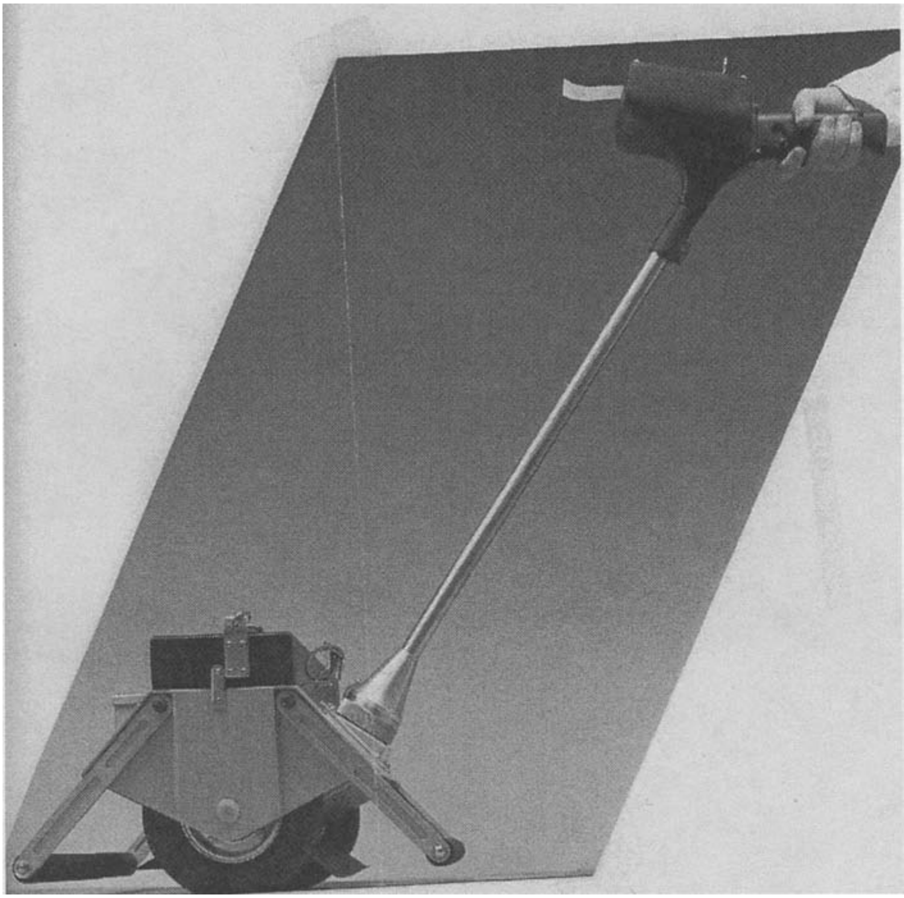


Figure 6-21. TRRL texture meter.

6.4 Friction Survey Procedures

6.4.1 Frequency

Pavement surface texture wears down over time due to traffic tire rolling and braking. Therefore, friction should be measured periodically and more frequently with heavier traffic. Figure 6–1 presents the FAA suggested friction survey schedule for airports. When any runway end has 20 percent or more wide body aircraft (L-1011, B-747, DC-10, MD-11, C-5, etc.) of the total aircraft mix, it is recommended that the airport operator should select the next higher level of aircraft operations in Figure 6–22 to determine the minimum survey frequency. As airport operators accumulate data on the rate of change of runway friction under various traffic conditions, the scheduling of friction surveys may be adjusted to ensure that evaluators will detect and predict marginal friction conditions in time to take corrective actions.

6.4.2 Visual Survey

It is always beneficial to conduct visual inspection of the surface condition in addition to the direct friction measurement. During the visual survey note the condition of pavement texture, evidence of drainage problems, and presence of distresses that could cause skidding or hydroplaning. Distresses that lead to decreased skid resistance or increased hydroplaning potential include bleeding, depressions, rutting, and joint faulting in concrete pavements. For airfields, the extent and degree of rubber accumulation on runways also should be noted.

6.4.3 Measurement Operation

Friction measurement should be conducted along the wheel path. On highways with four or more lanes, the outside lanes are tested, but some agencies test all four lanes. The test is conducted at intervals ranging from .1 to 1.0 mile.

On runways, the measurements are conducted along the entire length of the runway, 10 ft off the centerline.

NUMBER OF DAILY MINIMUM TURBOJET AIRCRAFT LANDINGS PER RUNWAY END	MINIMUM FRICTION SURVEY FREQUENCY
LESS THAN 15	1 YEAR
16 TO 30	6 MONTHS
31 TO 90	3 MONTHS
91 TO 150	1 MONTH
151 TO 210	2 WEEKS
GREATER THAN 210	1 WEEK

Figure 6-22. Friction Surveys Frequency (FAA 1997).

6.5 M&R Alternatives for Solving Skid Problems

Several M&R alternatives can be used if the friction condition survey revealed skid problems or potential problems. Figure 6–23 shows friction rating used by the US Airforce (USAF ETL 04-9, 2004) for selected equipment. The selection of any M&R technique should be coordinated with the results of the project evaluation presented in Chapter 11. Following is a brief description of common M&R techniques to improve skid resistance.

6.5.1 Overlays

Overlays are particularly feasible if the pavement is structurally deficient or needs to be strengthened for future traffic. Overlays are also feasible if defects are severe enough that a surface treatment will not correct them.

6.5.2 Porous Friction Course (PFC)

PFC is an open-graded thin, asphaltic concrete overlay about 1 to 1.5 in. thick. The overlay is designed with no fines so voids will allow water to drain through the overlay.

The use of PFC on runways with high traffic may be a problem because it is nearly impossible to remove rubber buildup without damaging the PFC layer. The FAA recommends that PFC overlays not be constructed on runways at airports that have high traffic operations (over 91 turbojet arrivals per day per runway end.) (FAA 1997).

Friction Rating	Friction Index 65 kph (40 mph) Nominal Test Speed, Unless Noted[4]			
	Grip Tester[1]	Mu-Meter	Runway Friction Tester[2]	Locked Wheel Devices[3]
Good	>0.49	>0.50	>0.51	>0.51
Fair	0.34-0.49	0.35-0.50	0.35-0.51	0.37-0.51
Poor	<.34	<0.35	<0.35	<0.37

1. Measurements obtained with smooth ASTM tire inflated to 140 kPa (20psi)
2. Measurements obtained with smooth ASTM 4in x 8.0in tire inflated to 210 kPa (30psi)
3. ASTM E-274 skid trailer and E-503 diagonal-brake vehicle equippedwith ASTM E-524 smooth test tires inflated to 170 kPa (24 psi)
4. A wet runway produces a drop in friction with an increase in speed. If the runway has good texture, allowing the water to escape beneath the tire, then friction values will be less affected by speed. Conversely, a poorly textured surface will produce a larger drop in friction with an increase in speed. Friction characteristics can be further reduced by poor drainage due to inadequate slopes or depressions in the runway surface.

Figure 6-23. Friction Ratings Used by the US Airforce (USAF 2004).

6.5.3 Chip Seal and Aggregate Slurry Seal/Surfacing.

Seals provide a feasible alternative for low-volume roads, but should be considered only an interim alternative for pavement with high-volume traffic. These alternatives are normally adequate for 2 to 5 years before they should be repeated or a more major alternative such as overlay is applied. The use of rubber additives will provide better bond and adhesion to the existing pavement surface, and thus longer life.

6.5.4 Saw-Cut Grooves

Grooves provide a significant improvement in friction during wet weather. The grooves provide channels for the water to escape, thus allowing direct contact between the vehicle tire and the pavement surface. The grooves are sawed transversely across the highway or runway. On runways, the grooves do not have to extend all the way to the edge of the runway to be effective. The recommended FAA groove configuration is 0.25 in. by 0.25 in., width by depth, and 1.5 in. center-to-center.

Grooves are mostly used on concrete pavements but can also be used on asphaltic concrete pavements. On runways, it is more difficult to remove rubber deposits from grooves in asphaltic concrete pavements than concrete pavements.

6.5.5 Removing Contaminants

Contaminants include rubber deposits, oil spills, dust, and any other material that may decrease skid resistance. If the friction survey indicates the buildup of such contaminants, several methods can be used to remove them. Some of these methods are high pressure water, chemical, and mechanical grinding. Another technique (Humble, 1993) is based on high velocity impact of tiny steel abrasive media against the pavement surface to be cleaned or textured. The steel media, 0.039 inch in diameter, is impelled at the surface of the pavement at speed of 400 ft/sec. It abrades and removes the contaminants and textures the pavement surface for improved skid resistance. The equipment used to apply this technique is known as the "Skidabrader" and is shown in Fig. 6–24. The steel abrasive media is continually recycled as the equipment moves down the road or airfield. The equipment is connected to a truck-mounted dust collector to insure a dust free environment.

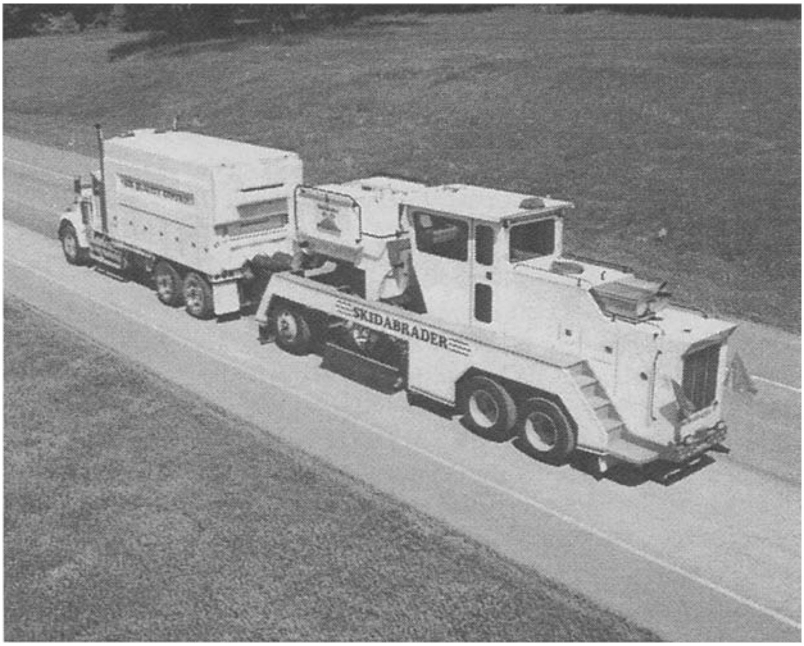


Figure 6-24. Skidabrader Equipment (Skidabrader 2004).

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7

Pavement Condition Prediction Models

Pavement condition prediction models are imperative for a complete pavement management system. In a management system, condition prediction models perform a function similar to that of a car engine.

This chapter presents the different aspects of pavement condition prediction modeling, including use of prediction models at the project and network levels, techniques for developing prediction models, and a description of the prediction models used in the Micro PAVER (Micro PAVER, 2004) pavement management system.

7.1 Uses of Prediction Models

Information on several pavement condition characteristics is critical to performing management functions. The characteristics include roughness, skid resistance, structural capacity, and distress. Several condition indices have been developed to quantify these characteristics, for example, the International Roughness Index (IRI) for measuring roughness (Fig. 5–4), and the skid number (SN) for measuring skid resistance (100 x friction factor). Indices referring to nondestructive deflection testing related indices, such as maximum deflection and the area of the deflection basin, are examples of structural indices. Individual distresses can be used as indices, for example, percent area with alligator cracking. Composite distress indices such as the Pavement Condition Index (PCI) have been successfully used in management systems. When correctly developed, a composite distress index will indirectly provide a measure of roughness, skid, and a structural integrity (not capacity) because of the relationship between the various distress types and each of the condition characteristics.

Condition prediction models are used at both the network and project levels to analyze the condition and determine maintenance and rehabilitation (M&R) requirements. At the network level, prediction models uses include condition forecasting, budget planning, inspection scheduling, and work planning. One of the most important network uses of prediction models is to conduct "what if" analyses—to study the effects of various budget levels on future pavement condition (see Chapter 10).

Prediction models are used at the project level to select specific rehabilitation alternatives to meet expected traffic and climatic conditions. The models provide the major input to performing life-cycle cost (LCC) analysis to compare the economics of various M&R alternatives.

When planning M&R at the network level, the concern is normally the level of M&R needed. At the project level, the concern is focused on specific M&R alternatives, including preliminary design of each alternative. Therefore, accuracy of prediction is more important for project level analysis than for network level analysis.

7.2 Techniques for Developing Prediction Models

Many techniques are available for developing pavement deterioration models. The techniques include straight-line extrapolation, regression (empirical), mechanistic empirical, polynomial constrained least square, S-shaped curve, probability distribution, and Markovian.

The degree of accuracy required of a prediction model is a function of its intended use. Models for project level analysis need to be more accurate than those for network level analysis.

7.2.1 Straight-Line Extrapolation

The simplest condition prediction is based on a straight-line extrapolation of the last two condition points. This method is applicable only for individual pavement sections and does not lead to the development of a model that can be used with other pavement sections. The method assumes that traffic loadings and previous maintenance levels will continue as in the past. This method requires that at least one condition measurement has been performed since construction, thereby providing two points: an initial pavement condition that can be assumed at the time of last construction, and a second pavement condition determined at inspection time. The straight-line extrapolation is used because it is not known whether the rate of deterioration is likely to increase or decrease (Fig. 7–1).

It should be noted that when predicting the condition of an individual pavement section, factors such as foundation support, climate, pavement structure, and past traffic are all accounted for. Although this method of predicting deterioration is accurate enough for a short period of time (a few years), it is not accurate for long periods of time. Also, the straight-line extrapolation method cannot be used to predict the rate of deterioration of a relatively new pavement or a pavement that has recently received major rehabilitation.

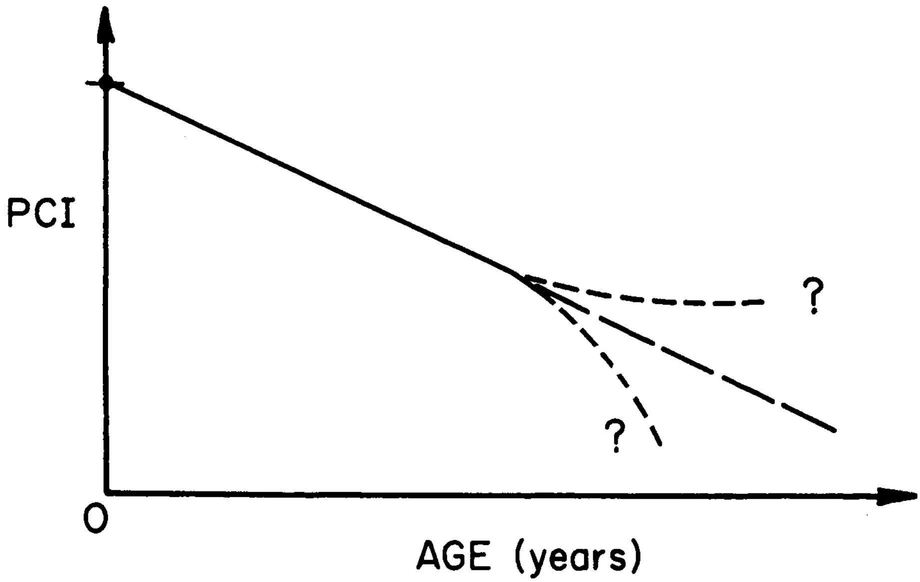

Figure 7-1. Straight-Line Extrapolation.

7.2.2 Regression (Empirical) Technique

Regression analysis is used to establish an empirical relationship between two or more variables. Each variable is described in terms of its mean and variance. Several forms of regression analysis are used, and the simplest form is linear regression between two variables; the model is described as:

$$Y_i = \alpha + \beta X_i + \varepsilon_i \tag{7-1}$$

where:

Y = dependent variable, that is, condition indices

X = explanatory or independent variable, that is, time since last major rehabilitation

ε = prediction error

α, β = regression parameters

The mean or estimated value of Y_i, $E(Y_i)$, for each value of X_i, can thus be determined as:

$$E(Y_i) = \hat{Y}_i = \hat{\alpha} + \hat{\beta} X_i \tag{7-2}$$

where

$\hat{Y}_i$, $\hat{\alpha}$, and $\hat{\beta}$ are estimates of Y_i, α, and β, respectively

The values of $\hat{\alpha}$ and $\hat{\beta}$ are determined so as to minimize the sum of squared errors of the observed values Y_i from their estimate $\hat{Y}_i$, that is, minimize s given by

$$s = \sum_{i=1}^{n}\left[Y_i - \hat{Y}_i\right]^2 \tag{7-3}$$

$$s = \sum_{i=1}^{n}\left[Y_i - \hat{\alpha} - \hat{\beta} X_i\right]^2 \tag{7-4}$$

where
 n is the number of observed data points

The above method is known as the method of "least squares." The value of $\hat{\alpha}$ and $\hat{\beta}$ are determined by setting the partial derivative of s with respect to $\hat{\alpha}$ and $\hat{\beta}$ in Equation 7-4, equal to 0. This will lead to the following:

$$\hat{\alpha} = \overline{Y} - \hat{\beta}\,\overline{X} \tag{7-5}$$

$$\hat{\beta} = \frac{\sum\left(X_i - \overline{X}\right)\left(Y_i - \overline{Y}\right)}{\sum\left(X_i - \overline{X}\right)^2} \tag{7-6}$$

where $\overline{X}$ and $\overline{Y}$ are the average values of X and Y, respectively. $\hat{\alpha}$ is the intercept of the line that measures the estimated value of Y corresponding to a value of X equal to zero. $\hat{\beta}$ is the slope of the line that measures the estimated value of Y corresponding to a unit change in the value of X. Figure 7–2 is geographical presentation of the regression line. Referring to Figure 7–2, it can be mathematically proven that:

$$\sum_{i=1}^{n}\left(Y_i - \overline{Y}\right)^2 = \sum_{i=1}^{n}\left(\hat{Y}_i - \overline{Y}\right)^2 + \sum_{i=1}^{n}\left(Y_i - \hat{Y}_i\right)^2 \tag{7-7}$$

$\sum\left(Y_i - \overline{Y}\right)^2$ = total sum of squares (SST)

$\sum\left(\hat{Y}_i - \overline{Y}\right)^2$ = regression sum of squares (SSR)

$\sum\left(Y_i - \hat{Y}_i\right)^2$ = error sum of squares (SSE)

The goodness of fit of the regression line can be measured using the coefficient of determination (R^2), which measures the proportion of total variation about the mean ($\overline{Y}$), which is explained by regression:

$$R^2 = \frac{\text{SSR}}{\text{SST}} = \frac{\sum\left(\hat{Y}_i - \overline{Y}\right)^2}{\sum\left(Y_i - \overline{Y}\right)^2} \tag{7-8}$$

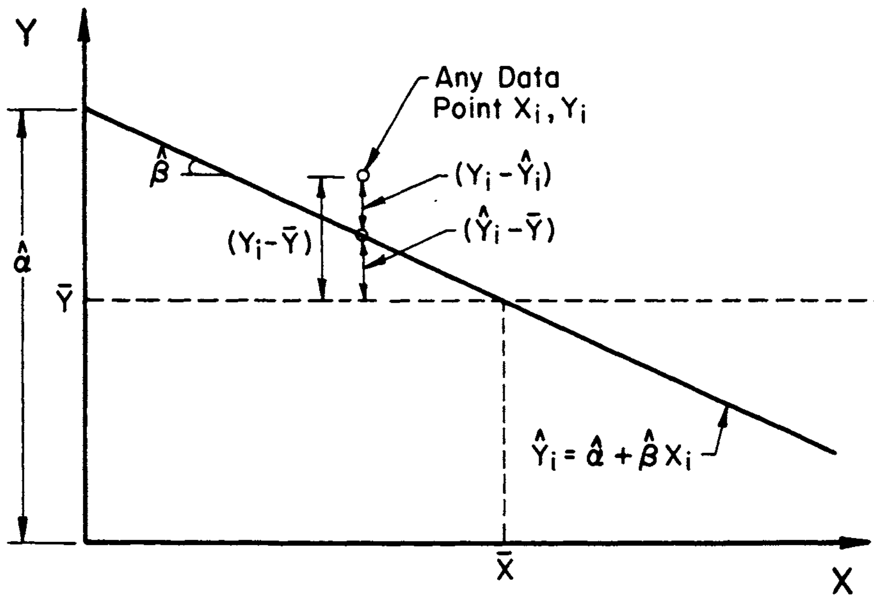

Figure 7-2. Regression Line.

Equation 7–8, can also be written as

$$R^2 = \frac{\hat{\beta}^2 \sum \left(X_i - \bar{X}\right)^2}{\sum \left(Y_i - \bar{Y}\right)^2} \tag{7-9}$$

Another important regression parameter to examine is the error term, *e*. The errors, $e_i = Y_i - \hat{Y}_i$, are assumed to be independent normal values with a mean of zero and a standard deviation of σ which can be computed as:

$$\sigma\left(Y_i - \hat{Y}_i\right) = \sqrt{\frac{\sum \left(Y_i - \hat{Y}_i\right)^2}{n-2}} \tag{7-10}$$

It is desirable that the value of $\sigma\left(Y_i - \hat{Y}\right)$ be small since it has a significant effect on the confidence band for prediction.

Linear regression analysis can be performed for more than two variables, and in that case is known as "multiple linear regression." It is assumed that the dependent variable, *Y*, is a linear function of the independent variables, that is,

$$E(Y) = a + b1\,X1 + b2\,X2 + \ldots \tag{7-11}$$

The estimation of the regression parameters is calculated in a way similar to that for straight-line regression analysis.

Nonlinear regression may be necessary when the relationship between Y and X is not linear. An example is the relationship between condition and time shown in Figure 7–3. A linear relationship may be used, but the model will underestimate the condition during the early life of the pavement and will overestimate the condition during the later part of the pavement life. A nonlinear relationship can be analyzed as a linear model by transforming the X variable. For example, the relationship could be

$$E(Y) = \hat{\alpha} + \beta f(x) \tag{7-12}$$

where

$f(x)$ = function of x, such as x^2 or $\ln(x)$

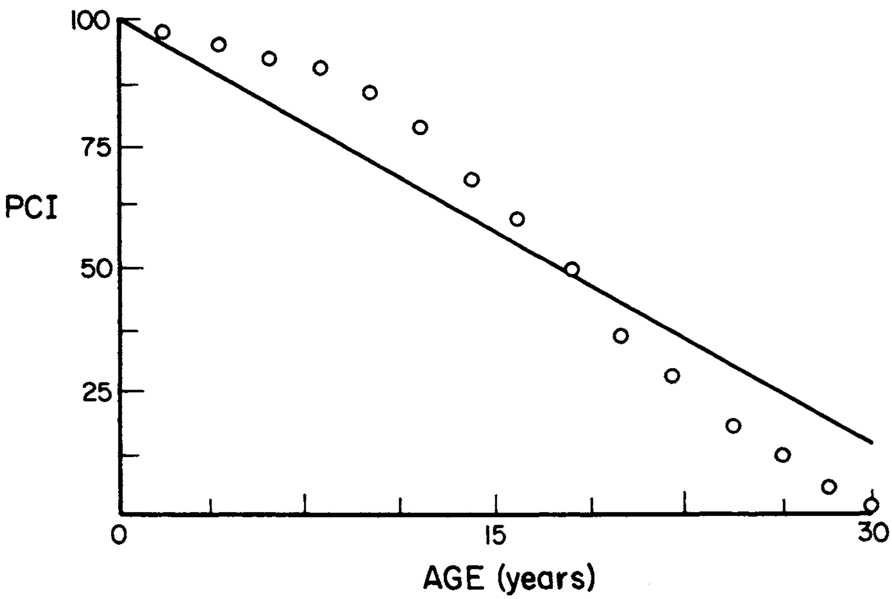

Figure 7-3. Example Nonlinear Relationship Between PCI and Age.

7.2.3 Mechanistic–Empirical Model

A pure mechanistic approach to modeling is applicable only to calculating pavement response (i.e., strain, stress, and deflection). This response is normally caused by forces created by traffic, climate, or a combination of the two. Pure mechanistic models for calculating stress and strain cannot be classified as prediction models. However, the calculated stress and strain can be used as input (independent variable) to a regression (empirical) prediction model as presented in the previous section. A prediction model developed using regression technique with pavement response as the dependent variable is called a mechanistic-empirical model.

An example of a mechanistic-empirical model is that used for predicting asphalt pavement fatigue life (N),

$$N = A*(l/e)**B \tag{7-13}$$

In that prediction model, the strain "*e*" produced by wheel loadings is calculated mechanistically. The coefficients *A* and *B*, however, are determined using regression techniques.

7.2.4 Polynomial Constrained Least Squares

This is one of the most powerful techniques for predicting the change in a variable *Y* (i.e., PCI or roughness) as function of one variable *X* (i.e., age or traffic).

Given the observations:

$$(x_i, Y_i) \qquad i = 1, 2, \ldots n$$

A polynomial of degree, n:

$$P(x) = a_0 + a_1 x + a_2 x^2 + \ldots + a_n x^n \tag{7-14}$$

is established such that a least squares fit is obtained and the desired constraint is met.

For example, when fitting PCI vs. age, it is desirable to ensure that the polynomial slope:

$$P'(x) = a_1 + 2a_2 x + 3a_3 x^2 + \ldots + na_n x^{n-1} \tag{7-15}$$

is nonpositive at any age $(x) = 0, 1, \ldots z$ when z is the highest age. Therefore, the polynomial coefficients $a_1, a_2, \ldots, a_n$ are determined such that

$\sum [Y_i - P(x_i)]^2$ is minimized, with the constraints that:

1. $a_0 = 100$, which ensures $P(0) = 100$, and
2. $a_0 + 2a_2 x + 3a_3 x^2 + \ldots + na_n x^{n-1} \leq 0$, which ensures for $x \leq 0$ a nonpositive slope

Figure 7–4 shows a comparison of the unconstrained and constrained least squared fourth-degree polynomials generated for a small network of asphalt concrete pavement roads.

7.2.5 S-Shape Curve

Similar to the polynomial constrained least squares, the S-shaped curve fitting technique is useful when predicting the change in a variable, Y, as a function one variable, X. R. E. Smith (1986) used an S-shaped model for relating PCI to pavement age. The model had the form

$$\text{PCI} = 100 - \rho / (\ln(\alpha) - \ln(\text{AGE}))^{**}(1/\beta) \tag{7-16}$$

Where ρ, α, and β are constants. The α constant controls the age at which the PCI is projected to reach 0 as shown in Figure 7–5. The β constant controls how sharp the curve bends as shown in Figure 7–6. The ρ constant controls the location of the inflection point in the curve as shown in Figure 7–7. These three constants are determined using regression analysis.

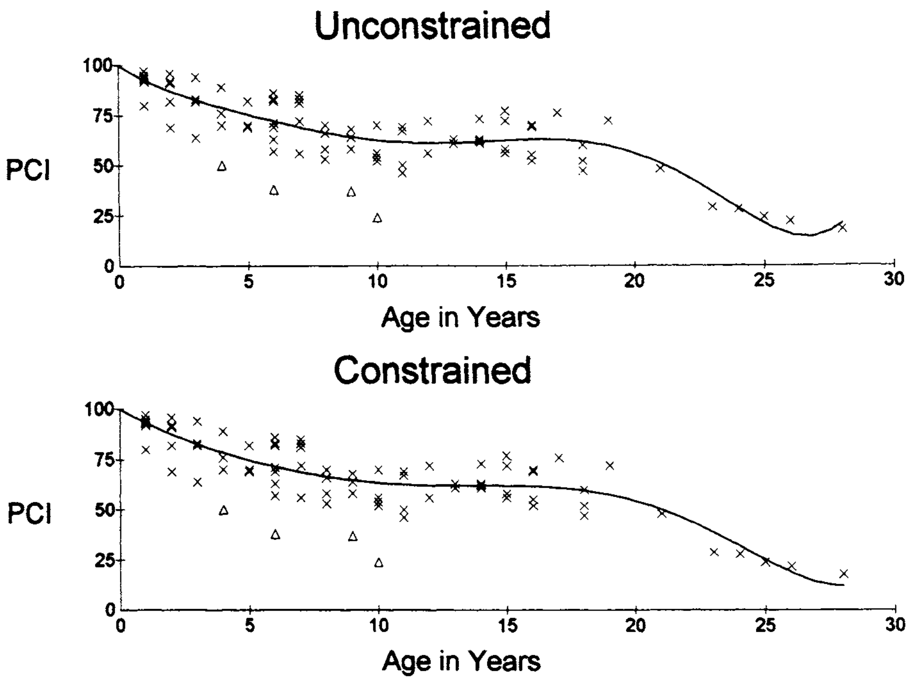

Figure 7-4. Example Constrained and Unconstrained Fourth-Degree Curve.

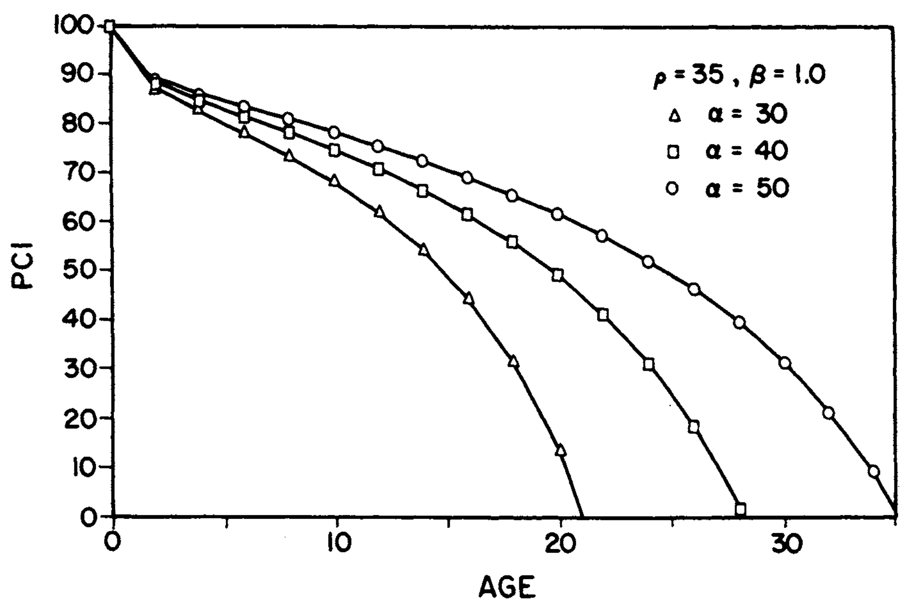

Figure 7-5. S-Shaped Curve, Effect of a Parameter (From Smith 1986).

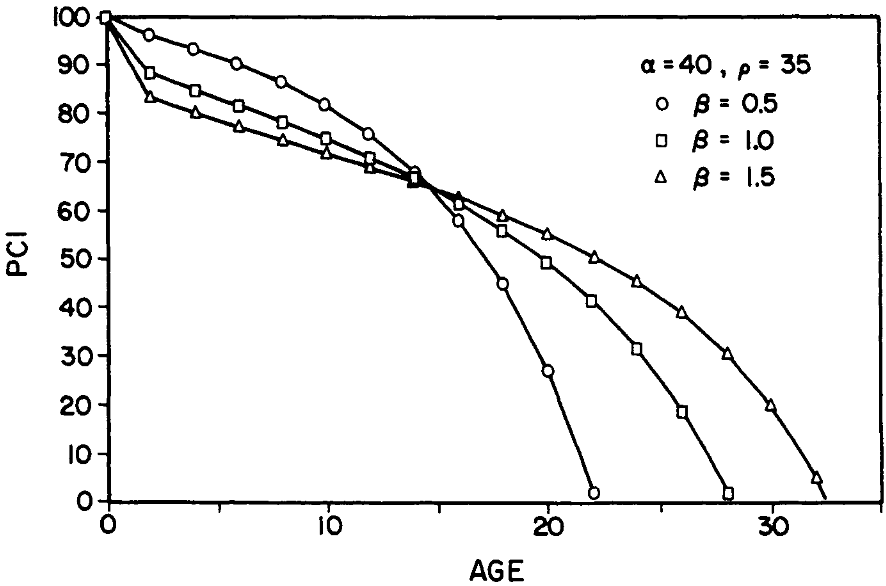

Figure 7-6. S-Shaped Curve. Effect of B Parameter (From Smith 1986).

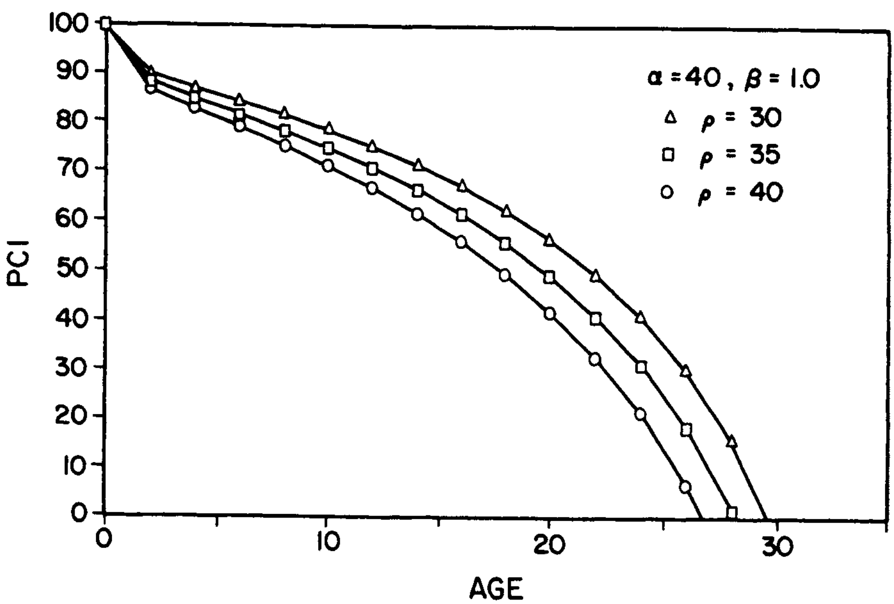

Figure 7-7. S-Shaped Curve, Effect of R Parameter (From Smith 1986).

7.2.6 Probability Distribution

A pavement condition measure such as the PCI or IRI can be treated as a random variable with probabilities associated with its values. A probability distribution describes the probabilities associated with all the values of a random variable. For example, if the random variable is the PCI, then its probability distribution can be described by its cumulative distribution function as shown in Figure 7–8. The vertical axis in Figure 7–8 is the probability of the PCI being less than or equal to a given value "pci." Figure 7–9 shows the Cumulative Distribution Function (CDF) at different points in time of the pavement life. This figure can be presented as probability vs. time for a selected PCI value in what is known as the "survivor curve" (Fig. 7–10). The concept of survivor curves has been presented by R.L. Lytton (1987). The probability drops off with time from a value of 1.0 down to 0 and it expresses the percentage of pavement that remains in service with a PCI greater than a selected value.

The use of probability distribution in predicting pavement condition requires the knowledge of the distribution law for the variable being predicted. This technique is particularly useful for individual distress prediction.

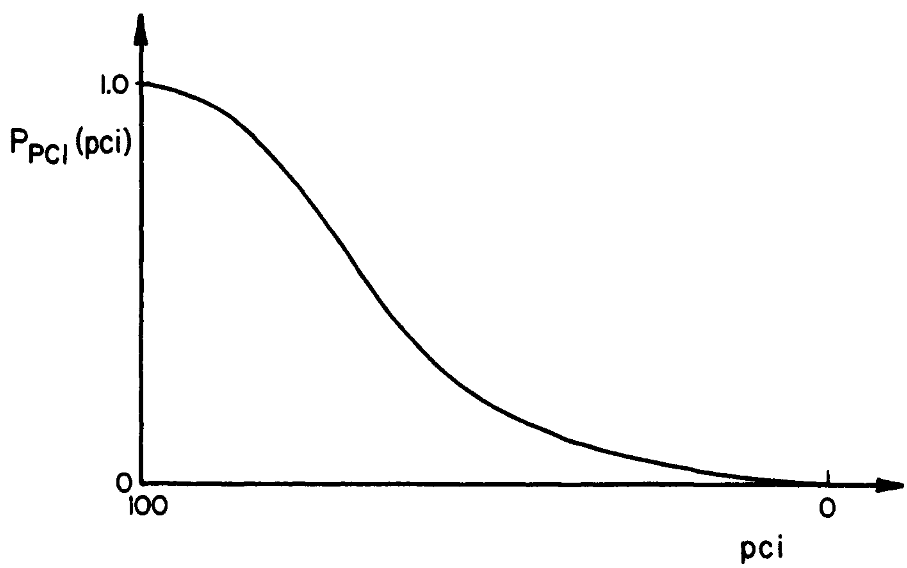

Figure 7-8. Cumulative Distribution Function.

7.2.7 Markovian

The Markovian technique has been described in detail by Butt (1991). In this technique, a pavement condition measuring scale is divided into discrete intervals called condition states. For example, the PCI (0 - 100) can be divided into 10 condition states each 10 points wide. The PCI condition states will be used to illustrate the Markovian technique; however, the same can be repeated for any other condition measure.

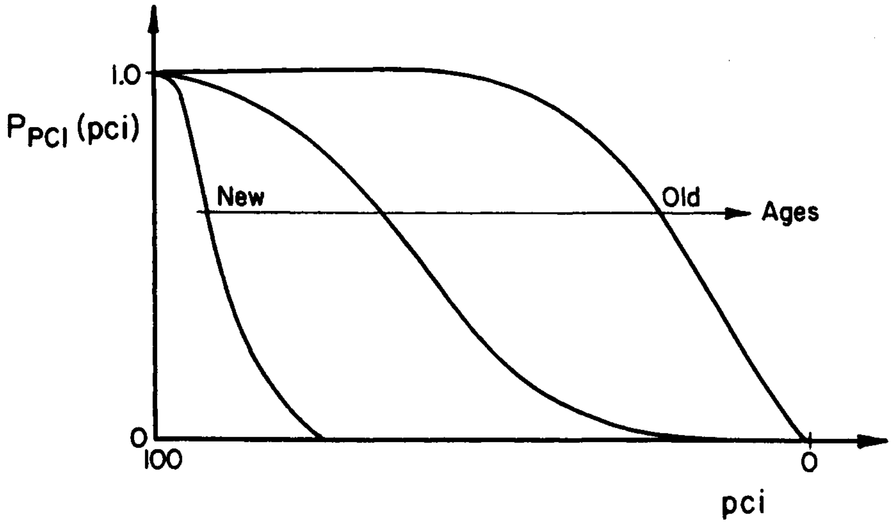

Figure 7-9. Cumulative Distribution Function at different points in pavement life.

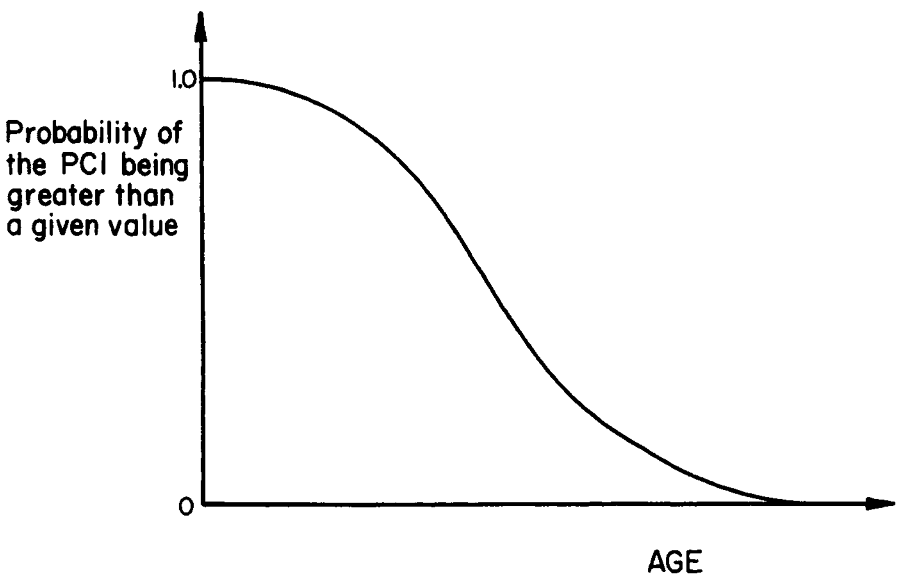

Figure 7-10. Probability vs. Time for a Given PCI Value.

The technique is based on determining the probabilities associated with pavement in a given condition state either staying in that state or deteriorating to the next state, after one duty cycle. A duty cycle can be a 1-year effect of weather and traffic loading, or a similar measure. A state vector indicates the probability of a pavement section being in each of the states in a given duty cycle. This is illustrated in Figure 7–11. At age 0 (duty cycle 0), the state vector is (1, 0, 0, 0, 0, 0, 0, 0, 0, 0,), which means that it is assumed that the probability is 1.0 that a pavement section will be in state 1 at duty cycle 0.

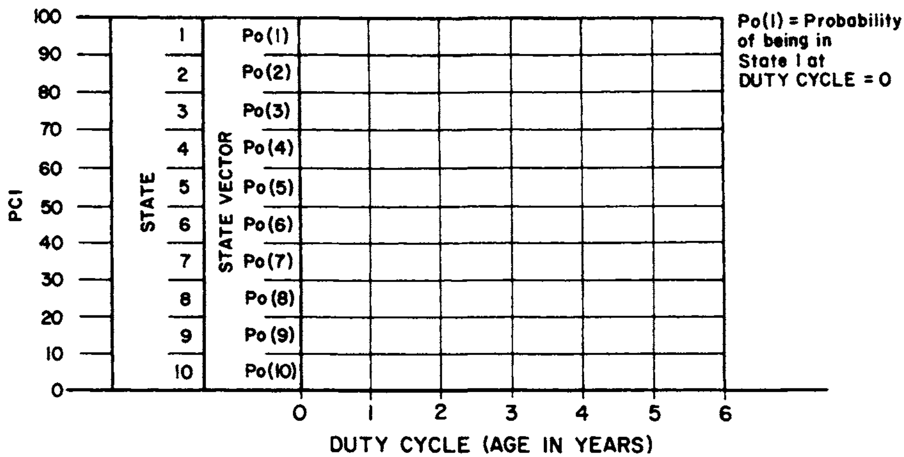

Figure 7-11. Probability of a Pavement Section Being in Each of the States in Given Duty Cycle.

The technique requires developing a probability transition matrix to predict the way the pavement deteriorates with time. If the assumption is made that the pavement condition will not drop by more than one state in a single duty cycle, the probability transition matrix can be written as follows:

$$P = \begin{bmatrix} p(1) & q(1) & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & p(2) & q(2) & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & p(3) & q(3) & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & p(4) & q(4) & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & p(5) & q(5) & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & p(6) & q(6) & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & p(7) & q(7) & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & p(8) & q(8) & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & p(9) & q(9) \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

Where $p(j)$ is the probability of a road staying in state j during one duty cycle and $q(j) = 1 - p(j)$ is the probability of a road transiting down to next state ($j + 1$) during one duty cycle.

The entry of 1 in the last row of the transition matrix corresponding to state 10 indicates a "holding" or "trapping" state. The pavement condition cannot transit from this state unless repair action is performed.

The state vector for any duty cycle, t, is obtained by multiplying the initial state vector $\tilde{p}(0)$ by the transition matrix P raised to the power of t. Thus,

$$\begin{aligned}
\tilde{p}(1) &= \tilde{p}(0) \times P \\
\tilde{p}(2) &= \tilde{p}(1) \times P = \tilde{p}(0) \times P^2 \\
\ldots & \quad \ldots \quad \ldots \quad \ldots \quad \ldots \\
\tilde{p}(t) &= \tilde{p}(t-1) \times P = \tilde{p}(0) \times P^t
\end{aligned} \tag{7-17}$$

With this procedure, if the transition matrix probabilities can be estimated, the future state of the road at any duty cycle, t, can be predicted.

The transition matrix probabilities, $p(1)$ through $p(9)$, are estimated so as to minimize the prediction error as compared to actual observations. In his doctoral thesis, Dr. Butt (1991) used a nonlinear programming approach to estimate the probabilities.

Some inherent assumptions in using this technique (Lytton 1987) should be pointed out:

1. The probability of making the transition from one state to another depends only on the present state.
2. The transition process is stationary; the probability of changing from one state to another is independent of time.

The second assumption is most likely not to be true (acceptable) for pavements because it implies that changes in weather or loading condition within a planning horizon will not affect the transition probabilities (rate of deterioration). Dr. Butt (1991) nearly eliminated this problem by introducing a zoning scheme with each zone representing a period of time within which assumption 2 is valid. Figures 7–12 and 7–13 show the quality of fit using the single zone and the new technique of multiple zones, respectively.

7.3 Prediction Models Used in Micro PAVER

An extensive research program conducted at the U.S. Army CERL resulted in the development of what is called the Family Method (Nunez and Shahin 1986, Shahin and Walther 1990). The method consists of the following steps:

1. Define the pavement family
2. Filter the data
3. Conduct data outlier analysis
4. Develop the family model
5. Predict the pavement section condition

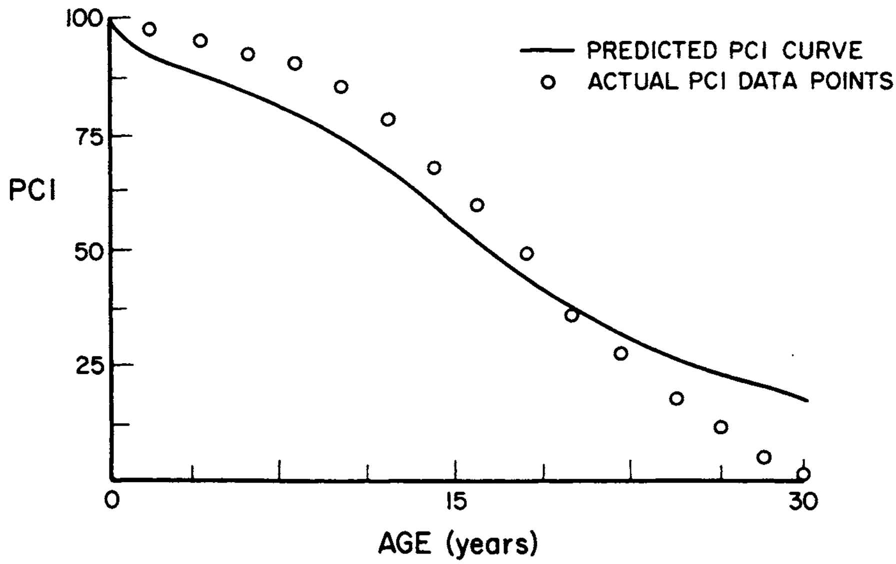

Figure 7-12. Markovian Single Zone Prediction (From Butt 1991).

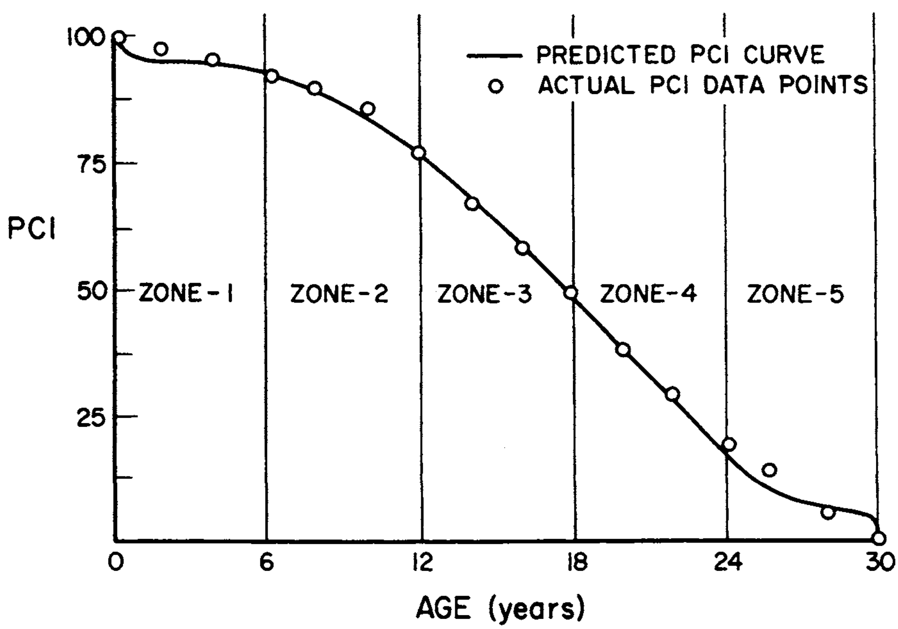

Figure 7-13. Markovian Multiple Zone Prediction (From Butt 1991).

The method was designed for use with Micro PAVER to predict PCI vs. time. However, the concept can be extended to predict other condition measures. A description of each of the above steps as used in Micro PAVER follows.

7.3.1 Define the Pavement Family

A pavement family is defined as a group of pavement sections with similar deterioration characteristics. The Micro PAVER system allows the user to define a family based on several factors including use, rank, surface type, zone, section category, last construction date, and PCI. Figure 7–14 is an example family definition using three of the factors: use, type, and rank. The user may define as many families as required for accurate condition prediction. Data availability may impose a limitation on appropriate family definition. For each defined family, a data file is automatically created by Micro PAVER containing pavement section identification, age, and PCI.

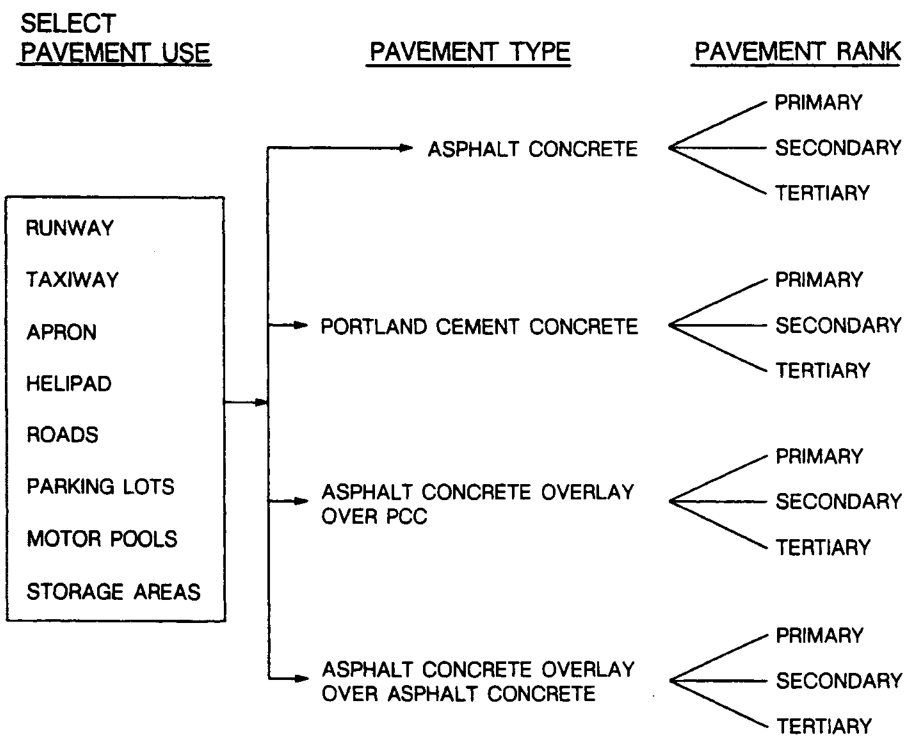

Figure 7-14. Family Definition.

7.3.2 Filter the Data

In this step, Micro PAVER allows the user to filter out suspicious data points. The data are first sorted by pavement section identification number, age, and PCI. When the same section is listed more than once, sequential cases of the same section are compared. If the PCI increases with age and the increase is greater than 20 points, the case with the higher PCI is moved to the "errors" file. This action indicates that either an error is present in one of the records or that major rehabilitation has been performed between condition surveys, which would place this section in a different family of pavements. If a pavement section of the same age is listed more than once and the PCIs are the same, only one pavement section is retained. If the PCIs are different for the same section and age, all cases are moved to the "error" file.

A further check on suspicious data is done using a set of boundaries defined by a maximum and minimum envelope expected over the life of the pavements. The program includes a default envelope developed by reviewing many databases; however, the user can easily modify these values. If a record falls outside the envelope boundaries, the record is moved to the "errors" file. Figure 7–15 shows example output from the filter procedures.

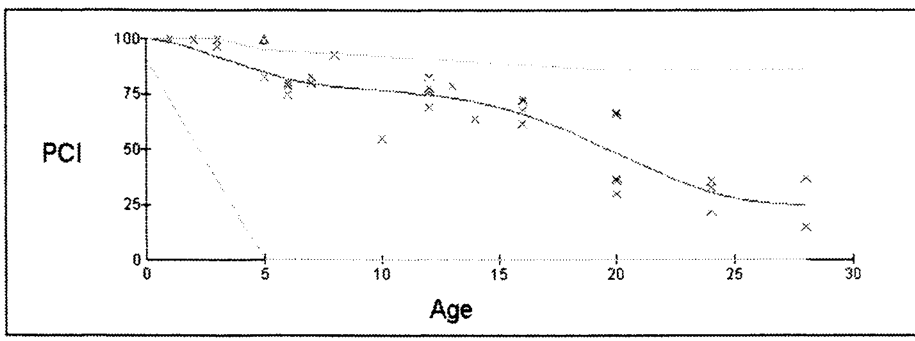

Figure 7-15. Example Output from Filter Process.

7.3.3 Conduct Data Outlier Analysis

The data-filtering procedure is used to remove obvious errors in the data as described above. Further examination of the data for statistical removal of extreme points is performed in the outlier analysis. This step is important because pavements with unusual performance can have a substantial impact on the way family behavior is modeled.

Micro PAVER calculates the prediction residuals, which are the differences between the observed and predicted PCI values using a fourth-degree polynomial least- error curve. The residuals were found to have a normal frequency distribution, (Nunez, M. N. and Shahin, M. Y. 1986), which allowed a confidence interval to be set. For example, an interval of three standard deviations in both directions contains 99.8 % of the observed PCIs. Micro PAVER allows the user to specify the confidence interval. Sections that are detected as outlier based on the confidence intervals are shown as circles in Figure 7–16. These section records are identified and removed to the outlier error file.

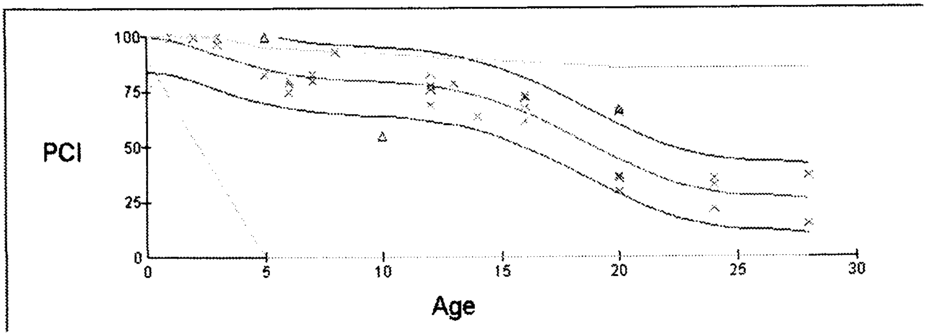

Figure 7-16. Example Output from Outlier Process.

7.3.4 Develop the Family Model

A fourth-degree polynomial constrained least squared error is developed using data after being processed through the filtering and outlier analysis. This polynomial is constrained in that it is not allowed to have a positive slope because the PCI cannot increase with age. At the request of the user, an unconstrained best fit can be viewed if a positive PCI vs. age slope is detected. This is a useful feature because it may imply a nonhomogeneous family. It also helps the user view where the problem is occurring.

This best-fit curve for the family analysis extends only as far as the available data. To predict future conditions, the curve is extrapolated by extending a tangent of the same slope as that of the curve at the last few years (currently set to 3 years).

7.3.5 Predict the Pavement Section Condition

PCI prediction at the section level uses the curve from the pavement family prediction model. The prediction function for a pavement family represents the average behavior of all the sections of that family. The prediction for each section is done by defining its position relative to the family prediction curve. It is assumed that the deterioration of all pavement sections in a family is similar and is a function of only their present condition, regardless of age. A section prediction curve is drawn through the latest PCI/age point for the pavement section being investigated, parallel to the family prediction curve as shown in Figure 7–17. The predicted PCI can then be determined at the desired future age.

Comparing the section to the family deterioration provides invaluable feedback on the effect of maintenance, traffic, drainage, and other factors on pavement behavior. This type of feedback is invaluable as a guide for revising pavement thickness design procedures. The family method was developed so that when more data are incorporated into the database, the deterioration model is continuously updated.

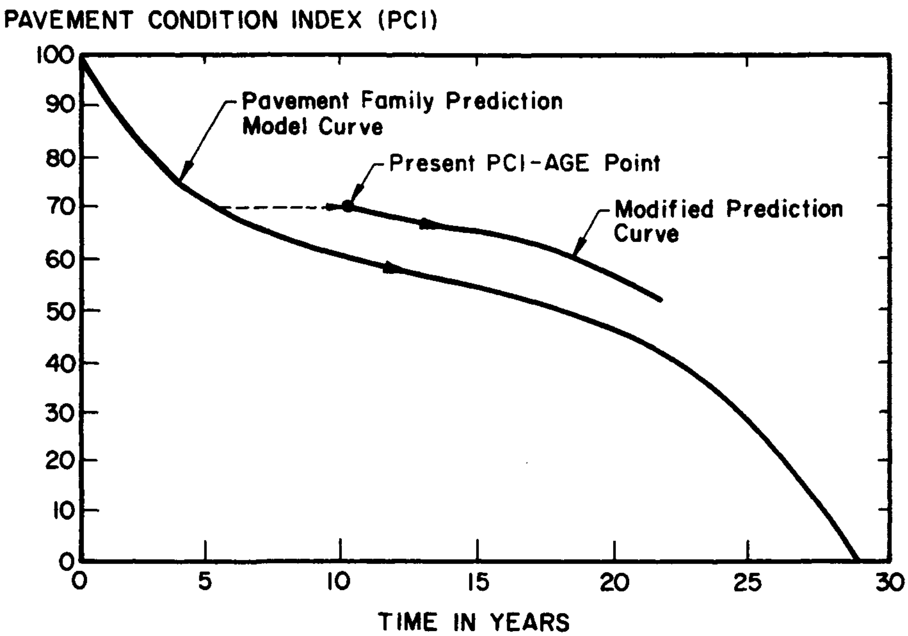

Figure 7-17. Pavement Section Prediction in Relation to Family Model.

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Smith, R. E. (1986). Structuring a Microcomputer Based Pavement Management System for Local Agencies. Ph. D. Thesis, University of Illinois at Urbana-Champaign, IL.

U.S. Army Engineering Research and Development Center-Construction Engineering Research Laboratory (ERDC-CERL), 2004. Micro PAVER Pavement Management System, 2004. e-mail: paver@cecer.army.mil web: www.cecr.army.mil/paver

8

Overview of Maintenance and Rehabilitation Methods

This chapter provides an overview of available maintenance and rehabilitation (M&R) methods. Knowledge of these methods is important to understanding and appreciating the next two chapters on network and project level management. This chapter does not discuss all available M&R methods, nor does it discuss the methods in full detail. A more detailed presentation of M&R techniques can be found in the references listed at the end of this chapter.

Maintenance and rehabilitation is also called maintenance and repair by some agencies. Therefore the "R" in "M&R" can be interpreted as either rehabilitation or repair. M&R methods are presented in this chapter under three categories: localized, global, and major. Localized M&R includes patching and crack sealing; global M&R includes applying fog seals and slurry seals; and major M&R includes overlays and recycling.

Many M&R projects combine methods to address the cause of the problem(s) rather than just fix the symptoms. For example, if PCC slab corner breaks occurred as a result of pumping and loss of support, patching the corner breaks alone will not correct the problem. Instead, a project level evaluation (see Chapter 11) should be conducted and the appropriate M&R alternative selected. The alternative may consist of slab undersealing, full-depth patching of the corner breaks, and joint sealing.

In the following sections, each M&R method is defined, a situation that warrants its consideration is identified, and its design or technique of application is described.

8.1 Localized M&R

Localized M&R can be applied either as a safety (stop-gap) measure or preventive measure. Common localized M&R methods are presented in the following sections.

8.1.1 Asphalt Pavement Localized M&R Methods

8.1.1.1 Crack Sealing (AC Pavement)

Definition

Crack sealing is the process of cleaning and sealing or resealing of cracks in AC pavement.

Use

This technique is used to fill longitudinal and transverse cracks, including joint reflection cracks from underlying PCC slabs, that are wider than 1/8 in. The primary purpose of crack sealing in AC pavement is to prevent surface water infiltration into the pavement foundation. It is more cost effective to use this technique as a preventative measure when the overall pavement condition is good or better. Sealing cracks in a deteriorated pavement is not cost effective. This will be further discussed in Chapter 10 under the critical PCI concept.

Design/Technique

The technique consists of the following steps:

1. Remove old sealant and form a sealant reservoir. Use a vertical spindle router or hand tools.
    a. Remove the loose material along edges.
    b. The sealant reservoir depth should be at least the width of the crack plus ¼ in.
2. After routing, clean the crack using compressed air (do not sandblast). Vacuum or sweep up the debris.
3. Apply sealant. Do not overfill the crack; fill to 1/8 in. below the pavement surface.

8.1.1.2 Full-Depth Patching (AC Pavement)

Definition

This technique involves replacing the full depth of the AC layer and may include replacement of the base and subbase layers.

Use

Full-depth patching is used to repair structural and material related distresses such as alligator cracking, rutting, and corrugation. In the case of slippage cracking where the failure may be limited to the top AC layer, the depth of the patch may be limited to the top AC layer if it can be removed easily.

Design/Technique

The AC full-depth patching technique is illustrated in Figure 8–1. The technique consists of the following steps:

1. Square off the area to be patched and mark off at least 6 in. to 12 in. beyond the distressed area. Make cuts to form straight lines with vertical sides. The patch boundary does not have to be rectangular.

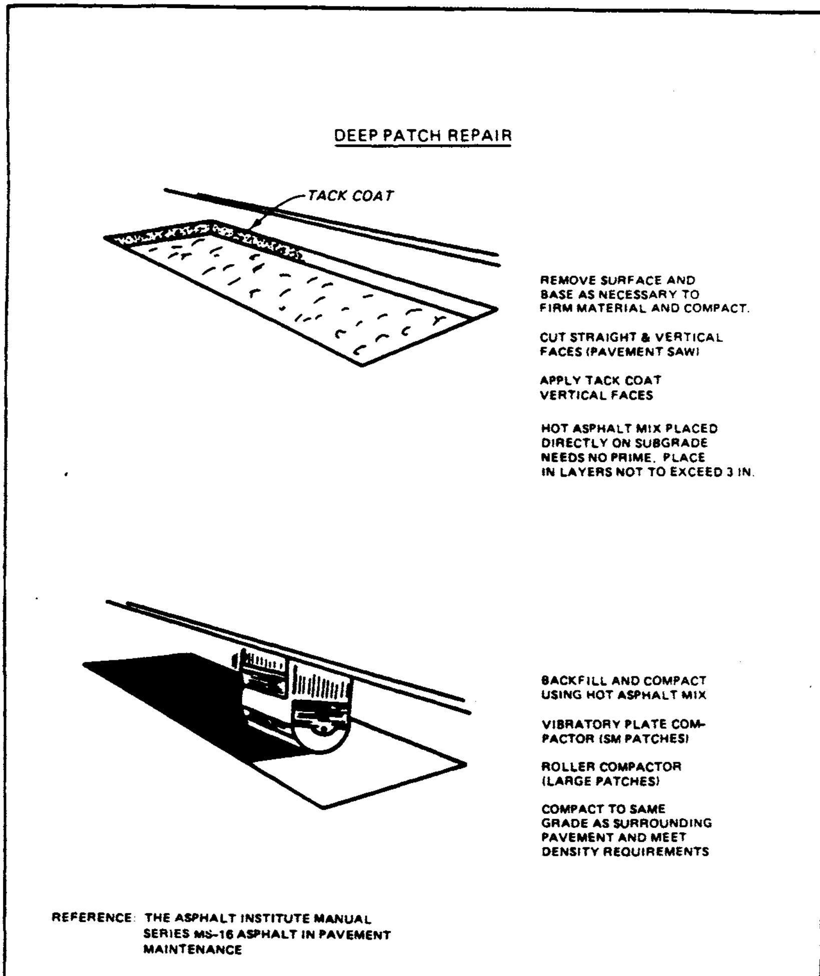


Figure 8-1. Full-Depth Patching of AC Pavement (Air Force 1992).

2. After cutting, remove material from the cut area.
   a. If base course or subgrade is damaged, remove and replace the material, and compact. As a minimum, compact the base course in place.
   b. After compaction, thoroughly clean the pavement surface outside the repair area to avoid debris getting into the tack coat or patching material.
3. Apply a light tack coat to the sides of the patch area and prime the bottom.
   a. Backfill and compact in 2 in. to 3 in. lifts with a dense-graded hot mix asphalt to the same grade as the existing asphalt. The use of a vibrating roller is strongly recommended. If rutting is present, rolling may be done transversely so that the roller will rest on the patch material and not on the old pavement.
   b. Seal 1 in. to 1 ½ in. past edges of patch to avoid water infiltration.

8.1.2 Portland Cement Concrete Localized M&R Methods

8.1.2.1 Crack Sealing (PCC Pavement)

Definition

Crack sealing is the process of cleaning and sealing or resealing cracks in PCC pavement.

Use

This technique is used to stop surface water infiltration into the pavement foundation and to stop the accumulation of incompressibles in the cracks. Water infiltration results in weakened support and eventual pumping, corner breaks, and slab shattering. Accumulation of incompressibles in cracks leads to spalling of the concrete and is a source of foreign object damage to aircraft engines.

Design/Technique

The crack filling technique is illustrated in Figure 8–2 (U.S. Air Force 1992). The technique consists of the following steps:

1. Remove old sealant if previously filled.
   a. Route the crack using a vertical spindle router (do not use a rotary impact router as it spalls the concrete).
   b. This step may be done by hand by breaking off the end of a hoe, and using the curved metal rod left at the end.
2. Different filling procedures are required based on crack width and amount of spalling for different classifications of cracks. The following procedures are recommended:
   a. Hairline to 1/8 in., no spalling: do not widen or seal. With minor spalling: blow out and seal.

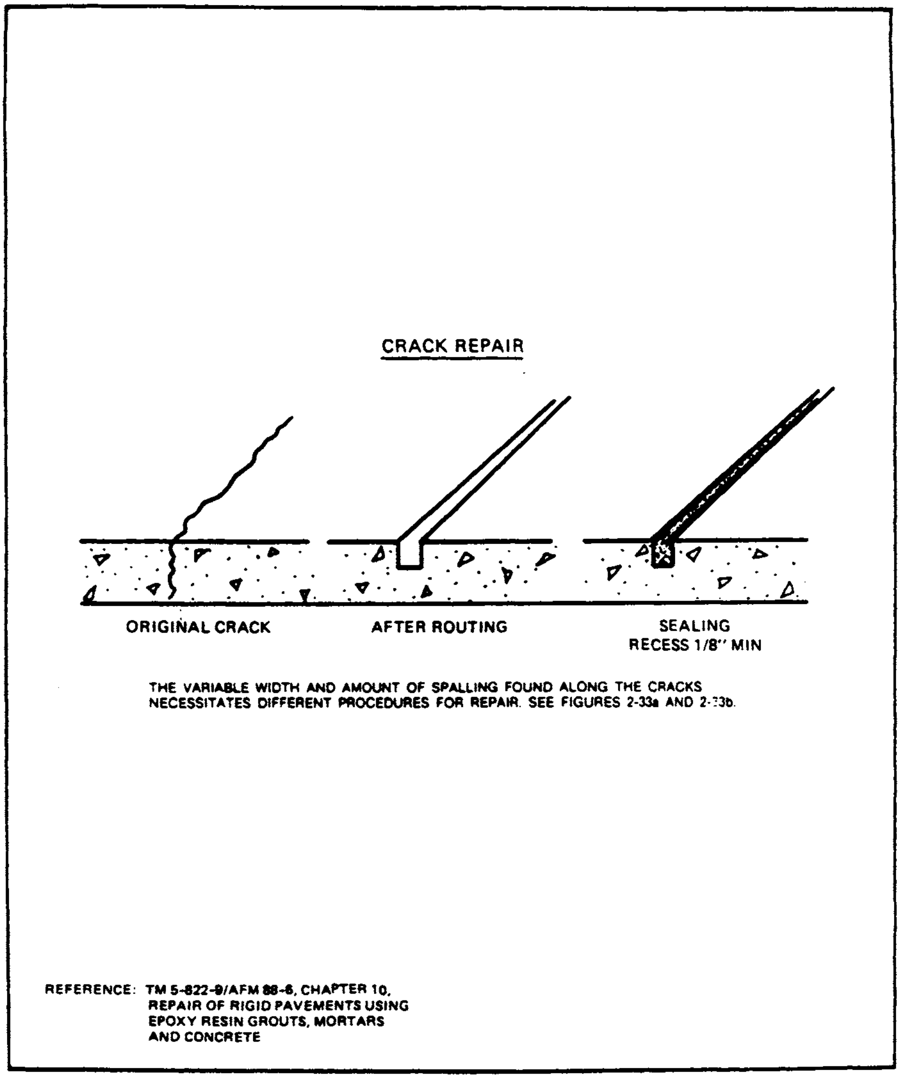


Figure 8-2. Crack Sealing of PCC Pavement (From U.S. Army and Air Force 1988).

b. 1/8 in. to 3/8 in., no spalling: if edges are rough, blow out or route before sealing. With minor spalling: route and seal.

c. 3/8 in. to 3/4 in., no spalling: route and consider using a backer rod if too deep. With major spalling: repair the spall as a joint, while maintaining the crack through the spall.

d. Greater than 3/4 in., no spalling: route and consider using a backer rod if too deep. With major spalling: rebuild crack as if it were a joint.

   e. Cracks approximately 1 ½ in. may be temporarily patched using bituminous concrete if they are not "working" cracks. For a permanent repair, rebuild as a joint.

3. After routing, sandblast the crack, then airblow and vacuum. As a minimum, airblow debris away from the crack and sweep it up.
4. Install a backer rod if required. Apply the proper sealant from bottom to top of crack.

8.1.2.2 Diamond Grinding (PCC Pavement)

Definition

Diamond grinding is the process of removing a thin layer of the existing concrete surface by grinding it with a series of closely spaced rotating diamond saw blades.

Use

This method is used to reprofile jointed concrete pavements that have developed a rough ride because of faulting or slab warping. It is also used to restore transverse drainage and to provide a textured pavement surface.

Design/Technique

Diamond grinding should be performed before faulting becomes excessive; otherwise, grinding will be expensive. Other M&R that may be needed before diamond grinding include:

1. Full- and partial-depth patching.
2. Slab undersealing.
3. Load transfer restoration.

Joint and crack sealing is performed after diamond grinding.

8.1.2.3 Full-Depth Patching (PCC Pavement)

Definition

This type of M&R involves full-depth replacement of part or all of a PCC slab. When the entire slab is replaced, it is called "slab replacement."

Use

Full-depth patching is used to repair a variety of distresses, most of which occur near joints or cracks. Such distresses include corner breaks and "D" cracking. When a full-depth patch is performed adjacent to a joint or crack, the load transfer across the joint or crack should be restored. Deterioration of a reflected joint or crack in an asphalt concrete overlay is also a candidate for full-depth patching of the underlying concrete pavement.

Design/Technique

The full-depth patching technique is illustrated in Figure 8–3 and consists of the following steps:

1. Square off the area to be patched, including all underlying deterioration. Deterioration near joints may be larger at the bottom of the slab by as much as a few feet. This may be verified by coring as part of the project evaluation (Chapter 10). To prevent rocking of the patch, it is recommended that the minimum dimension be >4 ft (for low-volume traffic) or 6 ft (for high-volume traffic). The minimum dimension is also a function of whether load transfer devices are used. The following is recommended for the design of the patch sides:

    a. Use deformed tie bars for sides away from joints.

    b. Use dowel bars or restore existing load transfer at transverse joints.

    c. Use butt joints along longitudinal joints. Other designs are possible, but they should be engineered to prevent premature failure of the patch.

2. Saw the panel boundaries, except when one of the boundaries is a joint. Partial- or full-depth sawing may be used. Partial-depth sawing leaves some aggregate interlock, but allows potential spalling at the bottom of the patch during breakup and removal of the concrete. Breakup of the concrete should start from the center of the patch using gravity or pneumatic air hammers. To avoid damage to the adjacent concrete, ball breakers should not be used.

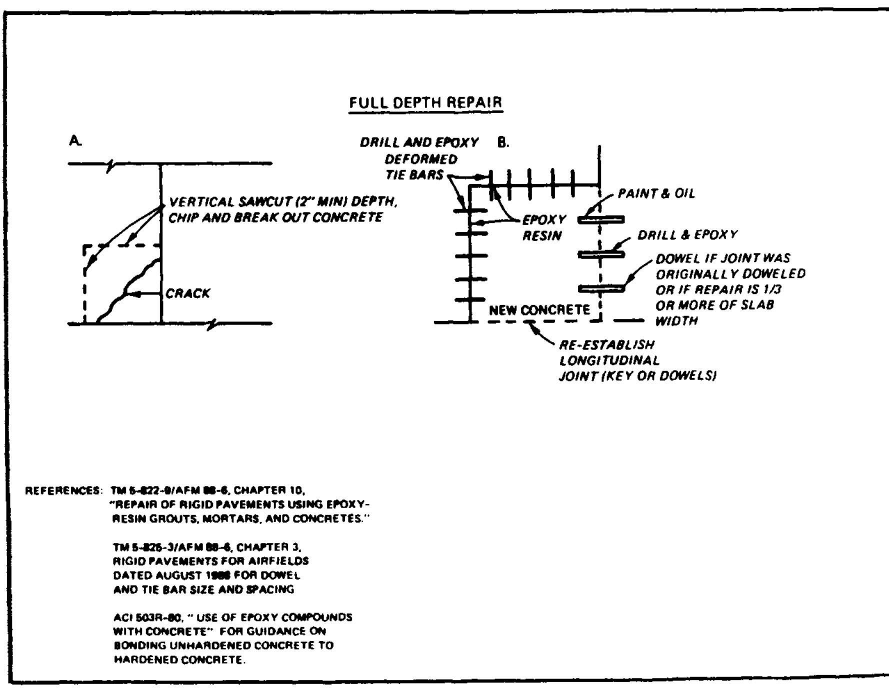


Figure 8-3. Full-Depth Patching of PCC Pavement (U.S. Army and Air Force 1988).

3. Remove all broken concrete and prepare the foundation. Excessive moisture should be removed or dried. Granular subbases are difficult to compact; therefore, partial or full replacement of the granular subbase with concrete is recommended.
4. Straighten or realign existing deformed tie bars and dowels. Install new tie bars or dowels as required by drilling holes in the face of the existing slab.
5. Place the concrete and ensure the edges receive good vibration. If the patch cannot be closed to traffic for several days, use an early strength concrete mix that contains a higher cement content and additives.

8.1.2.4 Joint Sealing (PCC Pavement)

Definition

Joint sealing is the process of cleaning and sealing or resealing PCC joints.

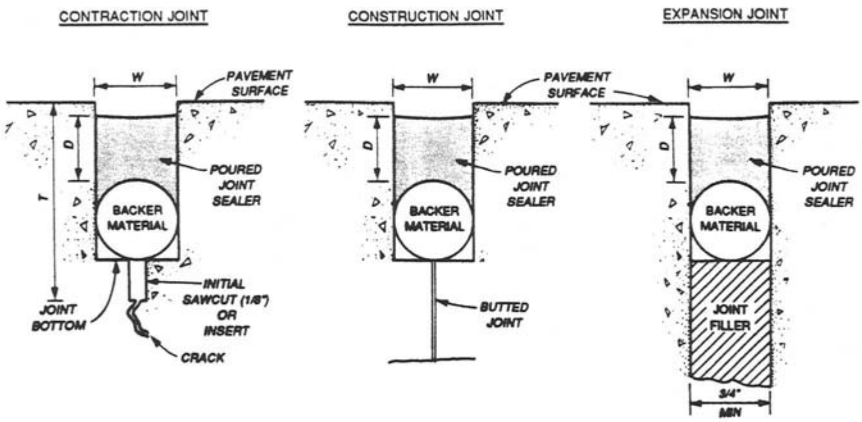
Use

This technique is used to stop surface water infiltration into the pavement foundation and to stop the accumulation of incompressibles in the joints. Water infiltration results in weakened support and eventual pumping, corner breaks, and slab shattering. Accumulation of incompressibles in joints leads to spalling of the concrete and is a source of foreign object damage to aircraft engines.

Design/Technique

The joint sealing technique is illustrated in Figure 8–4 (U.S. Army and Air Force 1988). The technique consists of the following steps:

1. Remove the old joint material with a joint plow attachment. The plow blade must not be rigid or V-shaped, as it will spall the concrete. High pressure water may be used as an alternative. Removal depth is typically twice the final width of the joint, averaging 1 in.
2. Preformed elastomeric compression seals may be removed by hand if they are short in length. If long seals are to be removed, start removal by hand, then attach it to a tractor to pull it out.
3. Airblow the joint and vacuum the debris. As a minimum, blow debris away from the joint and sweep it up.
4. Sawing or refacing (as required).
   a. The joint must be refaced if the sides are not vertical or if you need to widen the joint to a specified width and depth for proper shape factor and proper sealant bonding. The shape factor is a ratio of depth to width and should be between 0.5 to 2.0 to minimize stresses in the sealant (U.S. Army and Air Force 1988). Do not widen a joint more than 3/4 in., unless it is an expansion joint.



W - WIDTH OF SEALANT RESERVOIR (SEE TABLE)
D - DEPTH OF SEALANT (1.0 TO 1.5 X W)
T - DEPTH OF INITIAL SAWCUT OR INSERT TYPE JOINT FORMER (CONTRACTION JOINT)

TABLE

JOINT SPACING FT	WIDTH, IN.	
	MIN	MAX
<25	1/2	5/8
25 - 50	3/4	7/8
>50	1.0	1-1/8

NOTE: TOP OF SEALANT WILL BE 1/8-IN. TO 1/4-IN. BELOW TOP OF PAVEMENT.

REFERENCE: TM 5-825-3/AFM 88-6, CHAPTER 3, RIGID PAVEMENTS FOR AIRFIELDS

Figure 8-4. Joint Sealing of PCC Pavement (From U.S. Army and Air Force 1988).

b. If preformed compression seals are removed, refacing is not required unless the joint width is too small.

c. Joints that are severely spalled should be reconstructed using partial-depth patch procedures.

5. Following refacing, clean by sand- or water-blasting to remove all debris from joint. As a minimum, thoroughly airblow the joint and vacuum the debris.

6. Install a backer rod or separating medium at the proper depth, ensuring that the material is not twisted or stretched.

7. Apply the proper sealant using a pressure injection wand.
   a. The sealant MUST be applied from the bottom up. DO NOT overfill the joint. It should be 1/8 in. below the joint surface to avoid extrusion.
   b. If no pressure wand is available, use a pour pot with extreme caution to ensure the joint is adequately sealed.

8.1.2.5 Partial-Depth Patching (PCC Pavement)

Definition

Partial-depth patching involves removing shallow localized areas of deteriorated or spalled PCC pavement and replacing it with a suitable patch material such as cement concrete or epoxy concrete.

Use

This technique is used to repair PCC pavement distresses that are confined to the top few inches of the slab, such as joint and corner spalling.

Design/Technique

The partial depth-patching technique is illustrated in Figure 8–5 (U.S. Air Force 1992) and consists of the following steps:

1. Square off the area to be patched and mark off approximately 3 in. beyond the distressed area. Use a concrete saw to make a vertical cut a minimum of 2 in. deep around the marked area. Ensure cuts (both at the corners and along the edges) intersect to obtain a rectangular or square patch with vertical sides.
2. After cutting, break out the area with pneumatic drills or jackhammers down to sound concrete. After the unsound concrete is removed, blow out the hole with compressed air to remove residual material and dust. Then thoroughly pressure rinse the area.
3. Use a stiff broom or brush to apply a bonding grout approximately 1/16 in. thick over the entire area to be patched immediately before the concrete patch material is placed (epoxy grout for epoxies). If the material does not use a bonding grout, use a large brush to dampen the surface (no standing water) with water before placing material.

   Bonding grout mixture is 1 part Portland cement, 1 part fine sand, and no more than 5 gallons of water per sack of cement.
4. Insert a thin strip of wood, joint fiberboard, or oil-coated metal, for the new joint. Scoring the fiber board about ¾ in. from the top will make routing or removing the board easier. Another alternative is to completely cut and grease the material, removing it when the patch material is set.

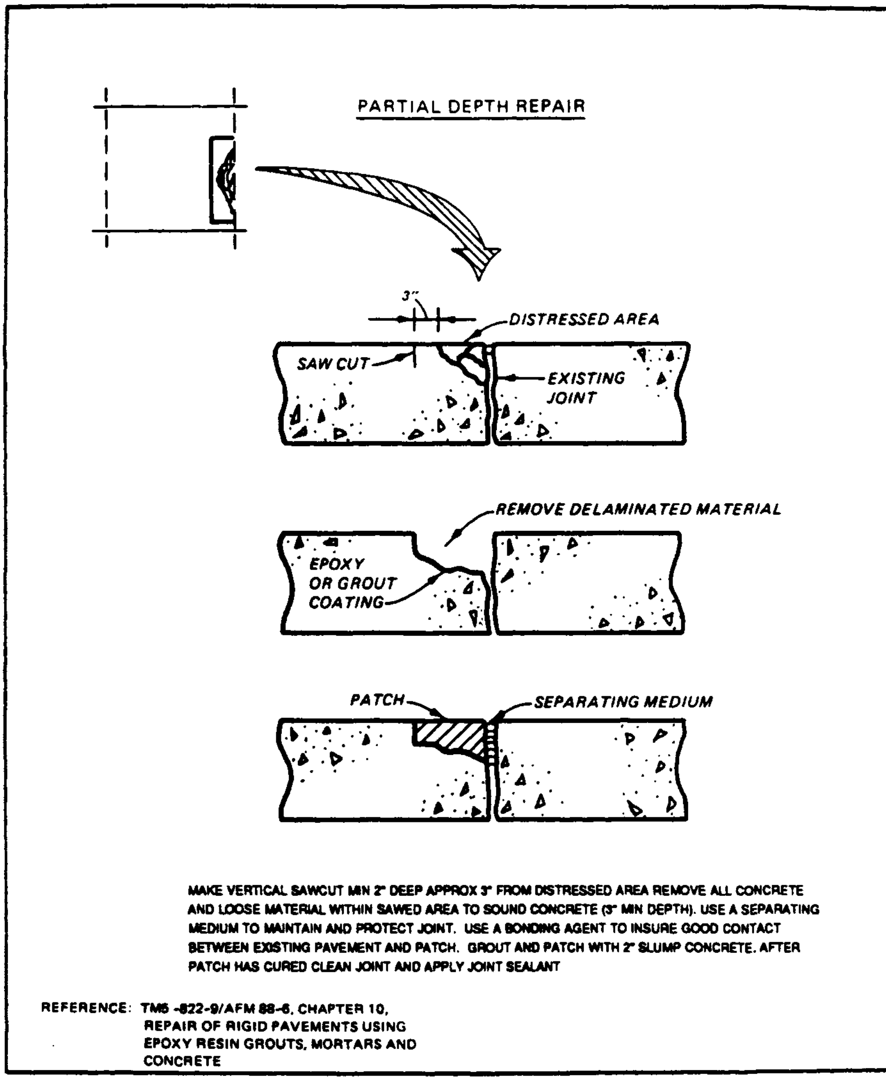


Figure 8-5. Partial-Depth Patching of PCC Pavement (From U.S. Army and Air Force 1988).

5. Mix, place, and finish the concrete. All concrete type materials should be extended with aggregate to match the existing concrete as closely as possible. Do not drop the concrete from heights greater than 12 in. Ensure that you have estimated for enough aggregate/patching material, plus extra, before mixing begins.

   a. When finishing has been completed, broom finish to match adjacent areas.

   b. After concrete has attained its initial set, remove filler board, if used.

6. Cure the patch a minimum of 3 days (unless using epoxy, polymer, etc.) using some type of pigmented curing compound.
7. Route out and clean the joint, finally filling the joint with the appropriate filler material.

8.1.2.6 Slab Undersealing Using Cement Grout (PCC Pavement)

Definition

This technique involves the pumping of cement grout underneath concrete slabs (or below the base if the base is stabilized) to fill voids and provide support.

Use

It is used when the results of project level evaluation (Chapter 10) indicate the presence of voids. This is normally determined based on deflection measurement analysis and on observing pumping, corner breaks, and faulting. Slab undersealing is a preventive measure and should be followed by joint sealing and reestablishment of the load transfer if needed. It should be applied as soon as distresses are observed. Applying slab undersealing to a deteriorated pavement is not cost effective except if it is performed as a preparation for an overlay.

Design/Technique

The first step in a slab undersealing is to determine whether the undersealing will be a blanket coverage or will be limited to areas with detected problems. Once the areas to be undersealed have been decided, holes are drilled to allow cement grout to be pumped into the slab. The holes are drilled about 2 to 4 ft from joints and spaced 6 to 12 feet apart, depending on field conditions. Pump the grout until it shows through adjacent joints or cracks, but stop immediately if the slabs are raised. It should be emphasized that slab undersealing should be done by experienced contractors.

8.2 Global M&R

Global M&R is cost effective when applied as a preventive measure (Chapter 10). Common global M&R methods are presented in the sections that follow.

8.2.1 Aggregate Surface Treatment (AC Pavement)

Definition

This technique involves applying an asphalt binder followed by a layer of aggregate, which is rolled into the binder. This process is also known as a chip seal. If sand is used instead of aggregate, the treatment is called a sand seal.

Use

It is used to provide a surface seal or skid-resistant surface to structurally sound pavement. This treatment is best suited to low-volume roads. Multiple treatments may be applied up to 1 in. thick. The cost, however, approaches that of a thin hot mix overlay. Some agencies consider applying a thin overlay as surface treatment.

Design/Technique

Suggested asphalt and aggregate quantities for a single surface treatment are shown in Figure 8–6. Multiple surface treatments are achieved by repeating the procedure for a single surface treatment but using smaller aggregate in each successive application (normally reduce the aggregate maximum size by 50%). Rapid-setting asphalt emulsions are normally used as a binder. The cover aggregate should be one size. The aggregate should also be clean and cubical. The use of elongated aggregates may result in the submersion of the aggregate into the asphalt and consequent bleeding. Even though Figure 8–6 provides approximate rates of the application for the asphalt and aggregate, it is recommended that a design method be used to compute the rates. Several design methods are available (FHW 1979), which may be supplemented by local experience.

Nominal Aggregate Size Range, in.	AASHTO Aggregate Designation	Quantites	
		Aggregate[a,b] lb/sy	Asphalt[a,c] gal/sy
3/4 to 3/8	6	40-50	0.40-0.50
1/2 to No. 4	7	25-30	0.30-0.45
3/8 to No. 8	8	20-25	0.20-0.35
No. 4 to No. 16	9	15-20	0.15-0.25
Sand	M-6	10-15	0.15-0.20

[a] The lower application rates of asphalt shown are for aggregates on the fine side of the spec limits. Higher application rates are for aggregates on the coarse side of the spec limits.

[b] Weights are based on an aggregate specific gravity of 2.65. If the specific gravity is >2.75 or <2.55, multiply the table by the aggregate specific gravity divided by 2.65.

[c] Asphalt is an emulsion of acceptable grade and type.

Figure 8-6. Suggested Asphalt and Aggregate Quantities for Single Surface Treatments and Seal Coats (FHWA 1979).

8.2.2 Fog Seal (AC Pavement)

Definition

This technique involves the spraying of a light coat of a bituminous material (0.03 to 0.05 gallon per square yard) on the surface of an existing pavement using a distributor.

Use

It is used to prolong the life of an asphalt concrete pavement by helping to reduce raveling and to improve waterproofing. Fog seals are especially good for treating pavements that carry little or no traffic. Without traffic, asphalt concrete pavements tend to ravel and harden faster than pavements that support moderate to heavy traffic.

Design/Technique

The material most frequently used is a SS-1 and SS-1h asphalt emulsion, which is normally heated to 150° F before application. A fog seal should be applied when the ambient temperature is above 40° F. Warmer temperatures are desirable so the emulsion will break faster. The pavement should be closed to traffic for 12 to 24 hours to allow the material to cure.

The asphalt emulsion can be applied at full strength or be diluted as much as one part emulsion to ten parts water. Normal application is a dilution of one part emulsion to at least four parts of water.

8.2.3 Rejuvenators (AC Pavement)

Definition

Rejuvenators are proprietary bituminous materials that are sprayed on the surface of an existing pavement using a distributor.

Use

Similar to fog seals, rejuvenators seal and waterproof asphalt concrete surfaces. Rejuvenators, however, penetrate the asphalt concrete and soften the asphalt binder. This added capability is of great value in reducing the rate of hardening of the asphalt concrete surface and thus reducing the severity of temperature cracking. In a study conducted by the U.S. Army Corps of Engineers (Brown and Johnson 1976), five materials including SS-1 asphalt emulsion were evaluated. Five pavement sections were treated by the materials and were compared to untreated control sections to determine relative performance. Figure 8–7 shows the relationship between material type and cracking wider than ¼ inch three years after application. The materials are identified as A, B, C, D, and E where A, B, C, and D are proprietary rejuvenators and E is the SS-1 asphalt emulsion. The figure clearly shows the beneficial effect of rejuvenators A, B, and C. It can also be noticed that the beneficial effect is more for the inside traffic lane where the traffic volume is less than the outside lane. The rejuvenators did not have much effect on the total amount of cracking after three years, but only on the wider cracks.

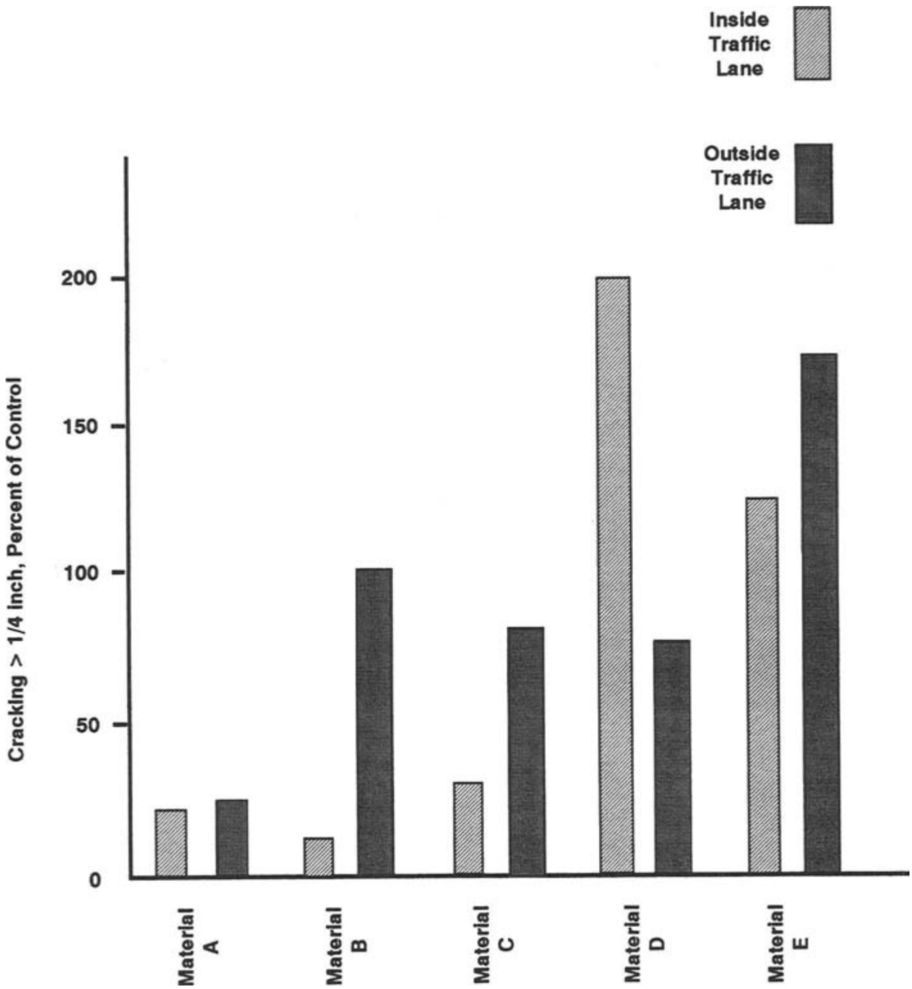


Figure 8-7. Relationship Between Material Type and Cracking Wider than ¼ inch (Brown 1988).

<u>Design/Technique</u>

One of the commonly used rejuvenators is Reclamite (Witco 1994). Rejuvenators should only be applied as recommended by the manufacturers. They should also be applied by experienced contractors. The rate of application may vary between 0.02 and 0.08 gal/sy based on the voids in the existing asphalt concrete surface mixture; the higher the voids, the higher the rate of application. If a higher rate of rejuvenator is used than should be, some of the rejuvenator will not penetrate the surface and skid resistance will be reduced significantly. If excess rejuvenator remains on the surface after 24 to 48 hours, it should be sanded and removed. Rejuvenators should not be applied to a pavement surface such as slurry seal or surface treatment that has a large amount of asphalt near the surface. Application of rejuvenators to these surface types may result in a sticky, soft surface (Brown 1988).

8.2.4 Slurry Seal/Micro Surfacing (AC Pavement)

Definition

Slurry Seals are a specially prepared mixture of asphalt emulsion, well-graded fine aggregate, water, and mineral filler. Micro Surfacing is a slurry seal where the asphalt emulsion is replaced with a polymer-modified asphalt emulsion.

Use

Slurry Seals are used to provide a surface seal or skid resistant surface to structurally sound pavement. Slurry seal will fill small cracks (less than 1/8 in. wide). Larger cracks need to be individually treated before the application of a slurry seal. In addition, Micro Surfacing is used to provide transverse surface leveling prior to overlays.

Design/Technique

A continuous mix slurry seal machine (Fig. 8–8), spreader box, and a pneumatic-tired compaction roller are used to apply the slurry seal. Emulsions of varying compositions and setting times are mixed with any one of three gradations of aggregate (Fig 8–9), to create slurry seal mixes for specific uses, Roberts et al. 1996. Type I slurry had the finest gradation and is primarily used for filling fine surface cracks, provides highest crack penetration, and provides a thin surface seal (less than 1/8 inch thick). It is most suitable

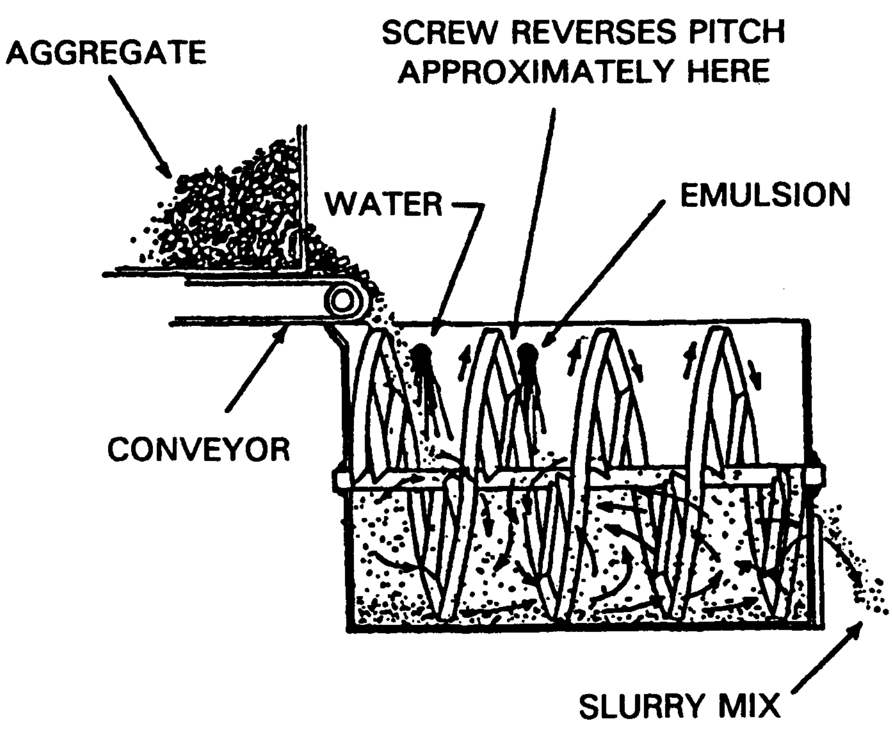


Figure 8-8. Flow Diagram of a Continuous Slurry Mixer (Roberts et al. 1996).

SIEVE SIZE		TYPE I PERCENT PASSING	TYPE II PERCENT PASSING	TYPE III PERCENT PASSING	STOCKPILE TOLERANCE
3/8	(9.5 mm)	100	100	100	
# 4	(4.75 mm)	100	90 - 100	70 – 90	± 5%
# 8	(2.36 mm)	90 – 100	65 - 90	45 – 70	± 5%
# 16	(1.18 mm)	65 – 90	45 - 70	28 – 50	± 5%
# 30	(600 um)	40 – 65	30 - 50	19 – 34	± 5%
# 50	(330 um)	25 – 42	18 - 30	12 – 25	± 4%
#100	(150 um)	15 – 30	10 - 21	7 – 18	± 3%
#200	(75 um)	10 – 20	5 - 15	5 – 15	± 2%

Figure 8-9. Gradations of Aggregate.

for low density/low-wear traffic areas such as parking lots. Type II slurry is the most commonly used and has a maximum size aggregate of approximately 1/4 inch. It is best suited where raveling has occurred, and to improve skid resistance. Type III, with maximum aggregate size of 3/8 inch, is used for severe raveling and skid resistance.

Micro Surfacing uses types II and III aggregate gradations. It is more suited for moderate to heavy traffic and for improving skid resistance. The aggregate has to be crushed stone to provide better resistance to skid and rutting.

Slurry Seal/Micro Surfacing should only be applied when the temperature is above 50 degrees Fahrenheit and no freezing occurs within 24 hours.

8.3 Major M&R

Major M&R is used to correct or improve structural and functional requirements. Major M&R is often economically justified for deteriorated pavements, pavements deteriorating at a rapid rate, and pavements subjected to a change in traffic loading. The M&R techniques presented below are normally combined to form appropriate M&R alternatives. For example, cold milling, cold or hot recycling, and overlaying can all be combined into one alternative.

8.3.1 Cold Milling (AC or PCC Pavement)

Definition

Cold milling is the removal of a given thickness of the surface layer using a machine containing a rotary drum with teeth.

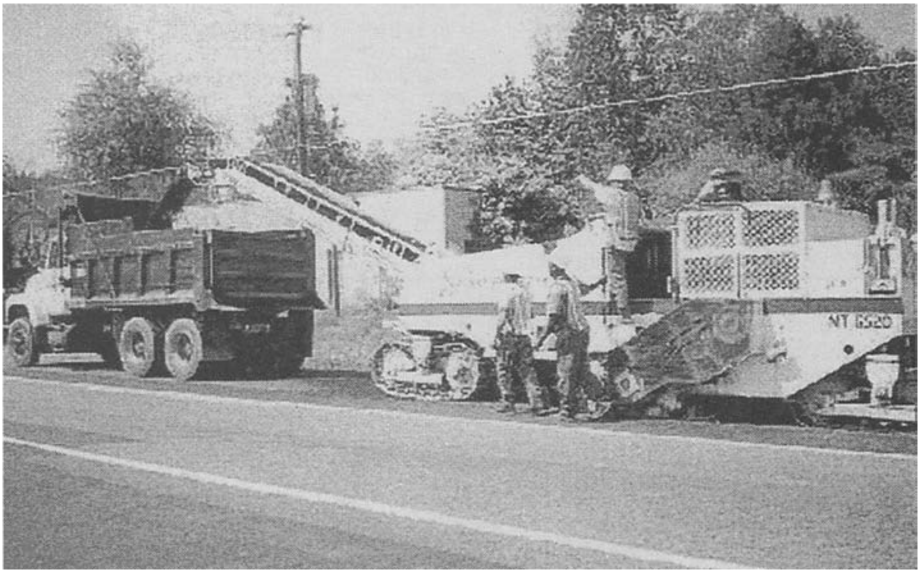


Figure 8-10. Cold Milling Machine.

Use

It is used in asphalt pavement to bring the pavement grade to an acceptable level, remove a deteriorated layer, and to provide good bonding with the overlay. The most frequent use of cold milling is in the recycling process (see Sections 8.3.2 and 8.3.3). The existing asphalt concrete material can be removed, blended with new aggregates and asphalt, and replaced. Cold milling can also be used in concrete pavement rehabilitation, especially when performing a bonded PCC overlay where the surface is cold milled and a cement grout is applied before the overlay.

Design/Technique

A cold milling machine (Fig. 8–10) can be used to remove up to 3 to 4 in. of asphalt concrete in one pass. The asphalt concrete can be removed full depth or to any desired depth. The machine has grade control devices and can accurately control the grade of the milled surface with the use of stringlines. Cold milling can be performed in any type of weather, except when the reclaimed material is to be used in recycling; milling in wet weather may cause excessive moisture in the reclaimed asphalt.

8.3.2 Cold Recycling (AC Pavement)

Definition

Cold recycling is the use of reclaimed asphalt pavement and additional water or asphalt without the use of heat to produce a paving mixture.

Use

It is used to rehabilitate badly deteriorated pavements. Pavements that are badly cracked or exhibit extensive ravelling can be removed and used to produce recycled cold mix. The material is normally used as a base course; however, it can be used as a surface on low-volume roads.

Design/Technique

The deteriorated surface is normally removed by cold milling and taken to a central plant or stockpiled locally for reuse. The reclaimed material is mixed at a central plant with water or asphalt emulsion and placed with an asphalt paving machine and compacted. It can also be placed in a self-propelled mobile plant capable of mixing and placing cold recycled mixes in one continuous operation. Compaction is normally accomplished with a combination of steel wheel, vibratory, and rubber-tired roller. This technique works best in dry, hot weather. A small amount of moisture does not affect the pavement quality, but excessive moisture is detrimental.

8.3.3 Hot Recycling (AC Pavement)

Definition

This technique involves using reclaimed asphalt pavement from a cold milling operation, new aggregate, new asphalt cement, and a recycling agent, if needed, to produce recycled hot mix.

Use

It is used for any application for which conventional hot mix can be used.

Design/Technique

The amount of reclaimed asphalt used with hot recycled mix usually does not exceed 50% to 60%. The mix can be produced in a modified batch plant or in a drum mixer. A hot recycled mix is handled the same as a conventional hot mix.

8.3.4 Hot In-Place Surface Recycling

Definition

The rejuvenation of existing AC surface in-place is achieved by applying infrared heat, loosening the softened pavement, mixing the reclaimed material with a design admixture, and replacing and compacting the new mix. This process may or may not be followed by an overlay.

Use

It is used on hardened asphalt surfaces to rejuvenate the surface, improve the bond with overlay, and reduce or delay reflective cracking.

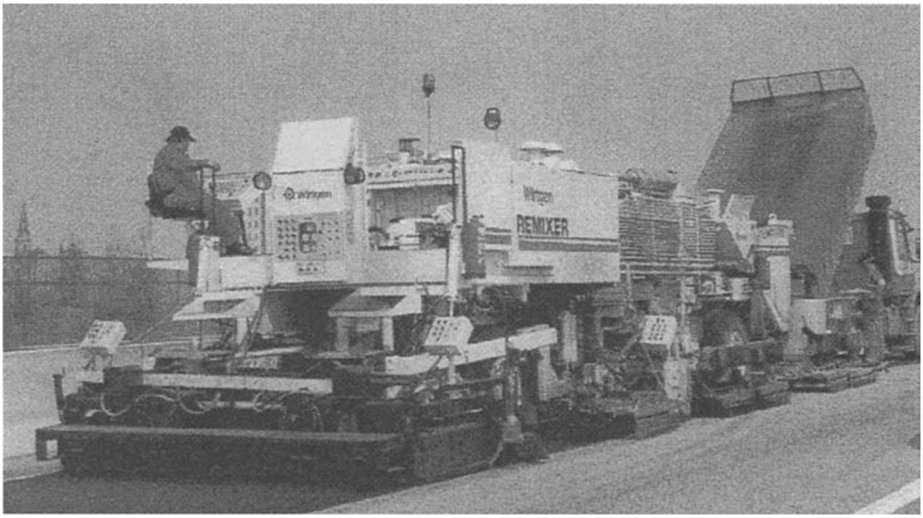


Figure 8-11. Hot Remixer Equipment.

Design/Technique

This process is not cost effective if the pavement has structurally failed because of a poor base or subgrade. The hot surface recycling process is accomplished by either a paving train or an integrated single-pass machine (Fig 8–11).

8.3.5 Cracking and Seating (PCC Pavement)

Definition

This technique involves cracking deteriorated PCC pavement into smaller pieces and seating the pieces using pneumatic-tired rollers.

Use

It is used to reduce the severity of reflective cracking after overlaying with an AC surface.

Design/Technique

The PCC pavement is broken into 18 to 24 in. pieces using a variety of equipment such as modified pile-driving, falling weight hammer, and hydraulic or pneumatic impact hammers. Equipment that produces spalling should not be used. Before cracking, any existing joint and crack sealer material should be removed to a minimum depth of 1 in. to prevent slippage of the AC overlay. The seating is performed using a heavy pneumatic roller the size of which is a function of the pavement thickness, loading, and underlying foundation strength. The tire pressure may range from 90 to 150 psi, the load on each wheel from 6500 to 25,000 lbs, and the number of coverages from 3 to 30. Before the overlay, the full depth of the cracking should be verified by coring, and large open areas should be sealed with a sand-asphalt mix. The asphalt overlay should be designed for the load but should not be less than 4 in. to minimize the amount and severity of the reflection cracks.

8.3.6 AC Overlay (AC or PCC Pavement)

Definition

This technique involves adding one or more AC layers to an existing AC or PCC pavement.

Use

It is used to correct or improve structural capacity or functional requirements such as skid resistance and ride quality. The use of an AC overlay is usually more economic when the existing pavement is still in good condition. An overlay may be combined with other M&R methods such as cold milling, cold recycling, hot recycling, and heater scarification.

Design/Technique

Several overlay thickness design approaches include: total structural requirement such as the AASHTO (American Association of State Highway and Transportation Officials) design method (AASHTO 1993), limiting deflection such as the Asphalt Institute method (Asphalt Institute MS-17), and limiting fatigue damage, which is based on mechanistic analysis (USACE-PCASE 2004). The majority of available design methods are easy to use. With the advent of powerful microcomputers, mechanistic analyses using elastic layer theory or finite element analysis are becoming user-friendly. Figure 8–12 is an example of deflection-based design method from the Asphalt Institute. The method is based on the maximum deflection as measured with the Benkelman Beam. Measurements made by a Falling Weight Deflectometer (FWD) are converted to Benkelman Beam measurements. If a correlation has not been locally developed, the FWD measurements are multiplied by a conversion factor of 1.6. The deflections are then reduced to a representative rebound deflection (RRD) using the following equation:

$$RRD = (X + 2S) * C * F \tag{8-1}$$

where:

X = average deflection from project testing (recommended minimum of 10 points)

S = deflected standard deviation

C = critical season adjustment factor

F = temperature adjustment factor (Fig. 8–13)

The critical section adjustment factor is a function of the location; more variation is expected in areas subjected to frost. Figure 8–14 is an example for one location.

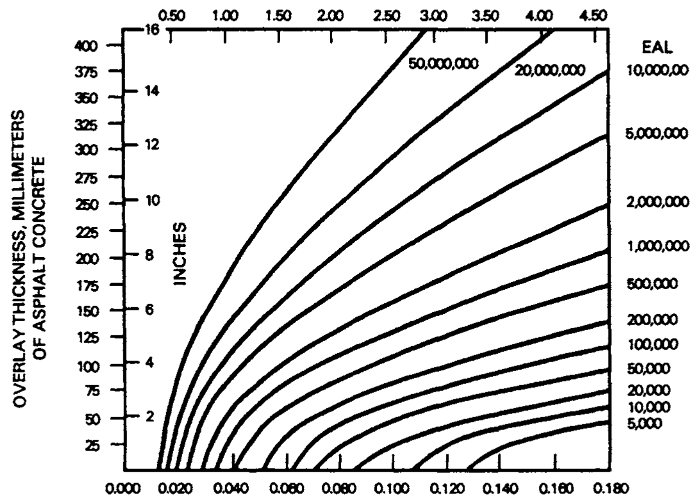


Figure 8-12. Asphalt Concrete Overlay Thickness Required to Reduce Pavement Deflections from a Measured to a Design Deflection Value (Rebound Test) (Asphalt Institute MS-17).

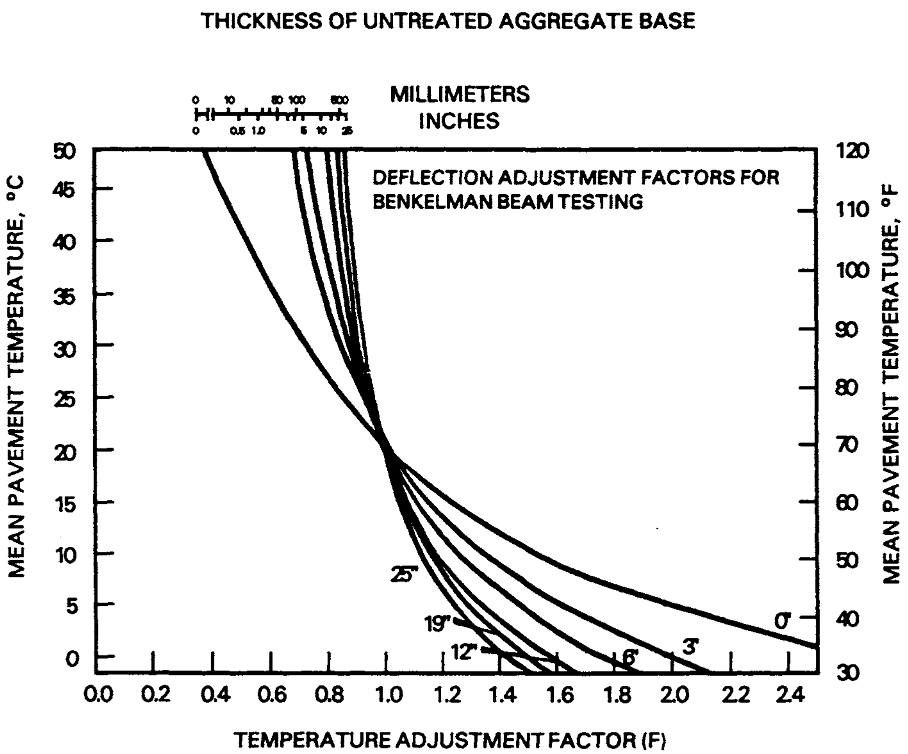


Figure 8-13. Average Pavement Temperature vs. Benkelman Beam Deflection Adjustment Factors for Full-Depth and Three-Layered Asphalt Concrete Pavements (Asphalt Institute MS-17).

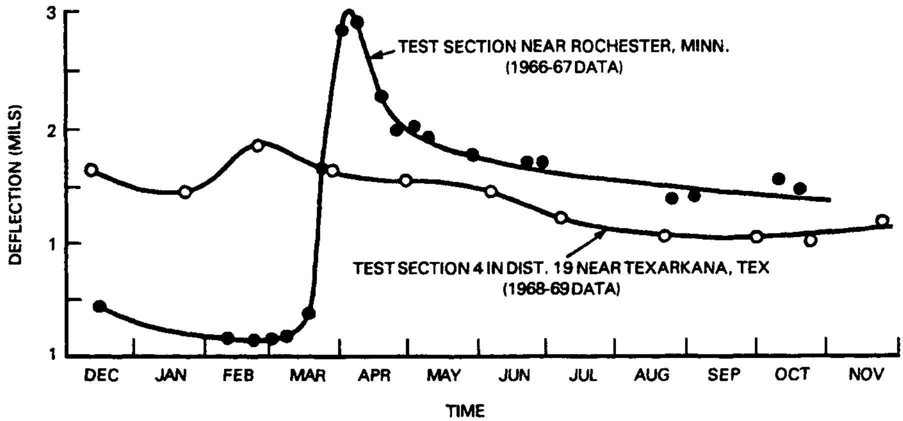


Figure 8-14. Illustration of the Effect of Geographic Location on Seasonal Variations in Deflections (Asphalt Institute MS-17).

8.3.7 PCC Pavement Overlay (AC or PCC Pavement)

Definition

This technique involves adding a PCC layer over an existing AC or PCC pavement.

Use

It is used to correct or improve the structural capacity or functional requirements such as skid resistance and ride quality. PCC overlay is mostly used over an existing PCC pavement.

Design/Technique

The three types of PCC overlay over PCC pavement are: unbonded, partially bonded, and fully bonded. Design approaches vary from total structural requirements to mechanistically based. Figure 8–15 is an example of overlay design procedure based on the total structural requirement approach.

8.3.8 Reconstruction (AC or PCC Pavement)

Definition

Reconstruction is the removal and replacement of existing pavement structure.

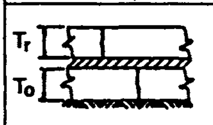
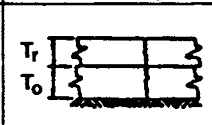
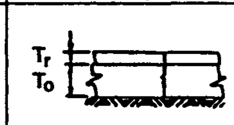
Use

It is used when the existing pavement is badly deteriorated and is based on economic analysis justification.

Design/Technique

The process is similar to designing and constructing a new pavement.

CONCRETE OVERLAYS ON CONCRETE PAVEMENT

	Unbonded or Separated Overlay	Partially Bonded or Direct Overlay	Bonded or Monolithic Overlay
TYPE OF OVERLAY			
PROCEDURE	Clean Surface Debris and Excess Joint Seal Place Separation Course-Place Overlay Concrete	Clean Surface Debris and Excess Joint Seal and Remove Excessive Oil and Rubber-Place Overlay Concrete	Scarify all Loose Concrete, Clean Joints, Clean and Acid Etch Surface-Place Bonding Grout and Overlay Concrete
Matching of Joints in Overlay & Pavement: Location / Type	Not Necessary Not Necessary	Required Not Necessary	Required Required
Reflection of Underlying Cracks to be Expected	Not Normally	Usually	Yes
Requirement for Steel Reinforcement	Requirement is Independent of the steel in existing pavement or condition of existing pavement.	Requirement is Independent of the steel in Existing Pavement. Steel may be used to control cracking which may be caued by limited Non-Structural Defects in Pavement	Normally not used in thin overlays. In thicker overlay steel may be used to supplement steel in existing pavement.
Formula for Computing Thickness of overlay (T_r) Note: T is the Thickness of monolithic pavement required for the design load on the existing support C is a structural condition factor T should be based on the flexural strength of	$T_r = \sqrt{T_n^2 - C T_e^2}$ Overlay Concrete	$T_r = \sqrt[1.4]{T_n^{1.4} - C T_e^{1.4}}$ Overlay Concrete	$T_r = T_n - T_e$ Note: The ability of the overlaid slab to transfer load at the joints should be assessed separately Existing Concrete
Minimum Thickness	6"	5"	7"

Applicability of Various Overlay Types			Unbonded or Separated Overlay	Partially Bonded or Direct Overlay	Bonded or Monolithic Overlay
	Structural condition of existing pavement	No Structural Defects C = 1.0*	YES	YES	YES
		Limited Struct. Defects C = 0.75*	YES	Only if defects can be repaired	Only if defects can be repaired
		Severe Struct. Defects C = 0.35*	YES	NO	NO
	Surface Cracks scaling, spalling and Shrinkage cracks	Negligible	YES	YES	YES
		Limited	YES	YES	YES
		Extensive	YES	NO	NO

* C Values apply to structural condition only, and should not be influenced by surface defects.

Figure 8-15. Summary of Concrete Overlays on Concrete Pavement (Ray 1967).

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9

Network-Level Pavement Management—Inventory and Condition Reporting

This chapter presents typical reporting of inventory and condition for network-level management. The reports answer questions such as: what is the pavement inventory by use (e.g., roadway, parking); what is the pavement condition in terms of the PCI, ACN/PCN, roughness, skid, etc.; how does the condition now compare to x years ago; and what would it be y years in the future if only stop-gap (safety) M&R is performed. Example Micro PAVER (ERDC-CERL 2004) reports are presented throughout the chapter.

9.1 Summary of Pavement Inventory and Condition at Last Inspection

This type of report is beneficial for quickly becoming familiar with the pavement network(s). The Micro PAVER "Summary Charts" report performs this function. In this report, a user is allowed to select a different variable for each of the x and y-axis, and the desired chart along with a summary table is automatically produced. The variables for the x and y axis are shown in Figure 9–1. Figure 9–2 is an example report output for a civil aviation airport showing the area-weighted average PCI vs. pavement use. The report can be generated for any selected condition (e.g. PCI, ACN/PCN) and for the entire database or any selected subset.

X-Axis Variables	Y-Axis Variables
Network ID	PCI
Branch Use	FOD NEW 60% (AC)
Section Rank	FOD PCC 7
Surface Type	Condition Index
Section Category	Cracking Index
Zone	Distress Index
Age at Time Report	Friction Index
Age at Last Inspection	Patching Index
Condition at Last Inspection	Roughness Index
	Rutting Index
	Structural Index
	AVG FOD RATING
	FOD MOD
	FOD AC L Crack
	Surface Deformation

Figure 9-1. X and Y–axis Variables in the Micro PAVER "Summary Charts" Report.

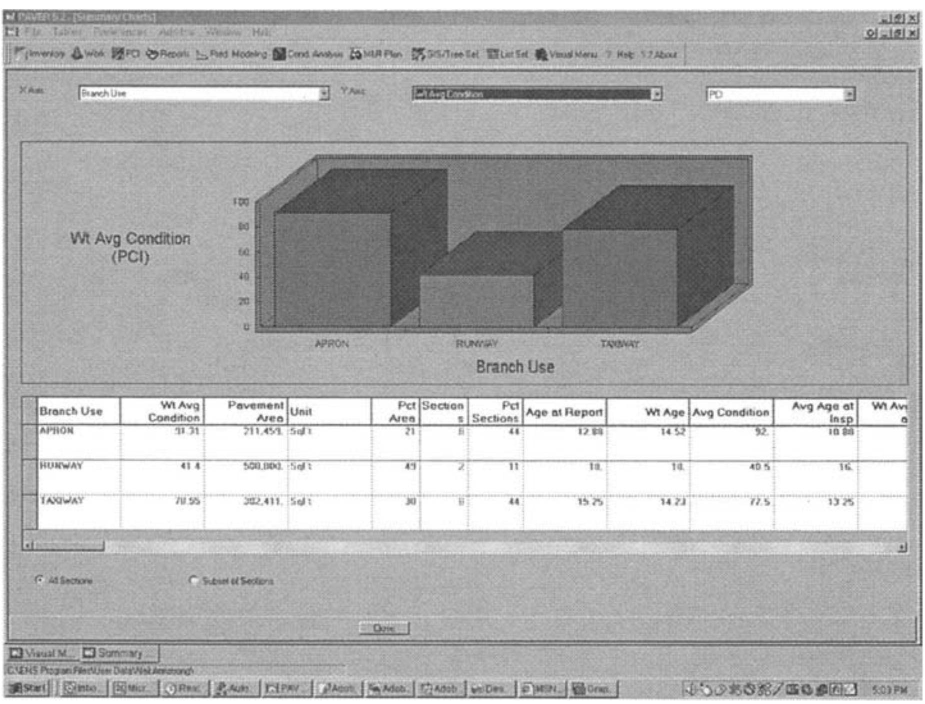


Figure 9-2. Example Micro PAVER "Summary Charts" Output for a Civil Aviation Airport.

9.2 Tabular Presentation of Pavement Condition at Last Inspection.

This provides a quick tabular presentation of pavement condition at last inspection. The condition presentation can be either at the Section or Branch level along with other relevant data. Micro PAVER reports that address these presentations are the "Section Condition Report" and "Branch Condition Report" shown in Figures 9–3 and 9–4a, respectively. Micro PAVER also provides a summary for all selected data such as shown in Figure 9–4b for the "Branch Condition Report." In this example, one can observe the clear difference between the arithmetic average PCI for the entire selected pavement (79.8) and the area weighted average PCI (62.8). The difference is due to the lower PCI for the runway, which represents about half the pavement area of the airfield pavement. Both the Section and the Branch condition reports can be generated for any selected condition such as FOD, SCI, ACN/PCN and IRI.

Date: 6 /8/2004

Section Condition Report

Pavement Database: NetworkID: NeilArms

1 of 2

Branch ID	Section ID	Last Const. Date	Surface	Use	Rank	Lanes	True Area (SqFt)	Last Inspection Date	Age At Inspection	PCI
8-26 (RUNWAY 8-26)	A	05/27/1986	AAC	RUNWAY	P	0	400,000.00	02/06/2002	16	42.00
8-26 (RUNWAY 8-26)	B	05/27/1986	AAC	RUNWAY	P	0	100,000.00	02/06/2002	16	39.00
APRON (APRON)	A	10/27/1988	PCC	APRON	S	0	46,960.00	02/06/2002	14	98.00
APRON (APRON)	B	10/27/1988	PCC	APRON	S	0	38,280.00	02/06/2002	14	91.00
APRON (APRON)	C	10/15/1988	PCC	APRON	S	0	36,320.00	02/06/2002	14	98.00
APRON (APRON)	D	01/13/1988	PCC	APRON	S	0	4,896.00	02/06/2002	14	92.00
APRON (APRON)	E	01/13/1980	PCC	APRON	S	0	29,952.00	02/06/2002	22	57.00
APRON (APRON)	F	10/01/1997	PCC	APRON	T	0	29,052.00	02/06/2002	5	100.00
APRON (APRON)	G	10/01/1999	PCC	APRON	T	0	23,542.00	02/06/2002	3	100.00
APRON (APRON)	H	11/01/2001	PCC	APRON	S	0	2,457.00	02/06/2002	1	100.00
TAXIW (Taxiway)	A1	11/13/2001	AAC	TAXIWAY	S	0	18,750.00	02/06/2002	1	100.00
TAXIW (Taxiway)	A2	11/13/2001	AC	TAXIWAY	S	0	17,500.00	02/06/2002	1	100.00
TAXIW (Taxiway)	B1	10/31/1989	AC	TAXIWAY	S	0	70,300.00	02/06/2002	13	65.00
TAXIW (Taxiway)	B2	10/27/1994	AC	TAXIWAY	S	0	58,444.00	02/06/2002	8	86.00
TAXIW (Taxiway)	B3	06/13/1999	AC	TAXIWAY	S	0	49,421.00	02/06/2002	3	96.00
TAXIW (Taxiway)	C	06/01/1980	AC	TAXIWAY	S	0	33,000.00	02/06/2002	22	61.00
TAXIW (Taxiway)	D	10/01/1979	AC	TAXIWAY	S	0	43,296.00	02/06/2002	23	78.00
TAXIW (Taxiway)	E	01/13/1967	AC	TAXIWAY	S	0	11,700.00	02/06/2002	35	34.00

Figure 9-3. Example "Section Condition Report."

Date: 6 /8/2004	Branch Condition Report Pavement Database: NetworkID: NeilArms							1 of 2
Branch ID	Number of Sections	Sum Section Length (Ft)	Avg Section Width (Ft)	True Area (SqFt)	Use	Average PCI	PCI Standard Deviation	Weighted Average PCI
8-26 (RUNWAY 8-26)	2	5,000.00	100.00	500,000.00	RUNWAY	40.50	1.50	41.40
APRON (APRON)	8	2,010.00	88.88	211,459.00	APRON	92.00	13.65	91.31
TAXIW (Taxiway)	8	6,885.00	48.38	302,411.00	TAXIWAY	77.50	21.64	78.55

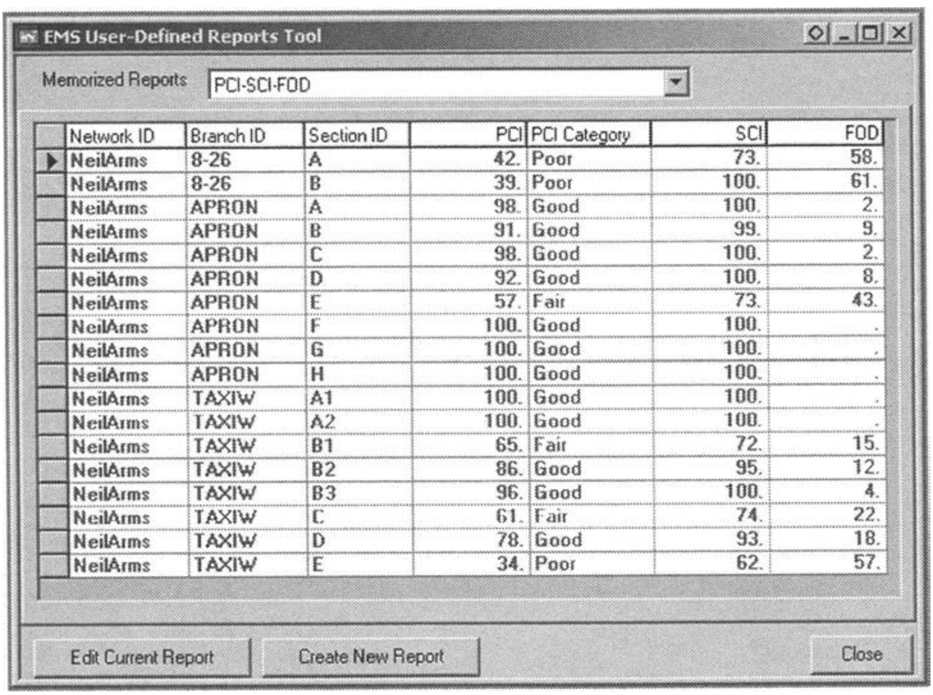
Figure 9-4a. Example "Branch Condition Report."

Use Category	Number of Sections	Total Area (SqFt)	Arithmetic Average PCI	Average PCI STD.	Weighted Average PCI
APRON	8	211,459.00	92.00	13.65	91.31
RUNWAY	2	500,000.00	40.50	1.50	41.40
TAXIWAY	8	302,411.00	77.50	21.64	78.55
All	18	1,013,870.01	79.83	23.05	62.89

Figure 9-4b. Example "Branch Condition Report" Summary Data.

9.3 User-Defined Reports

The Micro PAVER "User-Defined Reports" allow the user to define any desired report and save it for future use. The report can be defined in terms of columns, selected subset of data (rows), and the order of the rows to appear in the report. Also, the generated reports can be exported to Microsoft Excel for further processing or graphing. Figure 9–5 is an example user-defined report showing the PCI, SCI, and FOD at last inspection for the runway and taxiway sections at a civil aviation airport. Such a report is beneficial in analyzing causes of pavement deterioration and correct M&R action. For example, it can be seen that for Section "B" of runway 8-26, the PCI is 39 and the SCI is 100. It is therefore concluded that the major cause of distress is not structural. In this Section, the predominant distresses observed at inspection were raveling and block cracking.



Network ID	Branch ID	Section ID	PCI	PCI Category	SCI	FOD
NeilArms	8-26	A	42.	Poor	73.	58.
NeilArms	8-26	B	39.	Poor	100.	61.
NeilArms	APRON	A	98.	Good	100.	2.
NeilArms	APRON	B	91.	Good	99.	9.
NeilArms	APRON	C	98.	Good	100.	2.
NeilArms	APRON	D	92.	Good	100.	8.
NeilArms	APRON	E	57.	Fair	73.	43.
NeilArms	APRON	F	100.	Good	100.	.
NeilArms	APRON	G	100.	Good	100.	.
NeilArms	APRON	H	100.	Good	100.	.
NeilArms	TAXIW	A1	100.	Good	100.	.
NeilArms	TAXIW	A2	100.	Good	100.	.
NeilArms	TAXIW	B1	65.	Fair	72.	15.
NeilArms	TAXIW	B2	86.	Good	95.	12.
NeilArms	TAXIW	B3	96.	Good	100.	4.
NeilArms	TAXIW	C	61.	Fair	74.	22.
NeilArms	TAXIW	D	78.	Good	93.	18.
NeilArms	TAXIW	E	34.	Poor	62.	57.

Figure 9-5. Example "User-Defined Report" Showing PCI.

9.4 GIS Presentations

Inventory variables and conditions can be presented in a map format. Example inventory variables for map presentations are pavement surface type and pavement rank (e.g. arterial, collector, residential). Example condition indicators are PCI, SCI, FOD, ACN/PCN, and IRI. When presenting pavement condition indices, the condition ranges and labels need to be defined. For example, the PCI ranges could be 0-55, 56-70, and 71-100 with the corresponding labels poor, fair, and good, respectively. An example PCI at last inspection presentation for a small road network is shown in Figure 9–6.

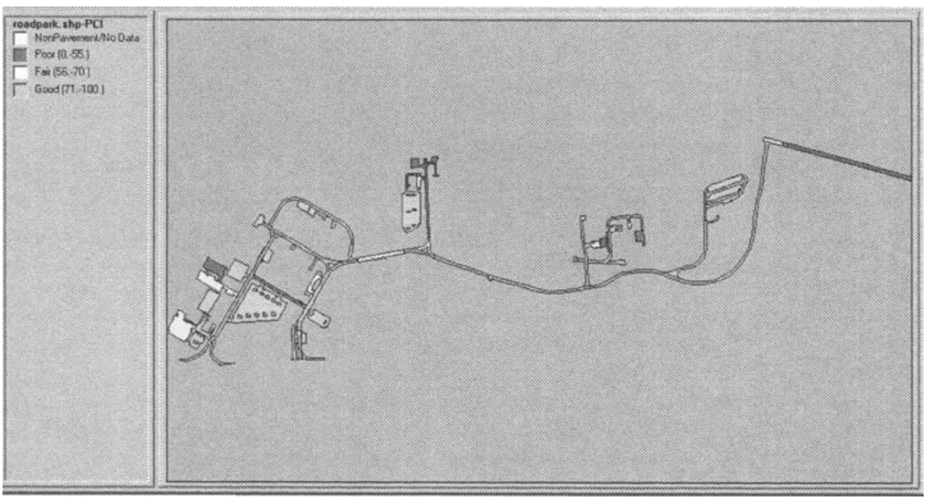


Figure 9-6. Example PCI at Last Inspection.

9.5 Pavement Condition Analysis, Past and Future

The main purpose of pavement condition analysis is to determine changes in pavement condition. Primarily, how is the pavement condition now compared to the condition *x* years in the past and what would it be *y* years in the future if no preventive or major M&R is performed? Past pavement condition (prior to last inspection date) is determined by interpolation using last construction date (last major M&R date) and previous inspections as demonstrated in Figure 9–7. Future pavement condition (after last inspection date) is determined using condition prediction techniques. In Micro PAVER, this is performed using the pavement's assigned condition deterioration family as described in Chapter 7. Condition Analysis is very beneficial to pavement managers since it provides feedback on pavement condition performance (condition over time) as a result of previous M&R budget spending and management policies. Also, being able to compare the condition of pavement sections at a selected date is an important feature since the pavement sections may have been inspected at different times. Figures 9–8a and 9–8b are example condition analyses performed for a civil aviation airport showing both a frequency diagram and map presentation of PCI condition categories at different dates.

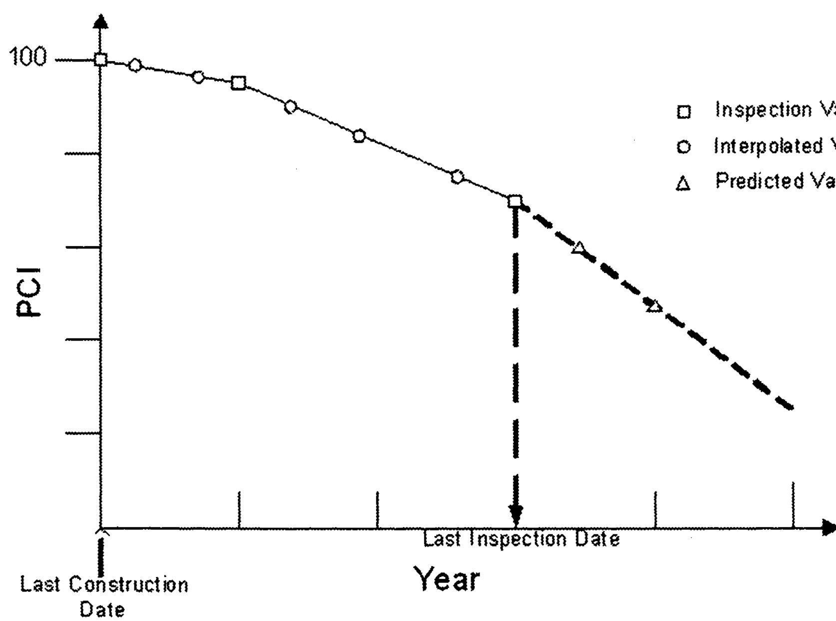


Figure 9-7. Computation of Past and Future Section PCI.

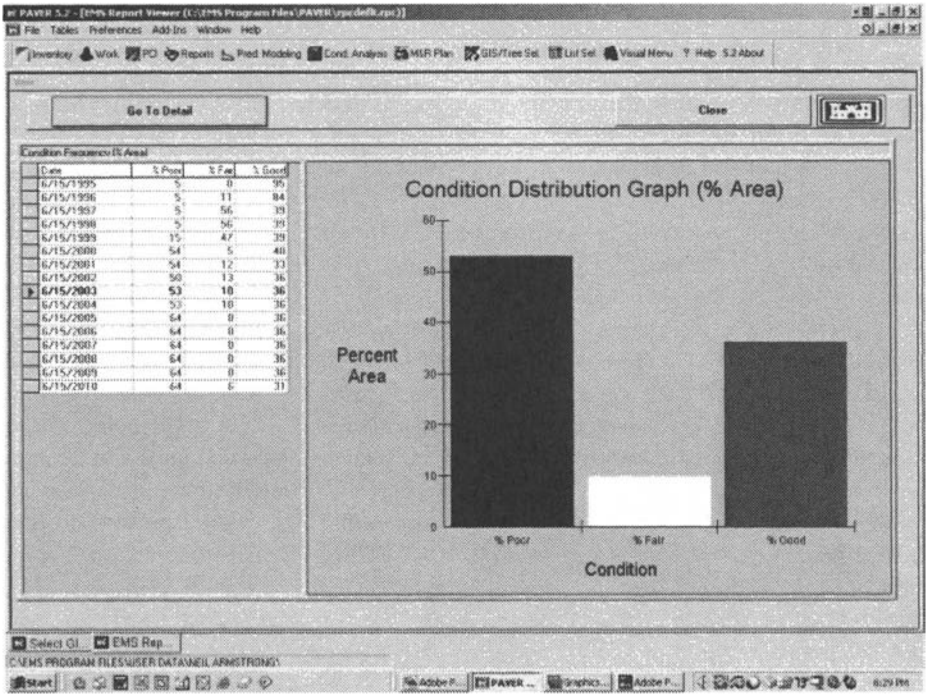


Figure 9-8a. Condition Analysis– Frequency Diagram.

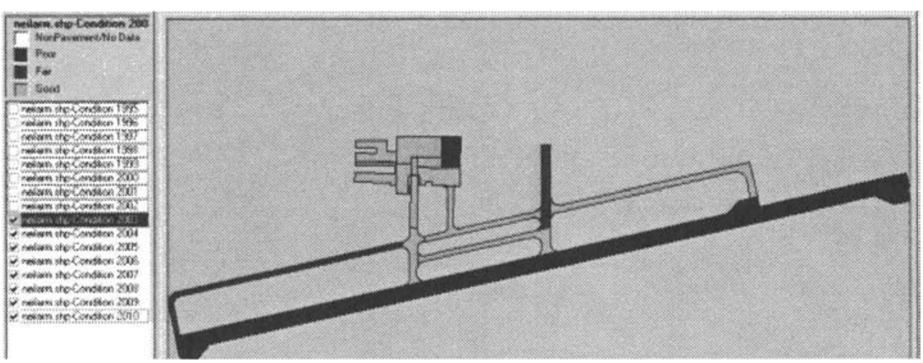


Figure 9-8b. Condition Analysis– GIS Map Display.

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10

Network-Level Pavement Management – M&R Work Planning

This chapter presents different methods for maintenance and rehabilitation (M&R) assignment to pavement sections, budget optimization, and project formulation and prioritization. Typical condition factors used in the assignment and planning process include pavement distress and other pavement condition indicators such as structural, roughness, and skid. A one-year M&R section assignment without prioritization is relatively simple as compared to multi-year assignment, budget optimization, and project formulation and prioritization.

The first part of the chapter defines M&R categories as used herein (10.1). The second part presents methods used for one-year M&R assignment to pavement sections (10.2). The third part presents multi-year "major" M&R budget forecasting based on a specified minimum Pavement Condition Index (PCI)(10.3). Two M&R multi-year assignment and planning methods that take into account budget optimization are presented in 10.4 and 10.5. The last part of this chapter (10.6) presents procedures for project formulation and prioritization where a project includes one or more sections that may or may not be contiguous and may or may not receive the same work type.

10.1 M&R Categories

M&R types are grouped into four categories: localized safety (stop-gap), localized preventive, global preventive, and major M&R. The following paragraphs briefly define each category.

<u>Localized Safety</u>

Localized safety M&R is defined as the localized distress repair needed to keep the pavement operational in a safe condition.

Localized preventive

Localized preventive M&R is defined as distress maintenance activities performed with the primary objective of slowing the rate of deterioration. These activities include crack sealing and patching.

Global preventive

Global preventive M&R is defined as activities applied to entire pavement sections with the primary objective of slowing the rate of deterioration. These activities are primarily for asphalt surfaced pavements, e.g. surface treatments.

Major M&R

Major M&R is defined as activities applied to the entire pavement section to correct or improve existing structural or functional requirements. Major M&R includes reconstruction and structural overlays. The PCI value after major M&R is assumed to be 100.

10.2 One Year M&R Section Assignment

10.2.1 Assignment of Localized Repair

Localized repair is assigned based on existing distress types and severities. Distress repair maintenance policies are therefore developed that assign specific localized M&R types, e.g. crack sealing, to specific distress type/ severity level. It is recommended that the agency develop at least two policies– one for pavements in good condition and one for pavements in bad condition. The distress maintenance policy for pavements in good condition will be referred to as "localized preventive policy" and for pavements in bad condition as "localized safety policy".

The objective of the localized preventive policy is to slow the rate of deterioration. Therefore, the policy will include recommendations for the repair of all distresses including those that may not cause user discomfort at present but may lead to a faster rate of deterioration (e.g. crack sealing of medium severity longitudinal and transverse cracks). Example localized preventive policies for roads and airfield pavements are shown in Figures 10–1 and 10–2 respectively.

The localized safety policy is a stop-gap measure until major M&R can be performed. Therefore, the policy is limited to repairing those distresses that could be a safety hazard or severely affect the intended function of the pavement. Example localized safety policies for roads and airfield pavements are shown in Figures 10–3 and 10–4 respectively.

It should be noted that applying *localized preventive* policies to pavements in bad condition is expensive and not cost-effective. The Micro PAVER program provides a tool for determining the cost and increase in PCI that will result from applying different distress maintenance policies.

Distress	Distress Severity	*Description*	Work Type	*Work Unit*
01	High	*ALLIGATOR CR*	Patching - AC Deep	*SqFt*
01	Medium	*ALLIGATOR CR*	Patching - AC Deep	*SqFt*
03	High	*BLOCK CR*	Crack Sealing - AC	*Ft*
03	Medium	*BLOCK CR*	Crack Sealing - AC	*Ft*
04	High	*BUMPS/SAGS*	Patching - AC Shallow	*SqFt*
04	Medium	*BUMPS/SAGS*	Patching - AC Shallow	*SqFt*
05	High	*CORRUGATION*	Patching - AC Shallow	*SqFt*
05	Medium	*CORRUGATION*	Patching - AC Shallow	*SqFt*
06	High	*DEPRESSION*	Patching - AC Deep	*SqFt*
06	Medium	*DEPRESSION*	Patching - AC Deep	*SqFt*
07	High	*EDGE CR*	Patching - AC Shallow	*SqFt*
07	Medium	*EDGE CR*	Crack Sealing - AC	*Ft*
08	High	*JT REF. CR*	Patching - AC Shallow	*SqFt*
08	Medium	*JT REF. CR*	Crack Sealing - AC	*Ft*
09	High	*LANE SH DROP*	Patching - AC Leveling	*SqFt*
09	Medium	*LANE SH DROP*	Patching - AC Leveling	*SqFt*
10	High	*L & T CR*	Patching - AC Shallow	*SqFt*
10	Medium	*L & T CR*	Crack Sealing - AC	*Ft*
11	High	*PATCH/UT CUT*	Patching - AC Deep	*SqFt*
13	High	*POTHOLE*	Patching - AC Deep	*SqFt*
13	Low	*POTHOLE*	Patching - AC Deep	*SqFt*
13	Medium	*POTHOLE*	Patching - AC Deep	*SqFt*
15	High	*RUTTING*	Patching - AC Deep	*SqFt*
15	Medium	*RUTTING*	Patching - AC Deep	*SqFt*
16	High	*SHOVING*	Shove Grinding	*SqFt*
16	Medium	*SHOVING*	Shove Grinding	*SqFt*
17	High	*SLIPPAGE CR*	Patching - AC Shallow	*SqFt*
17	Medium	*SLIPPAGE CR*	Patching - AC Shallow	*SqFt*
21	High	*BLOW UP*	Patching - PCC Full Depth	*SqFt*
21	Medium	*BLOW UP*	Patching - PCC Full Depth	*SqFt*
22	High	*CORNER BREAK*	Patching - PCC Full Depth	*SqFt*
23	High	*DIVIDED SLAB*	Slab Replacement - PCC	*SqFt*
23	Medium	*DIVIDED SLAB*	Slab Replacement - PCC	*SqFt*
24	High	*DURABIL. CR*	Patching - PCC Full Depth	*SqFt*
25	High	*FAULTING*	Grinding (Localized)	*Ft*
25	Medium	*FAULTING*	Grinding (Localized)	*Ft*
26	High	*JT SEAL DMG*	Joint Seal (Localized)	*Ft*
27	High	*LAND SH DROP*	Patching - AC Leveling	*SqFt*
27	Medium	*LAND SH DROP*	Patching - AC Leveling	*SqFt*
28	High	*LINEAR CR*	Slab Replacement - PCC	*SqFt*
28	Medium	*LINEAR CR*	Crack Sealing - PCC	*Ft*
29	High	*LARGE PATCH*	Patching - PCC Full Depth	*SqFt*
30	High	*SMALL PATCH*	Patching - PCC Partial Depth	*SqFt*
34	High	*PUNCHOUT*	Patching - PCC Full Depth	*SqFt*
36	High	*SCALING*	Slab Replacement - PCC	*SqFt*
38	High	*CORNER SPALL*	Patching - PCC Partial Depth	*SqFt*
39	High	*JOINT SPALL*	Patching - PCC Partial Depth	*SqFt*

Figure 10-1. Localized Preventive M&R Policy for Roads.

Distress	Distress Severity	Description	Work Type	Work Unit
41	High	ALLIGATOR CR	Patching - AC Deep	SqFt
41	Medium	ALLIGATOR CR	Patching - AC Deep	SqFt
43	High	BLOCK CR	Crack Sealing - AC	Ft
43	Medium	BLOCK CR	Crack Sealing - AC	Ft
45	High	DEPRESSION	Patching - AC Deep	SqFt
45	Medium	DEPRESSION	Patching - AC Deep	SqFt
47	High	JT REF. CR	Crack Sealing - AC	Ft
47	Medium	JT REF. CR	Crack Sealing - AC	Ft
48	High	L & T CR	Crack Sealing - AC	Ft
48	Medium	L & T CR	Crack Sealing - AC	Ft
49		OIL SPILLAGE	Patching - AC Shallow	SqFt
50	High	PATCHING	Patching - AC Deep	SqFt
50	Medium	PATCHING	Patching - AC Deep	SqFt
53	High	RUTTING	Patching - AC Deep	SqFt
53	Medium	RUTTING	Patching - AC Deep	SqFt
54	High	SHOVING	Shove Grinding	SqFt
54	Medium	SHOVING	Shove Grinding	SqFt
55		SLIPPAGE CR	Patching - AC Shallow	SqFt
56	High	SWELLING	Patching - AC Deep	SqFt
56	Medium	SWELLING	Patching - AC Deep	SqFt
61	High	BLOW-UP	Patching - PCC Full Depth	SqFt
61	Low	BLOW-UP	Patching - PCC Full Depth	SqFt
61	Medium	BLOW-UP	Patching - PCC Full Depth	SqFt
62	High	CORNER BREAK	Patching - PCC Full Depth	SqFt
62	Medium	CORNER BREAK	Patching - PCC Full Depth	SqFt
63	High	LINEAR CR	Crack Sealing - PCC	Ft
63	Medium	LINEAR CR	Crack Sealing - PCC	Ft
64	High	DURABIL. CR	Slab Replacement - PCC	SqFt
64	Medium	DURABIL. CR	Patching - PCC Full Depth	SqFt
66	High	SMALL PATCH	Patching - PCC Partial Depth	SqFt
66	Medium	SMALL PATCH	Patching - PCC Partial Depth	SqFt
67	High	LARGE PATCH	Patching - PCC Full Depth	SqFt
67	Medium	LARGE PATCH	Patching - PCC Full Depth	SqFt
70	High	SCALING	Slab Replacement - PCC	SqFt
70	Medium	SCALING	Slab Replacement - PCC	SqFt
71	High	FAULTING	Grinding (Localized)	Ft
71	Medium	FAULTING	Grinding (Localized)	Ft
72	High	SHAT. SLAB	Slab Replacement - PCC	SqFt
72	Medium	SHAT. SLAB	Slab Replacement - PCC	SqFt
74	High	JOINT SPALL	Patching - PCC Partial Depth	SqFt
74	Medium	JOINT SPALL	Patching - PCC Partial Depth	SqFt
75	High	CORNER SPALL	Patching - PCC Partial Depth	SqFt
75	Medium	CORNER SPALL	Patching - PCC Partial Depth	SqFt

Figure 10-2. Localized Preventive M&R Policy for Airfields.

Distress	Distress Severity	Description	Work Type	Work Unit
04	High	BUMPS/SAGS	Patching - AC Shallow	SqFt
05	High	CORRUGATION	Patching - AC Shallow	SqFt
09	High	LANE SH DROP	Patching - AC Leveling	SqFt
11	High	PATCH/UT CUT	Patching - AC Deep	SqFt
13	High	POTHOLE	Patching - AC Deep	SqFt
13	Medium	POTHOLE	Patching - AC Deep	SqFt
16	High	SHOVING	Shove Grinding	SqFt
21	High	BLOW UP	Slab Replacement - PCC	SqFt
21	Medium	BLOW UP	Patching - AC Shallow	SqFt
25	High	FAULTING	Slab Replacement - PCC	SqFt
27	High	LAND SH DROP	Patching - AC Leveling	SqFt
38	High	CORNER SPALL	Patching - AC Leveling	SqFt
39	High	JOINT SPALL	Patching - AC Leveling	SqFt

Figure 10-3. Localized Safety M&R for Roads.

Distress	Distress Severity	*Description*	Work Type	*Work Unit*
41	High	*ALLIGATOR CR*	Patching - AC Deep	*SqFt*
43	High	*BLOCK CR*	Crack Sealing - AC	*Ft*
45	High	*DEPRESSION*	Patching - AC Deep	*SqFt*
47	High	*JT REF. CR*	Crack Sealing - AC	*Ft*
48	High	*L & T CR*	Crack Sealing - AC	*Ft*
50	High	*PATCHING*	Patching - AC Deep	*SqFt*
53	High	*RUTTING*	Patching - AC Deep	*SqFt*
54	High	*SHOVING*	Shove Grinding	*SqFt*
55		*SLIPPAGE CR*	Patching - AC Shallow	*SqFt*
56	High	*SWELLING*	Patching - AC Deep	*SqFt*
61	High	*BLOW-UP*	Patching - PCC Full Depth	*SqFt*
61	Medium	*BLOW-UP*	Patching - PCC Full Depth	*SqFt*
62	High	*CORNER BREAK*	Patching - PCC Full Depth	*SqFt*
63	High	*LINEAR CR*	Crack Sealing - PCC	*Ft*
64	High	*DURABIL. CR*	Slab Replacement - PCC	*SqFt*
66	High	*SMALL PATCH*	Patching - PCC Partial Depth	*SqFt*
67	High	*LARGE PATCH*	Patching - PCC Full Depth	*SqFt*
70	High	*SCALING*	Slab Replacement - PCC	*SqFt*
71	High	*FAULTING*	Grinding (Localized)	*Ft*
72	High	*SHAT. SLAB*	Slab Replacement - PCC	*SqFt*
74	High	*JOINT SPALL*	Patching - PCC Partial Depth	*SqFt*
75	High	*CORNER SPALL*	Patching - PCC Partial Depth	*SqFt*

Figure 10-4. Localized Safety M&R Policy for Airfields.

10.2.1.1 Determining Consequences of Localized Repair Using Micro PAVER

Calculating the cost of repair and increase in PCI requires two steps. First, distress quantities must be converted to work quantities. Appendix G shows how Micro PAVER converts distress quantities into work (repair) quantities. Second, each distress should be adjusted based on the applied repair method in order to calculate the resulting PCI. For example, when applying crack sealing to medium severity cracks, the cracks become low severity. Similarly after patching medium or high severity alligator cracking, the resulting distress will be a low severity patch. Micro PAVER has built-in tables to show the consequences of applying each repair method to a distress, i.e. distress before and after repair.

Using the "consequence of localized repair" tool, one can examine the cost-effectiveness of a specific distress maintenance policy by applying the policy to a homogeneous group of pavement sections (e.g. asphalt roadways). Figure 10–5 shows an example of PCI vs. unit cost as a result of applying localized preventive policy to asphalt surfaced roadways in a small pavement network. The figure shows a significant increase in unit cost of repair at a PCI value of approximately 60. For this group of pavements, the economic approach would be to apply the localized preventive policy to pavement sections with a PCI above 60 and the localized safety policy to pavement sections with a PCI below 60.

10.2.2 Assignment of Global Preventive (Surface Treatments)

As implied by the word preventive, global preventive M&R should be applied to pavements in good condition. Applying global preventive (surface treatments) to asphalt pavements in bad condition is not cost-effective since these treatments correct

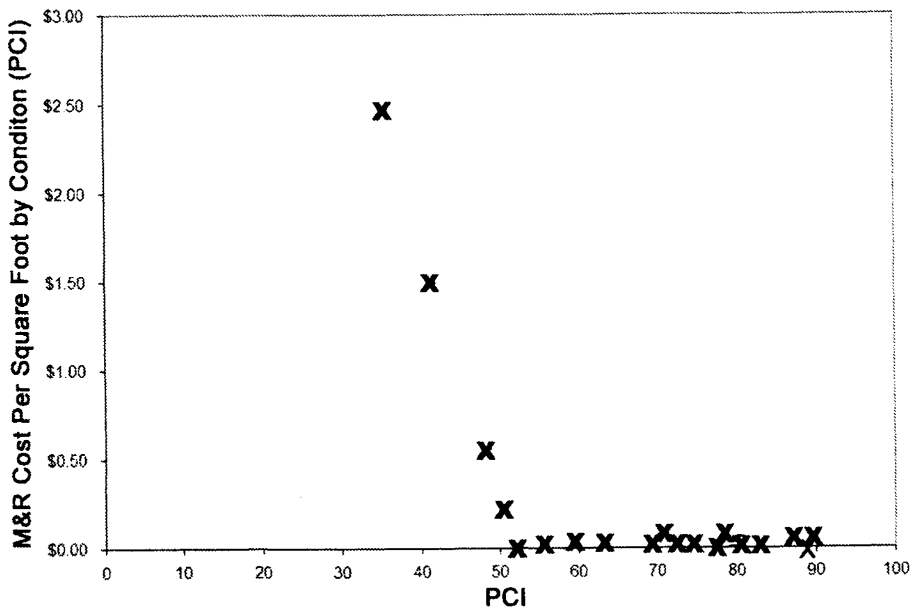

Figure 10-5. PCI vs. Unit Cost as a Result of Applying Localized Preventive M&R.

neither the structural capacity nor the roughness of the pavement. Three types of global preventive M&R are defined for asphalt surfaced pavements. The primary objective of global preventive M&R is to increase the life of the pavement by slowing its rate of deterioration.

The assignment of specific types of surface treatments to pavement sections can be optimized based on existing distress types. Figure 10–6 shows an assignment methodology that allows the application of up to three different types of surface treatments. Type 3 is assigned to pavements with skid-causing distresses such as bleeding. Type 2 is assigned to pavements with climate-related distresses such as block cracking. Type 1 is assigned to pavements with little or no distress. It should be noted that a maintenance agency may select one surface treatment type such as slurry seal regardless of existing distress types. Selection of the surface treatment type is also a function of the use and rank (functional classification) of the pavement. For example, aggregate seals may not be appropriate for runways due to the fear of foreign object damage (FOD) potential to aircraft engines. Instead, a thin overlay may be used.

10.2.3 Assignment of Different M&R Types Based on Condition—Condition Matrix Approach

The PCI by itself is not sufficient to identify the needed specific M&R type; however, it is a good indicator of the needed M&R category. Figure 10–7 shows a correlation between the PCI and collective judgment of experienced engineers recommending localized preventive and/or global preventive M&R. The study (Shahin et al 1977) was conducted on 37 airfield pavement sections, 18 of which were asphalt and 19 concrete. Ten experienced engineers were asked their opinion based on a summary of distresses,

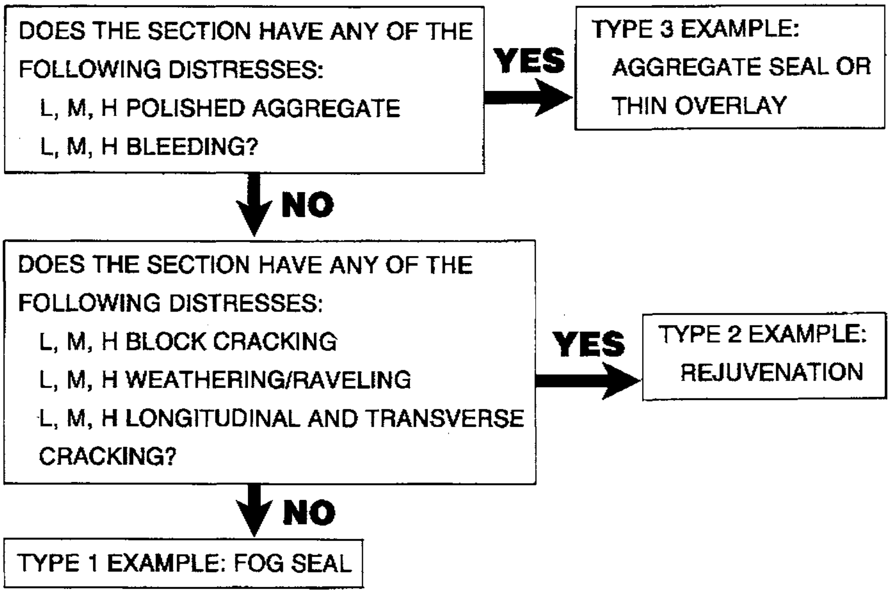

Figure 10-6. Assignment Methodology for Applying up to Three Surface Treatments.

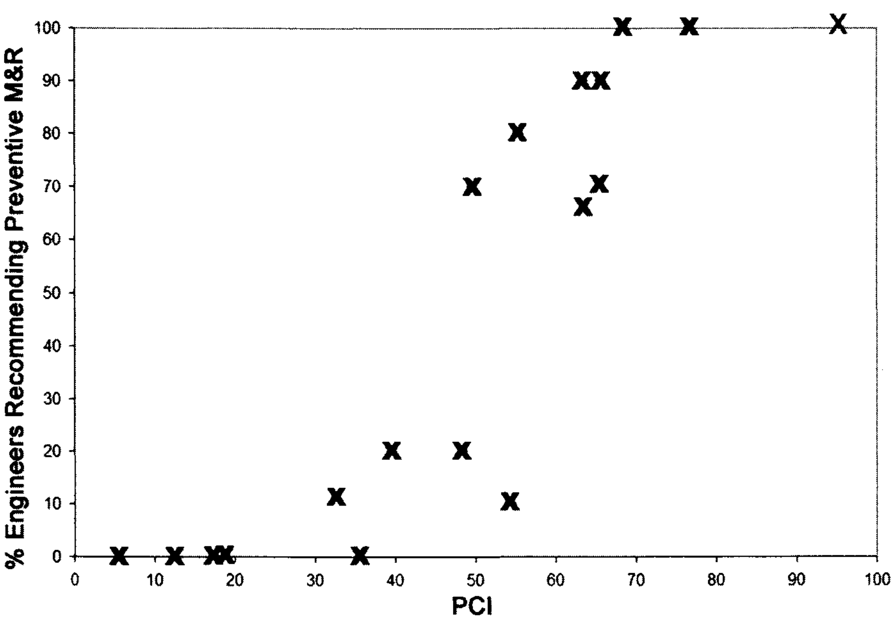

Figure 10-7. Percent of Engineers Recommending Preventive M&R.

photographs, and relevant pavement information including structure and traffic. The engineers did not know the PCI values. It can be seen from the figure that there was a nearly unanimous opinion above a PCI of 70 and below a PCI 50. Between a PCI of 50 and 70, additional information may be needed and economic analysis may be required.

Several highway and airport agencies tend to develop familiarity with specific M&R types (e.g. slurry seal, mill and overlay) and apply these types based on existing condition, type of pavement use, and pavement rank. Figure 10–8 is an example M&R assignment by condition matrix for city roads. The condition indices used are structural distress, climatic distress, and ride. The structural and climatic distress indices can be calculated using Micro PAVER by identifying which distresses to include and using the PCI engine as described in Chapter 3. The agency engineer will have to decide on the limits that define good, fair, and poor for each of the indices. The ride classification can be based on the International Roughness Index (IRI) or calculated based on distresses that affect roughness similar to the structural and climatic indices.

Figure 10–9 is an example M&R assignment by condition matrix for an airfield. The condition indices used are the PCI and the ACN/PCN. The ACN/PCN is a load carrying capacity indicator which is described in Chapter 4.

The M&R assignment based on condition is a rational method as long as there is no budget limitation. In the case of a limited budget, the method does not take into account budget optimization for the entire network.

Ride	Climatic Distress	Structural Distress: Good	Fair	Poor
Good	Good	Surface Seal	Base Failure + Resurface	Base Failure + Resurface
	Fair	Crack Seal + Surface Seal	Base Failure + Resurface	Base Failure + Resurface
	Poor	Crack Seal + Surface Seal	Reconstruction	Reconstruction
Fair	Good	Thin Overlay	Profile (Coldmill) + Base Failure + Resurface	Profile (Coldmill) + Base Failure + Resurface
	Fair	Profile + Thin Overlay	Profile (Coldmill) + Base Failure + Resurface	Profile (Coldmill) + Base Failure + Resurface
	Poor	Profile + Thin Overlay	Reconstruction	Reconstruction
Poor	Good	Profile + Thin Overlay	Profile (Coldmill) + Base Failure + Resurface	Reconstruction
	Fair	Profile + Thin Overlay	Reconstruction	Reconstruction
	Poor	Reconstruction	Reconstruction	Reconstruction

Figure 10-8. Example M&R Assignment by Condition Matrix for City Roads.

ACN/PCN	PCI: Good	PCI: Fair	PCI: Poor
Good	Localized	2.5" Overlay	Coldmill & Overlay
Fair	Localized	Coldmill & Overlay	Coldmill & Overlay
Poor	Structural Overlay	Coldmill & Structural Overlay	Reconstruction

Figure 10-9. Example M&R Assignment by Condition Matrix for Airfield Pavements.

10.3 Multi-Year Major M&R Planning based on Minimum PCI

The minimum condition approach is basically a worst first approach which does not optimize budget spending and thus return on investment. The Micro PAVER major M&R planning based on minimum PCI provides budget requirements needed to maintain the pavement condition above the specified level. The specified level can be varied by pavement use and rank. The specified minimum values can also be changed by year. If a high minimum PCI value is specified in the first year, the required major M&R budget is likely to be high and unaffordable. Instead, the minimum PCI can be gradually increased over several years until the desirable value is reached, thus avoiding the need for a high budget in the first year. The PCI condition projection is performed using the family concept presented in Chapter 7.

The cost for applying the major M&R is estimated for each section by projecting the year in which the section will deteriorate to the specified minimum condition and multiplying the section area by the unit M&R cost. The user must specify the inflation rate as well as the relationship between PCI and unit cost of major M&R. The PCI vs. major M&R unit cost should be established for each pavement use/rank/surface type combination. For example, the cost is likely to be higher for primary (arterial) concrete roadways compared to secondary asphalt parking lots. Figure 10–10 can be used as a guide in developing the PCI vs. cost relationship for major M&R. At a PCI value of 75 or higher, assume that a 2–inch overlay can be performed. At a PCI of 30 or below, assume that reconstruction will be required. A straight line can be assumed between these two boundaries even though the cost is likely to be curved as shown in the figure.

Figure 10–11 shows an example major M&R budget requirement for a network of roads where the minimum condition was specified as PCI = 55 for the 10 years. The figure also shows the result of gradually increasing the minimum PCI to 55 over the first 5 years and then maintaining it at 55 for the remaining 5 years. The advantage of that approach is to reduce budget requirements in the first year; however, it does result in a slightly increased total cost over the planning period.

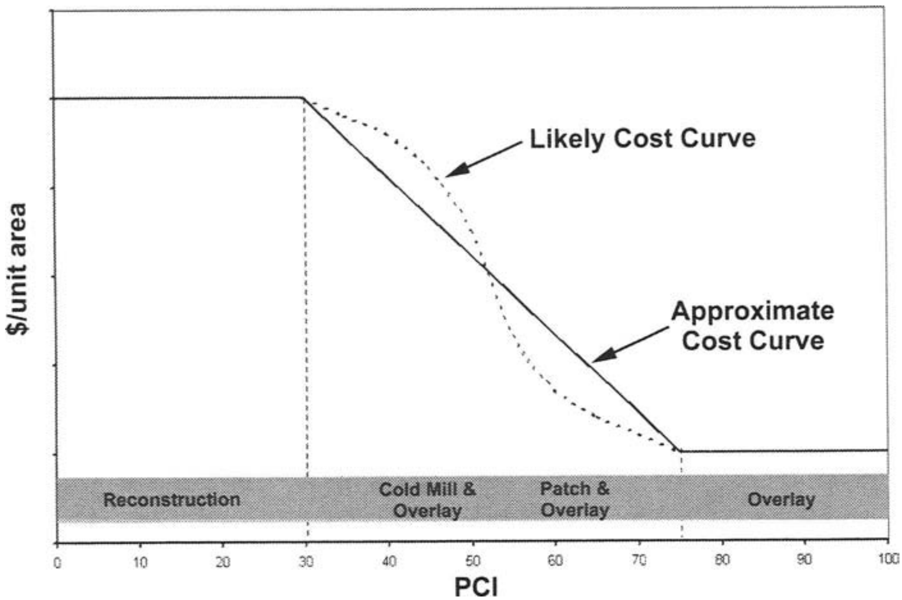

Figure 10-10. Guideline for Development of PCI vs. Unit Cost for Major M&R.

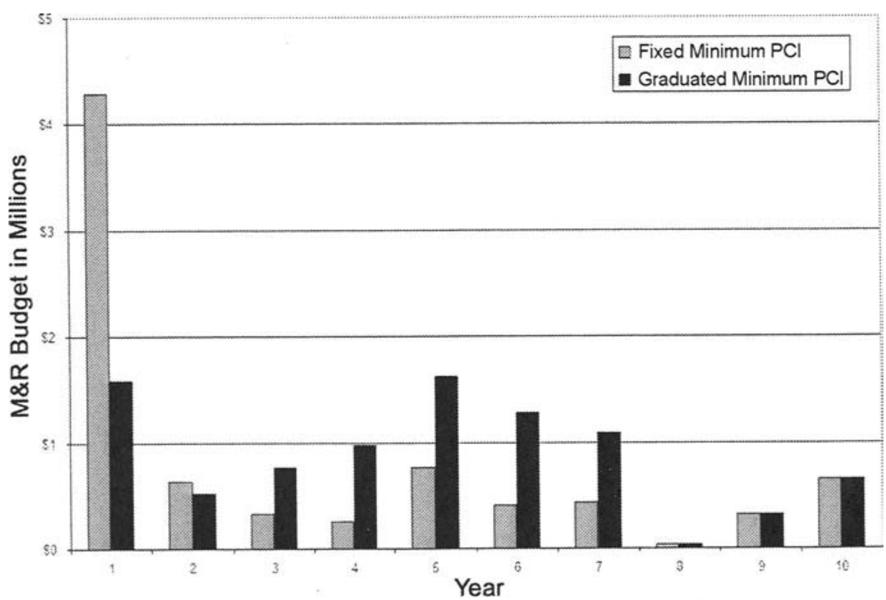

Figure 10-11. Example Major M&R Budget Requirements for Two Minimum PCI Scenarios.

10.4 Multi-Year M&R Section Assignment (Work Planning) – Critical PCI Method

The critical PCI procedure (Shahin and Walther 1990) is based on the concept that it is more economical to maintain pavements above rather than below the critical PCI. It was developed by studying results from the dynamic programming network optimization analysis (presented in 10.5), and by performing many life-cycle cost analyses on many projects. The procedure is presented in the following paragraphs by defining the critical PCI (10.4.1), describing how the effect of applying each M&R category is considered in work planning (10.4.2), describing the process by which the M&R categories are assigned to each pavement section (10.4.3), work prioritization (10.4.4), and how the procedure is used in determining budget consequence (10.4.5) and budget requirements (10.4.6).

10.4.1 Critical PCI Definition

A critical PCI is defined as the PCI value at which the rate of PCI loss increases with time or the cost of applying localized preventive maintenance increases significantly. Figure 10–12 depicts an example deterioration curve of the usual range of the critical PCI, which is 55 to 70.

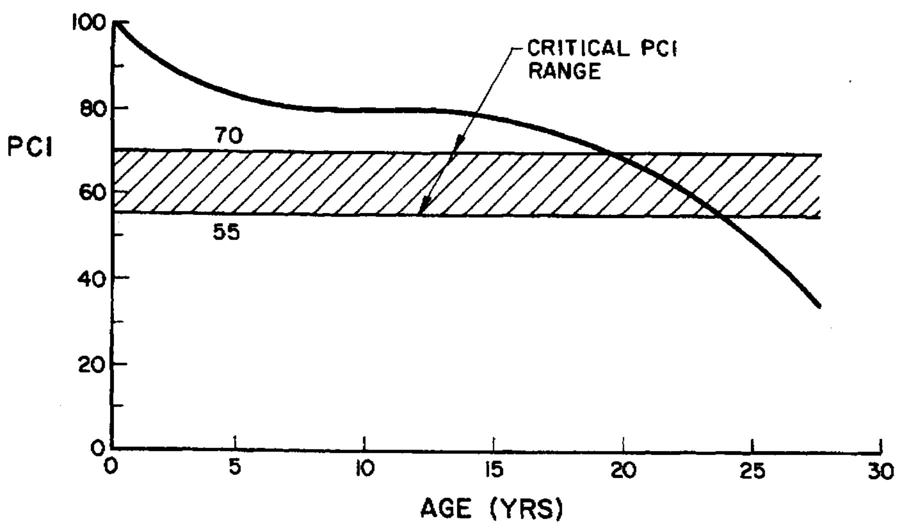

Figure 10-12 Deterioration Curve Showing Typical Critical PCI Range.

The following procedure is recommended for establishing the value of the critical PCI:

1. Develop a family curve for the pavement under consideration and visually select the critical PCI based on the PCI rate of deterioration.
2. Select the localized preventive distress maintenance policy to be used in developing the work plans.
3. Apply the selected preventive policy to the pavement sections and plot the cost of localized preventive maintenance per unit area for each of the sections as was shown in Figure 10–5.
4. Select the critical PCI based on results from steps1 and 3 supplemented with engineering judgment.

10.4.2 Considering the Effect of Applying M&R in Work Planning

In multi-year work planning, a pavement section may receive different or repeated M&R based on its condition, rate of deterioration, and length of the work plan period. Applying major M&R (see definitions in 10.1) will increase the PCI of the pavement section to 100. Applying global preventive or localized preventive M&R is likely to increase the life of the pavement section. Applying localized safety M&R is not likely to increase the life of the pavement section.

10.4.2.1 Considering the Effect of Global Preventive M&R in Work Planning

Two approaches credit the application of global preventive M&R, either increasing the PCI at the time of application and calculating the increase in life or specifying the increase in life and calculating the increase in the PCI. The first approach is not recommended since increasing the PCI by few points can lead to an increase in life from one year to over 10 years based on the family deterioration curve the section is assigned to. The second approach is preferred, that is to specify the increase in pavement life (ΔT) for each type of global preventive M&R and calculate the effective increase in PCI (ΔPCI), as shown in Figure 10–13 . The calculated increase in PCI will depend on the family deterioration curve assigned to the section. For example, if the PCI of the section at the time of application was 75 and the increase in life is 4 years, then the increase in PCI will be calculated so that the PCI will return to 75 in 4 years.

10.4.2.2 Considering the Effect of Localized Preventive M&R in Work Planning

The credit from applying localized preventive maintenance can be treated in an approach similar to the global preventive M&R method described above. The recommended approach is shown in Figure 10–14. The application of localized preventive M&R is not likely to start until several years after the last construction or major M&R date. That is normally when crack filling and patching may be required. To credit the performance of the pavement section, one has to specify the expected total increase in life (ΔT). The specified increase can be assigned based on the maintenance agency distress maintenance policy and the type, use, and rank of the pavement section. As shown in the figure, the annual increase in life (Δt) is calculated by dividing ΔT by the

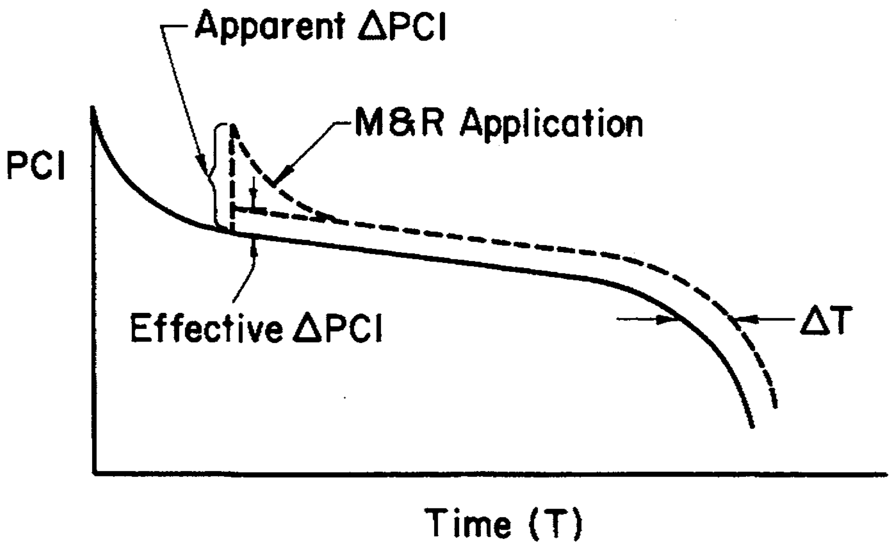

Figure 10-13. Determination of Increase in PCI (ΔPCI) Due to Increase in Life (ΔT).

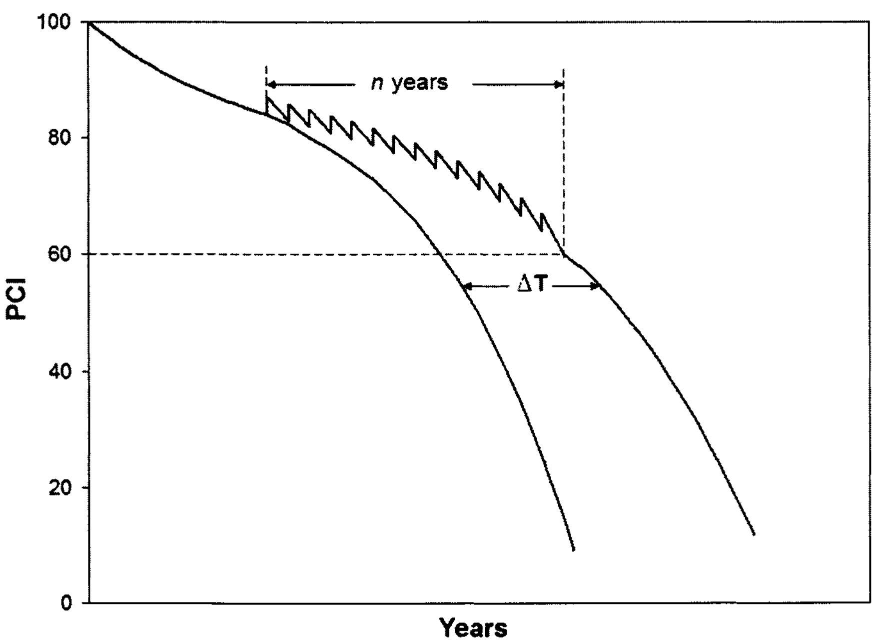

Figure 10-14. Effect of Localized Preventive M&R on PCI.

number of years (n) during which the localized preventive M&R is applied. There is no annual increase during the early years when no localized preventive M&R is applied.

Another important consideration when applying the credit is whether the condition deterioration family curve has built into it the localized preventive M&R policy. If the policy is already built in, then no increase in life should be credited. Also, in this case, if the localized preventive M&R is not included in the work plan, then a negative credit should be applied.

10.4.3 M&R Assignment to Pavement Sections

Assigning an M&R category is a function of whether the section PCI is above or below the critical PCI as shown in Figure 10–15. If the section PCI is above critical, localized preventive and/or global preventive M&R are applied. Major M&R is applied only if the pavement section is structurally deficient. If the section PCI is below critical, localized safety or major M&R is applied. No preventive M&R is applied. A more detailed description of the assignment process is presented in the following paragraphs with examples of how it is implemented in Micro PAVER.

10.4.3.1 M&R Assignment for Sections Above or Equal to The Critical PCI

The first step, Figure 10–16 is to check for the structural deficiency of the pavement section. If the pavement is structurally deficient, then major M&R should be applied. The existing pavement distress can be used to indicate structural deficiency. Figure 10–17 provides a list of distresses used in the Micro PAVER Work Plan to identify pavement sections above the critical PCI that are structurally deficient. The cost of major M&R is determined based on the PCI vs. unit cost relationship described in paragraph 10.3 and

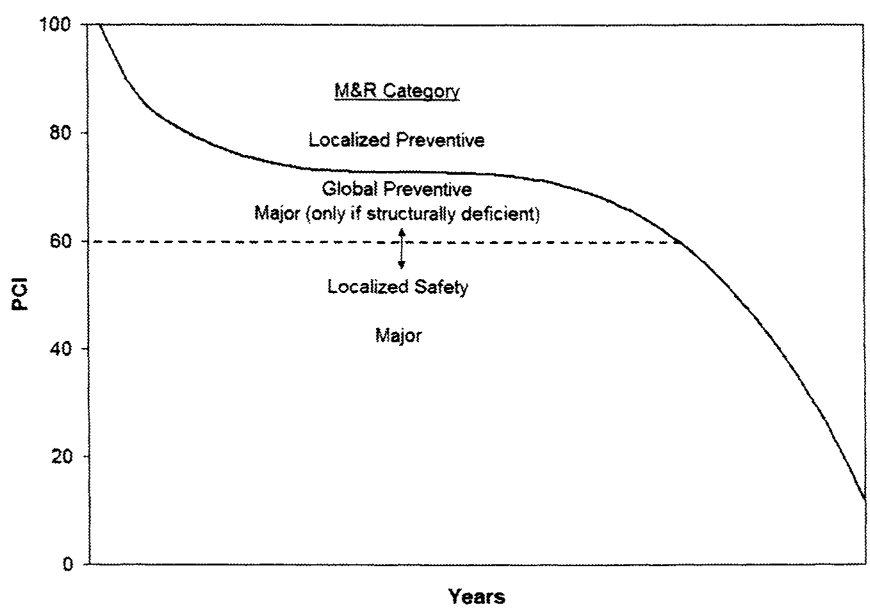

Figure 10-15. Assigning M&R Category Using Critical PCI Method.

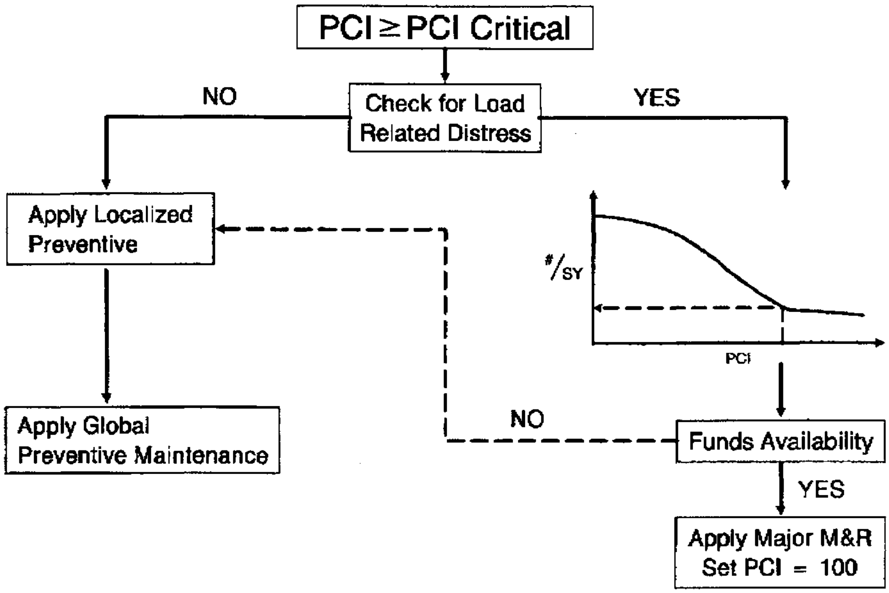

Figure 10-16. M&R Assignment for Sections above or equal to The Critical PCI.

Asphalt Pavement		
Alligator Cracking	L+M+H	>0.5%
Patching	M+H	>10%
Potholes	L+M+H	>0.1%
Rutting	M+H	>1.0%
Concrete Pavement		
Large Patching	M+H	>10%
Corner Break	L+M+H	>5%
+ Divided (Shattered Slab)	L+M+H	
+ Punchout	M+H	

Figure 10-17. List of Structural Distresses Used in Work Plan.

shown in Figure 10-10. The unit cost at the section's PCI is multiplied by the section area to determine the cost. If the section is structurally deficient, the next step is to check on funds availability based on available budget and major M&R priorities (as discussed later in this chapter). If funds are available, major M&R is applied and the PCI value is set to 100. If funds are not available localized preventive and/or global preventive are applied as described below for the current year, and funds availability is checked in the following years.

Pavement sections that are not structurally deficient receive localized preventive M&R. The cost of application is determined in the first year, either through using the PCI vs. cost relationship (see Fig. 10–5) or based on the results of the most recent distress inspection. For the second year and beyond, only the PCI vs. cost method can be used since the distress information is not available. If the PCI vs. cost approach is used, the unit cost at the section's PCI is multiplied by the section area to determine the cost. If the first year cost is determined based on distress, the specified distress maintenance policy (see Figures 10–1 and 10–2) is applied to the work quantities (see Appendix G) to determine the cost.

The global preventive M&R is applied based on the specified interval between applications. The process of selecting the specific type of global M&R for asphalt pavements was described in paragraph 10.2.2 and shown in Figure 10–6 .

10.4.3.2 M&R Assignment for Sections below The Critical PCI

The first step (Figure 10–18) is to check on funding availability based on budget and major M&R priorities (as discussed later in this chapter). If funds are available, major M&R is applied and the PCI value is set to 100. If funds are not available, localized safety M&R is applied and fund availability is checked in the following years.

The cost of applying localized safety M&R is determined in the first year either through using the PCI vs. cost relationship (see Figure 10–5) or based on the results of the most recent distress inspection. For the second year and beyond, only the PCI vs. cost method can be used since the distress information is not available. If the PCI vs. cost approach is used, the unit cost at the section's PCI is multiplied by the section area to determine the cost. If the first year cost is determined based on distress, the specified distress maintenance policy (see Figures 10–3 and 10–4) is applied to the work quantities (see Appendix G) to determine the cost.

10.4.4 M&R Budget Prioritization/Optimization

The primary objective of the Critical PCI method is to keep all pavements above the critical PCI value, thus minimizing M&R spending. By keeping the pavement above the critical PCI the primary emphasis is placed on preventive M&R, i.e. localized preventive and global preventive. When pavements reach the critical PCI, they should receive major M&R as soon as funds are available, which will bring the PCI value back to 100.

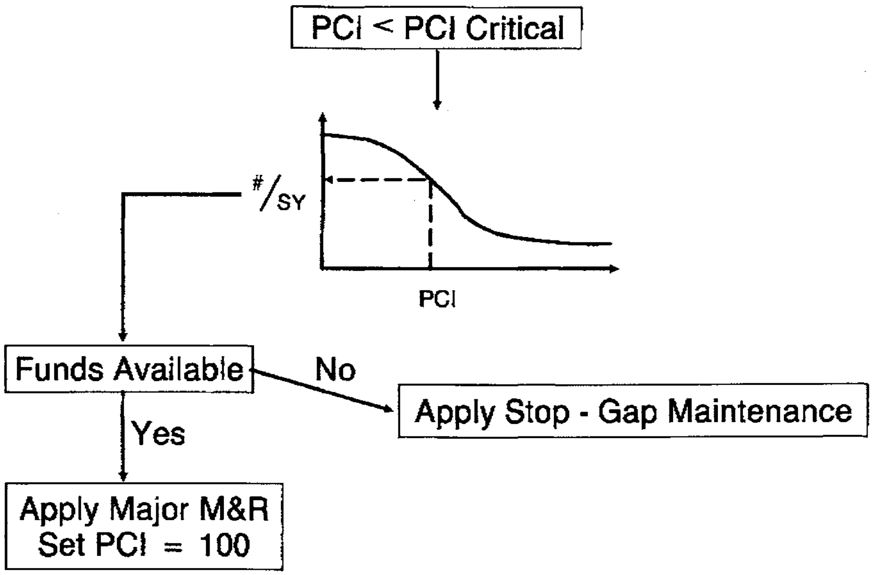

Figure 10-18. M&R Assignment for Sections Below The Critical PCI.

10.4.4.1 Unlimited vs. Limited Budget

In a scenario where the available M&R budget is "unlimited", there is no need for prioritization. In this case M&R spending is optimized by using the M&R pavement section assignment described in 10.4.3 above. When the M&R budget is limited (i.e., less than that needed to perform all optimum M&R), then optimization and prioritization are necessary to achieve maximum return on investment. It is to be expected that certain projects must be performed regardless of budget optimization. This can be due to functional requirements (e.g., user cost), mission objectives (e.g., increase in traffic loading), and in some cases, political/social realities.

10.4.4.2 Prioritization Scheme

The prioritization scheme presented here emphasizes budget optimization. This is achieved by giving the highest priority to preventive M&R. The exception is localized safety, which should be performed only when the PCI is below critical and there are no funds available to perform major M&R.

The first factor considered in budget prioritization is the M&R category. The following lists the order in which M&R categories are prioritized:

1. Localized safety
2. Localized preventive
3. Global preventive
4. Major above critical PCI
5. Major below critical PCI

The reason major M&R above critical is higher priority than major M&R below critical is to minimize the cost before the pavement's rate of deterioration increases. The unit cost of major above critical is much less than that below critical (see Figure 10-10). For example, one may perform a 2-inch to 3-inch overlay above critical PCI as compared to mill and overlay or reconstruct below critical PCI. Major above critical PCI is only performed when there is structural deficiency or heavier traffic is expected.

Within each M&R category, a priority factor is assigned based on the combination of pavement use and rank (i.e. functional classification). Figure 10–19 shows an example priority table where each pavement use and rank is classified into three levels; low, medium, and high. Example use levels for airfields would be runways (high), taxiways (medium), and aprons (low). Example rank levels would be primary (high), secondary (medium), and tertiary (low).

It is to be expected, however, that within each M&R category and priority factor (based on use/ rank), there is likely to be more than one pavement section requiring M&R. In that case, the PCI value is used to break the tie. In general, pavement sections with PCI closer to the critical PCI get higher priority with the exception of sections needing localized safety M&R where the sections with lower PCI get higher priority. Figure 10–20 shows a summary of the overall prioritization/optimization procedure used in the critical PCI method.

RANK / USE	High	Medium	Low
High	1	3	6
Medium	2	5	8
Low	4	7	9

Figure 10-19. Priority Based on Use/Rank.

10.4.5 Determining Budget Consequence

The consequence of different budget scenarios is determined in terms of resulting condition and backlog of major M&R (deferred M&R). The Micro PAVER program provides the capability to perform such analyses using the critical PCI method. The results are provided at the network, branch, and section levels. Figure 10–21 shows an example budget consequence analysis for a small road and parking network. Two budgets are analyzed: $0.5 and $ 1.5 million/year. The figure shows the resulting unfunded major M&R for each budget. Figure 10–22 shows the resulting network condition. For each budget, the program also provides a summary by section output that shows each section's recommended M&R category for each year of the plan as well as the PCI before and after the application.

10.4.6 Determining Budget Requirements

Budget requirements are determined for different management objectives. Common management objectives are:

1. Eliminate backlog of major M&R in a specified period of time.
2. Maintain current area-weighted PCI over a specified period of time.
3. Reach desired area-weighted PCI in a specified period of time.

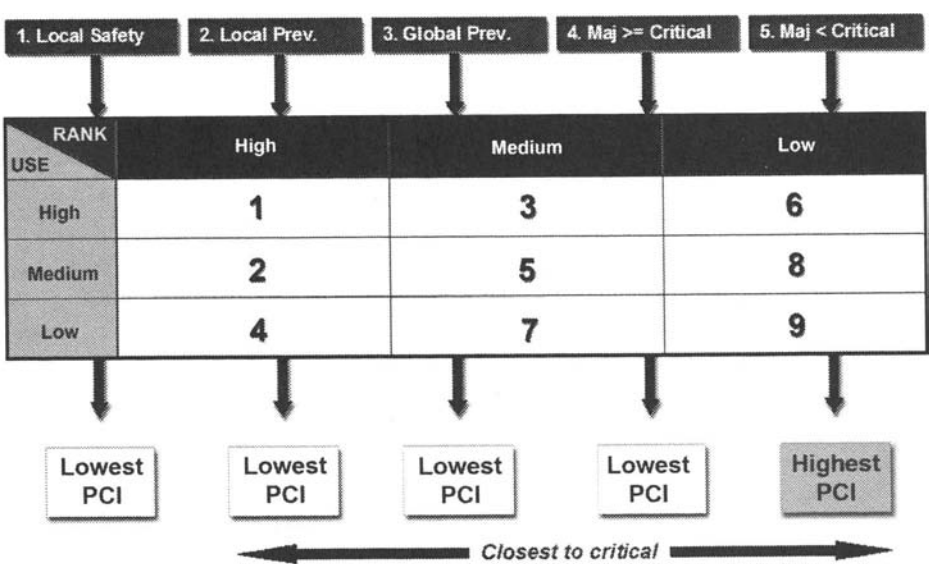

Figure 10-20. Summary of the Prioritization/Optimization Procedure Used in the Critical PCI Method.

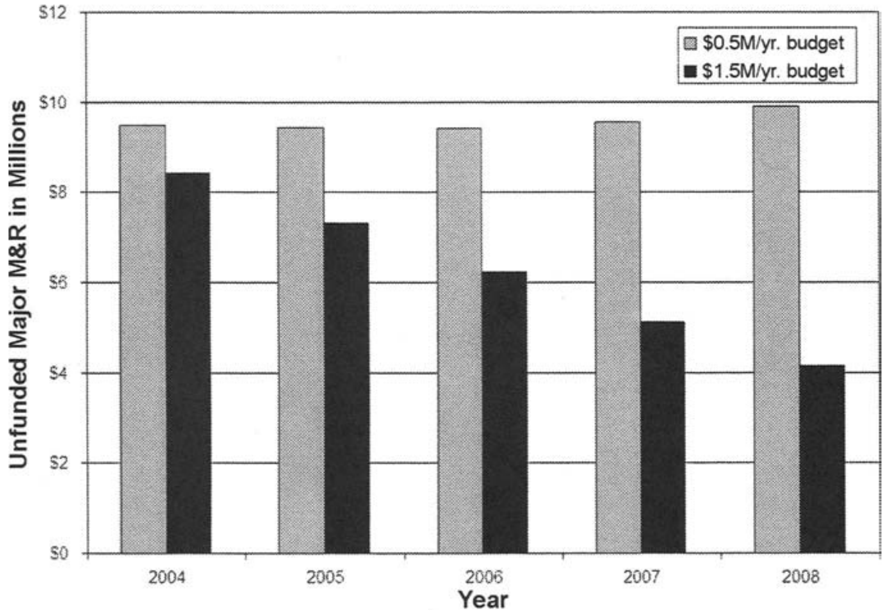

Figure 10-21. Budget Consequence Analysis for a Small Road Network .

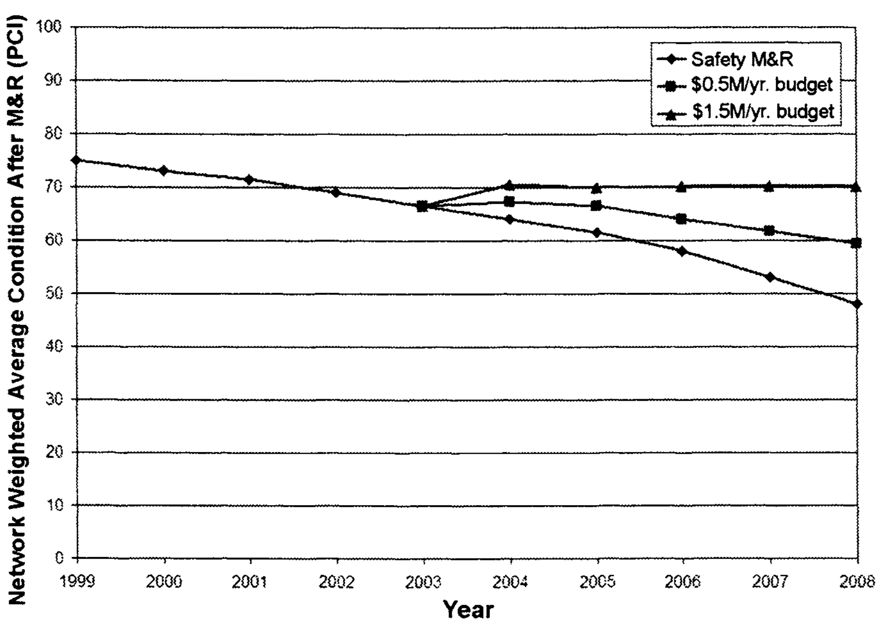

Figure 10-22. Annual Condition Plot of the Area Weighted Average Condition After M&R .

Micro PAVER calculates budget requirements for any of the above objectives by performing budget consequences with a built-in iterative procedure as follows:

Step 1: Run a budget consequence scenario plan with unlimited budget and set the following:

–The maximum budget equal to the highest annual budget during the analysis period which is usually the first year budget.

–The minimum budget equal to zero.

Step 2: If an unlimited budget cannot achieve the goal, stop the analysis. This normally happens when the desired PCI at the end of the analysis period is higher than what is possible. For example if the critical PCI is 60, so that major M&R is performed on every section that reaches a PCI of 60, the expected average PCI of the network at the end of the analysis period will be 80. Specifying a network average PCI greater than 80 will be difficult to achieve. If the goal can be achieved, then continue to Step 3.

Step 3: Use a budget equal to:

$$\text{Current Budget} = \left(\frac{\text{max. budget} + \text{min. budget}}{2}\right)$$

–If goal achieved, set max. = Current Budget

–If goal not achieved, set min. = Current Budget

Step 4: Repeat Step 3 until the end condition tolerance or allowed number of iterations is achieved.

The objectives are considered to be achieved as follows:

- Backlog Elimination: No unfunded major M&R in the last year of the analysis period, Figure 10–23.
- Maintain Condition: Compare the area weighted PCI at the beginning of the analysis period (before M&R) with the value at the end of the analysis period plus one year where no work is performed in the last year, Figure 10–24.
- Reach a Specified Condition Value: Compare the specified PCI value with the value at the end of the analysis period plus one year where no work is performed in the last year.

10.4.7 Meeting GASB 34 Requirements

In 1999, the Governmental Accounting Standards Board (GASB) issued Statement 34, "Basic Financial Statements and Management's Discussion and Analysis for State and Local Governments". The statement calls for state and local governments to capitalize their long-lived infrastructure assets that were built or received major additions after June 15, 1980 (FHWA 2000). Capitalization means that the amount expended to acquire

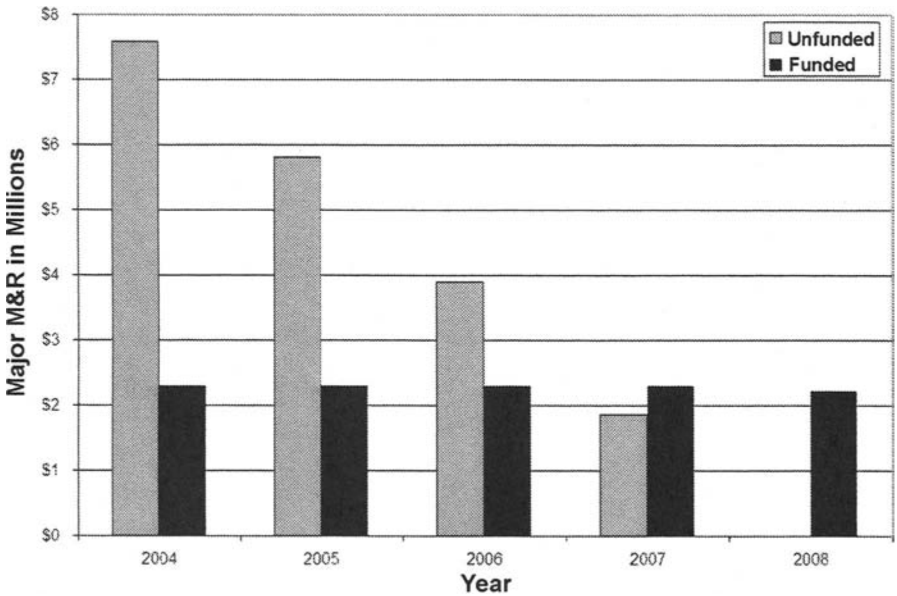

Figure 10-23. Example of Eliminating Backlog by Major M&R in 5 years.

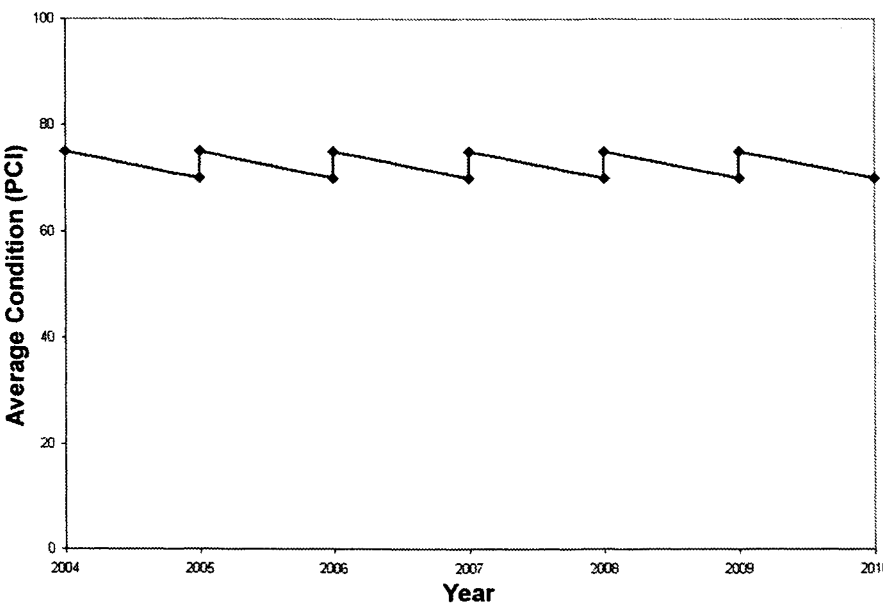

Figure 10-24. Maintain Current Condition .

a capital resource should be considered an asset rather than an expense. The amount expended should be calculated at historical cost or using deflated current replacement cost. The Statement also required that the cost of using the asset must be reflected. GASB allowed governments two methods for reflecting the cost of using the asset: depreciation (traditional approach) or preservation (modified approach).

10.4.7.1 Traditional Approach

In this approach, the annual cost of using the capital asset consists of two components: (1) operating M&R and (2) depreciation. The operating M&R annual cost is calculated based on the M&R necessary to insure that the asset will reach its useful life prior to major M&R. This cost can be determined using the Work Plan feature of the Micro PAVER management system. The annual depreciation component of the cost can be calculated as follows:

Annual Depreciation Cost = (Initial Cost – Salvage Value)/Useful Life

If the Initial Cost is not known, it can be estimated by deflating the current reconstruction cost. The Salvage Value is the portion of the Initial cost that will remain at the end of the asset Useful Life.

10.4.7.2 Modified Approach

Governments using the modified approach do not have to depreciate their assets. The government, however, must meet the following requirements for the modified approach to be acceptable:

1. Make public a condition goal for the asset.
2. Estimate the spending level necessary to meet the published condition goal.
3. Compare actual spending with the estimated level above.
4. Document that the asset is maintained at or better than the published condition goal.

To meet the above requirements the government must have in place a managing system that is capable of: (1) accounting for the inventory, determining the inventory condition using a repeatable condition index, (2) determining the needed spending level to achieve the published condition goal, and (3) documenting that the assets are being preserved at or above the condition goal. All these functions can be performed using the Micro PAVER system. Specifically, Micro PAVER does have an automated function to determine the required spending to reach a specified condition level as described in 10.4.6.

If the published condition level is not met, the government will no longer be eligible to report using the modified approach and may either depreciate the assets or lower the published condition level.

10.4.8 Determining Penalty Cost of Delaying Major M&R

The management objectives can only be achieved with the calculated budget when work is performed on timely basis. There is likely to be increase in cost if the scheduled Major M&R for a pavement section is delayed by several years. The amount of increase is a function of the pavement section's PCI at the time major M&R was scheduled and the projected rate of deterioration. The following equation is used to calculate the penalty cost for Major M&R delay:

$$\text{Penalty \%} = \left(\frac{C_f - C_s}{C_s}\right) \text{x}\,100$$

Where:

C_S = Cost in originally scheduled year
C_F = Future Cost

= [Major M&R Cost for projected PCI *((1+ i)**n)] + Localized Safety M&R cost over the delay period

i = Inflation rate

n = Time delay, in years

The reason for normalizing the penalty cost by dividing it by the original cost (C_S) is to determine the relative penalty regardless of the pavement section size This allows for its use as a priority tool. Following is an example penalty cost calculation:

Year 2005 Major M&R (PCI = 70) = $40,650
Year 2008 Major M&R (PCI = 62) = $53,741
n = 3, i = 3%
Future Major M&R cost = (53,741) * (1 + 0.03)**3 = $58,724
Localized Safety cost over the 3-year delay = $220
C_F = 58,724 220 = $58, 944

$$\text{Penalty Cost \%} = \left(\frac{58{,}944 - 40{,}650}{40{,}650}\right) \text{x}\,100 = 45\%$$

10.4.9 Project Formulation

The results of the budget analysis include the recommended M&R category for each pavement section for each year. Because of the economy of scale, it is unlikely that a project will be generated for each pavement section. Instead sections will be grouped to formulate projects that will reduce unit cost and minimize interruption to traffic. The use of a geographic information system (GIS) is very helpful in formulating projects. Figure

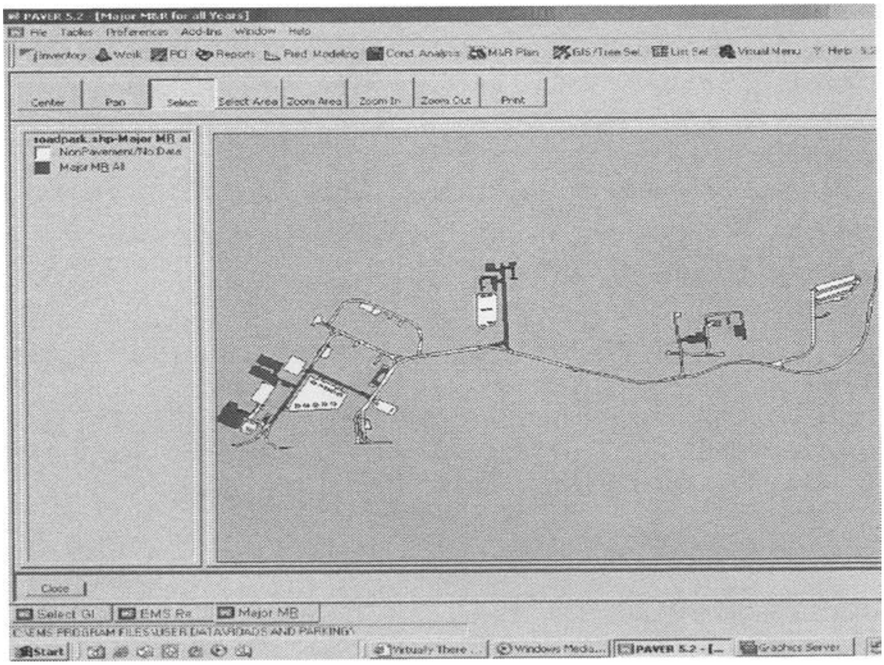

Figure 10-25. Example GIS Display of Recommended Major M&R for Next Five Years.

10–25 shows an example GIS display of predicted section major M&R requirements for 5 years. This information is used in combining sections based on traffic flow and work similarity. Also when formulating projects, work will be specified in terms of M&R type (e.g. 3.0 inch overlay) rather than M&R category (e.g. major M&R). Each project is defined as follows:

1. Select a project name.
2. Select pavement sections to be included in the project.
3. Select M&R types to be performed and assign work date and cost per unit area for each.
4. Add/delete work items for individual sections if different from the rest of the sections.

Projects can be defined after reviewing the results of the budget analysis, regardless of the budget analysis results. The formulation process is the same for either. Once projects have been formulated, the budget consequence analysis can be re-executed and the resulting annual budget variations analyzed in terms of annual deficit and surplus. When re-executing the budget consequence analysis, the previous M&R

category assignment to a pavement section should be adjusted based on the timing of the project in which the section was included. User-specified rules for the minimum number of years between the M&R categories is used in performing the adjustment. The following is a list of such rules:

1. Minimum number of years between major M&R applications.
2. Minimum number of years between global preventive M&R applications.
3. Minimum number of years for major M&R following global preventive.
4. Minimum number of years for global preventive following major M&R.

Example: a pavement section was to receive global preventive M&R in 2005, based on the critical PCI method, but it was included in a project for major M&R in 2007. If the minimum number of years for major M&R following global preventive is 5, then the section will not receive global preventive in 2005.

A major M&R delay penalty is also calculated for each project. It is calculated as the cost weighted average of the penalty for each of the sections included in the project:

$$\text{Major M\&R Delay Penalty \%} = \frac{\sum_{i=1}^{n} C_i P_i}{\sum_{i=1}^{n} C_i}$$

Where:

C_i = area of section i scheduled to receive major M&R as part of the project.
P_i = penalty cost in % for major M&R delay for section i.
n = number of sections in the project receiving major M&R.

A total project penalty is obtained by dividing the total increase in major M&R for all sections by the total project cost.

Figure 10–26 shows an example project list in descending order of project delay penalty cost %. The project priorities can be adjusted by the managing agency based on additional factors such as mission needs.

Title	# of Sections	Total Area	Total Cost	Area-weighted PCI Before	Major M&R Delay Penalty	Project Delay Penalty
Dahlgren Overlay	3	50,671	$50,671	60	25	25
Kuester Overlay	1	16,274	$9,764	50	12	10
Parking, Surface	3	89,577	$265,794	33	5	5

Figure 10-26. Example Project List in Descending Order of Penalty Cost.

10.5 Multi-Year M&R Section Assignment—Dynamic Programming Procedure

The dynamic programming procedure (Feighan 1988) is based on the principal that "every optimal policy consists only of optimal subpolicies." Instead of examining all possible combinations, dynamic programming examines a small, carefully chosen subset of combinations, while rejecting those combinations that cannot possibly lead to an optimal solution. The subset examined is guaranteed to contain the optimal solution. So dynamic programming is mathematical modeling that divides a large problem into a number of smaller problems that are easier to solve. An advantage of dynamic programming, for pavement network optimization, is that once the problem has been solved for the longest analysis period, the results are readily available for shorter analysis periods.

To best understand the dynamic programming procedure, it is recommended that the reader become familiar with the Markovian Prediction discussed in Chapter 7.

10.5.1 Structure of Dynamic Programming

The basic components of dynamic programming are states, stages, decision variables, transition functions, and returns. These components will be explained in terms of the PCI; however, the problem can be structured in terms of any other condition indicator.

A condition state is defined in terms of a PCI bracket. For example, each 10 PCI points can be defined as a state. Therefore, the PCI scale is divided into 10 condition states. The pavement condition is assumed to progress (deteriorate) through a series of consecutive stages. Each year in the life of the pavement is considered a stage (Fig. 10–27).

At each stage, for every possible state, the dynamic programming model regarding M&R alternatives makes a set of decisions. These decisions include what M&R alternative to implement in each state at every stage for every pavement family. Once an M&R alternative has been applied to a pavement in a given state at a given stage, a Markovian transition function is used to determine to which state and family the pavement section moves (Butt 1991).

In summary, the dynamic programming parameters are:

States: Each bracket of 10 PCI points between 0 and 100 in each family.
Stages: Each year in the analysis period.
Decision Variables: At each stage, for every state in every family, a decision is made as to which M&R alternative to apply.
Transformation: The transformation from one stage to the next is defined by the Markov Transition Probability Matrix (represents pavement deterioration).
Return: Expected cost if a particular decision is made in each state of each family at each stage.

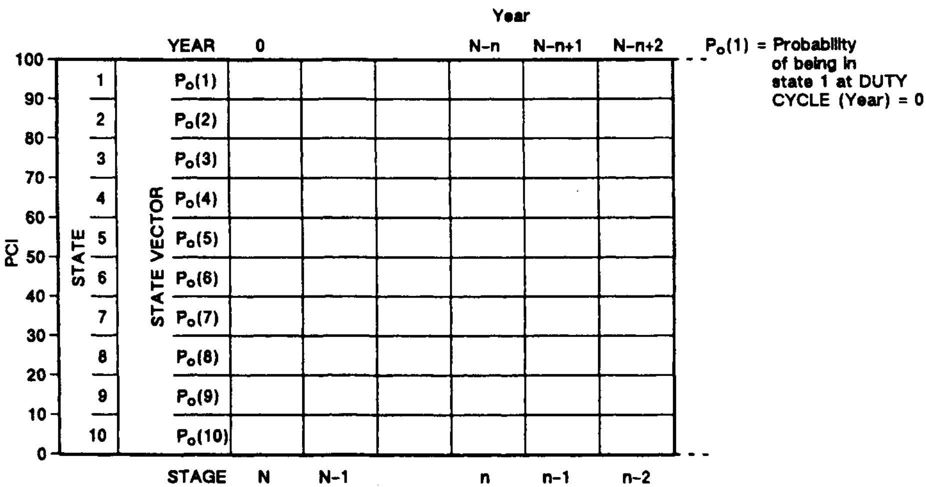

Figure 10-27. Markov Schematic Representation.

10.5.2 Objective Function

The objective function is to optimize the return. If the return is measured in terms of cost, the objective function is to minimize the expected cost over a specified life cycle, subject to keeping all pavement families (or a specified pavement family) above a defined condition (state).

10.5.3 Inputs for the Dynamic Programming Algorithm (Feighan et al. 1989)

The inputs required for the dynamic programming algorithm are:

1. Markov transition probabilities for state i of family j:

   P_{ij}; i = 1....10 states

   j = 1,...m families

2. Cost of applying treatment k to family j in state i:

   C_{ijk}; k = 1,..,n maintenance alternatives.

   Routine maintenance is always designated as $k = 1$. The cost is entered on a dollar per square yard basis.

3. Feasibility indicator for alternative k when in state i of family j:

   R_{ijk} = 1 if maintenance alternative is feasible.

   = 0 if maintenance alternative is infeasible.

4. Number of years in the life-cycle analysis: N.

5. Interest rate: r.

6. Inflation rate: f.

7. Rate of increase in funding: q.

8. The associated benefit over 1 year of being in state i:

   $B_i = 95, 85,.., 5$ for $i = 1,2,.., 10$

   The benefit is taken to be the area below the PCI curve over a period of 1 year.

9. The minimum allowable state for each family; the lowest state that the network manager will allow a particular family to deteriorate to before performing some major maintenance. This is designated as S_j.

10. The transformations that define the new family to move to if treatment k is applied in family j: (j,k).

10.5.4 Dynamic Programming Algorithm (Feighan et al. 1989)

The dynamic programming process starts at year N, the final year of the life-cycle analysis. In dynamic programming terms, this is stage 0. Effectively, the life-cycle cost analysis is being performed over 0 years at this stage.

The first step in the algorithm is to calculate the routine maintenance cost for each state in every family in year N. Routine maintenance is not feasible if (a) $R_{ijk} = 0$, or (b) condition below allowable minimum. If routine maintenance is not feasible, a very large value is added to the cost to ensure that it will not be chosen as the cheapest alternative.

All other feasible alternative costs are also calculated for all states in each family. The optimum repair strategy for each state in year N is then given by:

$$C^*_{ij,N} = \text{MIN}\,[\,C_{ij1,N}, C_{ijk,N}\,] \text{ for all } i,j. \qquad (10\text{-}1)$$

where $C^*_{i,j,N}$ is the optimum cost for state i, family j, and year N

In general, the decision process can be described for year N-n, or equivalently for stage n. As before, routine maintenance is examined for feasibility. If routine maintenance is found to be feasible, the following expression is used to calculate the total present worth of applying routine maintenance now when the analysis period is n years long:

$$C_{ij1,N-n} = \underset{(a)}{C_{ij1}} + \underset{(b)}{[P_{ij}C^*_{ij,N-n-1} + (1-P_{ij})C^*_{i-1,j,N-n-1}] * (1+f)/(1+r)} \qquad (10\text{-}2)$$

This expression is composed of two parts. The part indicated by (a) is the immediate cost of routine maintenance in year N-n, while (b) is the total expected cost to be incurred in the remaining n years as a consequence of applying routine maintenance in year N-n. As shown in Figure 10–28, this expected cost is obtained by identifying the probability of remaining in a given state, multiplying this probability by the expected cost of that state, and then finding the associated probability of dropping a state if routine maintenance is applied and multiplying this by the expected cost of the lower state. This sum is then discounted to bring the total into present worth dollars in the year N-n.

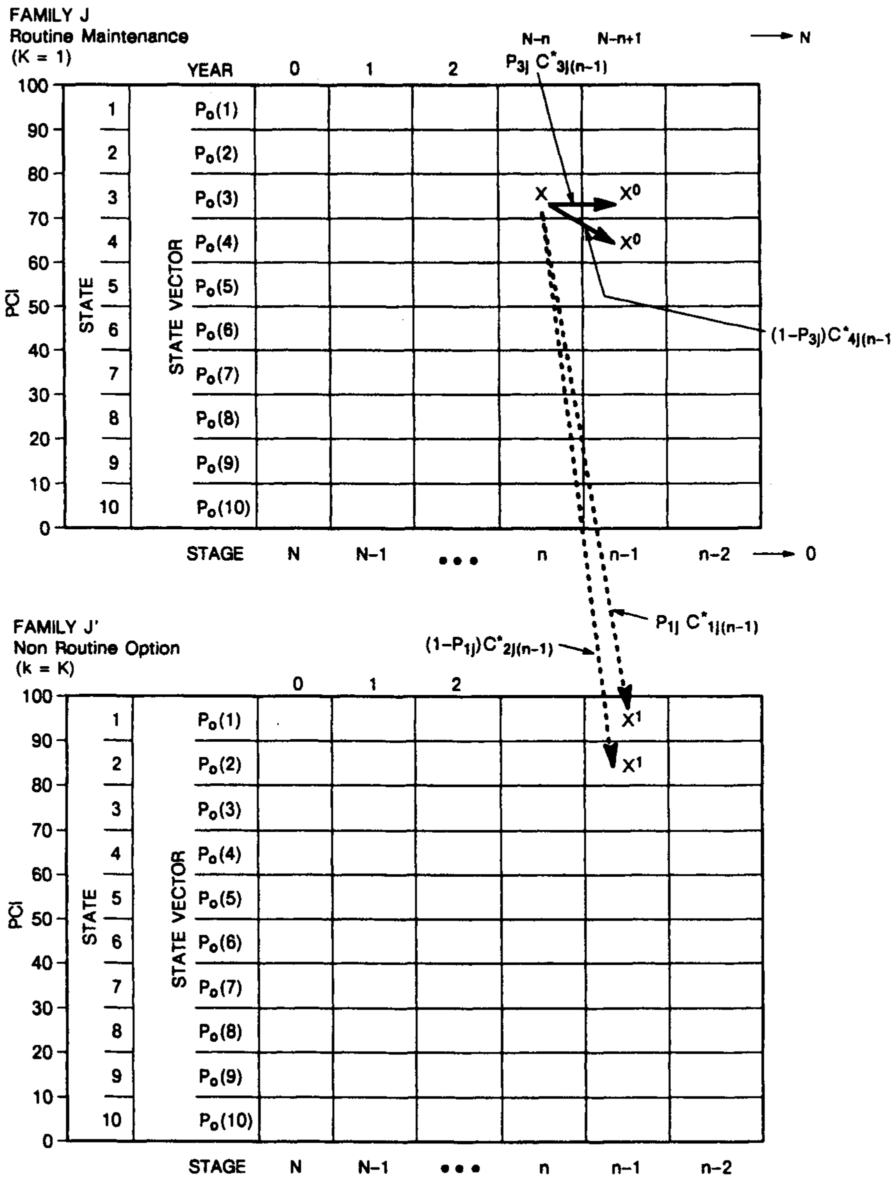

Figure 10-28. Calculation of Expected Costs for Any Given Year.

Similarly, the cost of all other feasible maintenance alternatives can be calculated. The expression used is:

$$C_{ijk.N\text{-}n} = C_{ijk} + [p_{ij} C^*_{ij'.N\text{-}n-1} + (1\text{-}p_{ij}) C^*_{2j'.N\text{-}n-1}] * (1+f)/(1+r) \qquad (10\text{-}3)$$

This expression differs from the expression for routine maintenance in that it is known that the pavement condition will return to state 1 after the repair alternative is carried out. The family to which the pavement moves to, j', as a result of having this alternative performed is defined in the input transformation matrix.

This backward recursion is performed for every successive year of the analysis period until the analysis for year 0, or stage N, is reached.

10.5.5 Dynamic Programming Output (Feighan et al. 1989)

The output from dynamic programming consists of:

1. The optimal maintenance alternative in every year for every family/state combination.
2. The discounted present worth costs expected to be accrued over the life cycle specified if the optimal decisions are implemented.
3. The expected benefit accrued as a result of following the optimal decisions, calculated for every family/state combination.
4. The calculated benefit/cost ratio for every family/state combination.

Thus, it is only necessary to define the family/state combination for any particualr section and the optimal maintenance alternative with associated costs and benefits are readily obtained.

Example (Feighan et al. 1989):

A short example follows to illustrate how the program works. Network performance curves were developed based on PCI condition surveys. Family performance curves were developed based on branch use and surface type. For the branch use of "roadway," four families were defined: asphalt concrete, surface treated, functional overlay, and structural overlay.

Four maintenance alternatives were considered: routine maintenance, surface treatment, functional overlay, and structural overlay. Dollar cost as a function of PCI was defined for both initial repair cost and subsequent routine maintenance cost. Markov probabilities were calculated for each family and the probability transition matrices were obtained.

A minimum allowable state of 7 (PCI of 30 to 40) was specified. In other words, if the condition of the pavement section falls to between 30 and 40, it must be repaired. It is, of course, very possible that the section will be chosen for repair at a greater PCI.

The dynamic programming results are shown in Figure 10–29 for a 25-year life cycle analysis. The optimal decisions corresponding to the numbers shown are:

1. Routine maintenance
2. Surface treatment
3. Functional overlay

To obtain the optimal treatment for any section in the network, it is only necessary to decide what state and family the section is currently in, and look up the optimal treatment for that family/state combination in Figure 10–29. The minimum allowable states and/or interest and inflation rates used can be varied to determine their effect on the optimal decisions reached through dynamic programming.

Family	State	Optimal Decision	Optimal Cost
Family 1	1	1	0.48
	2	1	3.73
	3	2	4.59
	4	2	4.96
	5	3	6.26
	6	3	7.51
	7	3	9.43
Family 2	1	1	2.39
	2	1	3.41
	3	2	4.35
	4	2	5.59
	5	3	8.61
	6	3	10.51
	7	3	12.08
Family 3	1	1	1.07
	2	1	3.38
	3	2	4.35
	4	2	5.59
	5	3	8.61
	6	3	10.51
	7	3	12.08
Family 4	1	1	0.58
	2	1	2.97
	3	1	3.89
	4	2	4.35
	5	3	5.47
	6	3	6.57
	7	3	8.49

Figure 10-29 Dynamic Programming Results for 25–year Analysis.

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11

Project-Level Management

This chapter provides guidelines for conducting project-level investigations and selecting the best Maintenance and Rehabilitation (M&R) alternative for a project. Pavement design procedures are not included in this chapter.

Project-level evaluations should be performed before preparing plans and specifications for a given M&R project. The data obtained from the project-level evaluation will be used in the design process.

11.1 Background Data Collection

11.1.1 Construction and Maintenance History

Knowledge of the construction and maintenance history of a pavement is of great importance to project development. Construction and maintenance historical data to be gathered should include the following:

* Pavement structure and date of original construction,
* Dates and thicknesses of any subsequent overlays,
* Maintenance history including joint and crack sealing, surface treatment application, and patching,
* Properties of materials used in each construction phase.

The construction and maintenance historical information is necessary to properly design rehabilitation alternatives and to provide valuable feedback on what did and did not work for that specific site. Following are examples of such feedback:

1. A pavement was originally constructed in 1940. It received an AC overlay after 20 years, a second overlay 10 years later, and a third overlay 5 years later. It is obvious that a fourth overlay may not be cost effective.
2. A slurry seal was applied 6 months ago; however, it has been sheared off in many places. Different methods of slurry seal application should be considered or slurry seals should be avoided in future rehabilitation of this facility.

3. Cracks and joints on a pavement were never maintained. The life of this pavement is relatively low compared to others where this type of maintenance was applied. Joint and crack sealing should be a major consideration in any future maintenance.

11.1.2 Traffic History

The traffic record includes both traffic history and projected future traffic. An accurate traffic record is essential for assessing past damage and determining an effective rehabilitative design that takes into consideration future traffic. Figure 11–1 is an example of historical traffic data for a major airport. For airports it is also important to determine the distribution of traffic among the branches and sections.

11.1.3 Project-Level Pavement Condition Index (PCI) Inspection

Because the results of a project-level inspection will be used in detailed analysis of the section, the section PCI and the distress types, severity, and amounts must be accurate. The number of sample units to be surveyed for a desired level of confidence can be determined as described in Chapter 3. However, because the quantity of distresses is also used in preparing plans and specifications at the contract level, a 100% survey may be necessary.

11.1.4 Drainage

The condition of the drainage structures and the pavement section's overall ability to drain must be investigated during the detailed distress survey. Specific items that should be looked for in the field are:

- Is the storm sewer performing as designed?
- Are inlets and culverts clear and set at proper elevations?
- Is water standing on the pavement?
- Where appropriate, are ditch lines clear and free of standing water? Inspectors should always be aware of moisture induced distresses that can worsen moisture damage.

11.1.5 Nondestructive Deflection Testing (NDT)

Nondestructive deflection testing (NDT) provides valuable information for project analysis. Many types of NDT equipment are available and were discussed in Chapter 4. Experienced engineering judgment must be used to interpret and use NDT data properly. NDT results are used to determine the following information.

1. Asphalt pavements
    - Elastic modulus of each of the structural layers, which in turn is used for load fatigue analysis.
    - Overlay thickness design.

Figure 11-1. Example Historical Traffic Data for Major Airport.

1962-1982
Estimated Historical Jet Ramp Activity
(for Total Operations Mutiply by Two)

Year	B747	DC10/L-1011	B-707/DC-8	Concorde	B-727	B-737 DC-9/BAC-111
1962	0	0	773	0	0	0
1963	0	0	8,743	0	0	0
1964	0	0	8,797	0	364	0
1965	0	0	9,151	0	2,396	436
1966	0	0	9,280	0	3,397	1,010
1967	0	0	11,391	0	6,184	481
1968	0	0	13,668	0	8,126	927
1969	0	0	14,364	0	9,167	2,089
1970	36	0	15,218	0	7,791	2,277
1971	762	15	13,972	0	7,701	699
1972	1,479	642	12,236	0	7,757	1,789
1973	1,608	1,320	12,390	0	7,549	1,046
1974	1,260	1,131	11,579	0	6,557	806
1975	1,445	1,171	11,114	0	7,615	865
1976	1,688	2,230	10,392	148	8,492	438
1977	1,074	3,432	8,352	200	8,374	774
1978	941	2,690	9,615	231	7,497	790
1979	1,022	2,680	8,059	231	7,833	731
1980	737	3,130	3,725	231	6,016	382
1981	1,223	2,811	1,050	231	6,933	192
1982	1,265	2,907	1,086	200	7,169	199
Total	14,540	24,159	194,955	1,472	126,918	15,921

- Deflection profile for both trafficked and nontrafficked areas. The profile is used to identify failed areas or those with the potential for failure. Higher deflection of trafficked areas compared with nontrafficked areas indicates a structural inadequacy or potential failure, assuming the pavement has the same construction history in both areas. Figure 11–2 is an example of maximum deflection profile for the central 50 ft of a runway as compared to the two outside 50 feet. This profile indicates serious load carrying capacity deficiency as evident by the higher central deflections. The runway has the same pavement structure and construction history. Figure 11–3 shows how deflection normally changes with traffic loading. Assuming temperature and seasonal variations have been accounted for, deflection will not change until close to failure where it will increase rapidly.

2. Concrete pavements
   - Load transfer across joints
   - Void detection
   - Concrete elastic modulus and subgrade modulus of reaction, which are used (along with load transfer) to determine critical stresses and perform a fatigue analysis
   - Overlay thickness design

NDT offers several advantages over destructive testing, including the ability to test hundreds of locations in the same amount of time it takes to perform only a few tests of the field— California Bearing Ratio (CBR), or subgrade modulus (k) destructive tests. Also, the results obtained from NDT are true *in situ* values in contrast to destructive testing results for which undisturbed samples are difficult to obtain. However, destructive testing may be necessary in some cases as discussed below.

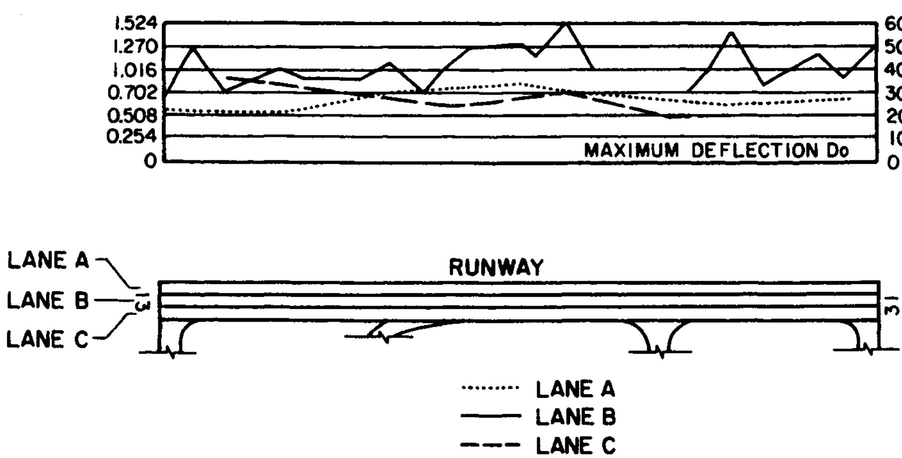

Figure 11-2. Example Deflection Profile.

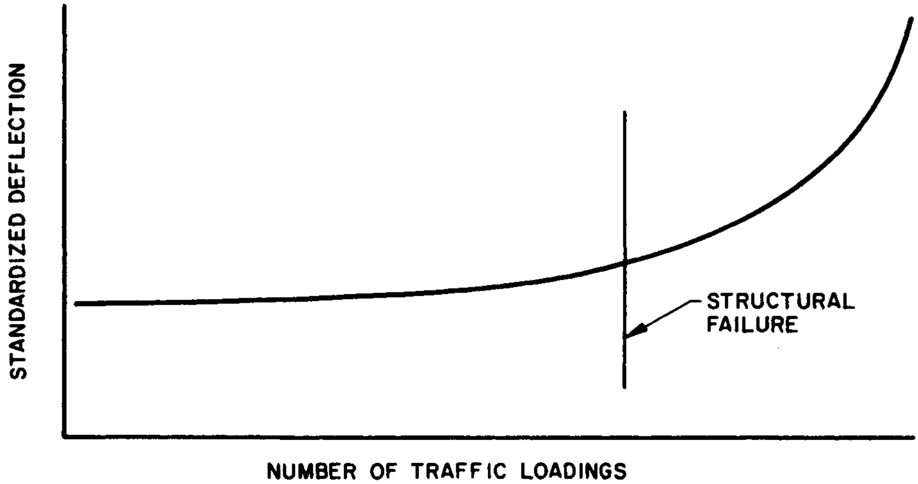

Figure 11-3. Typical Deflection Increase Response After Structural Failure.

11.1.6 Destructive Testing

Destructive testing can be used to supplement NDT results or to provide necessary information without NDT. With the current state-of-the-art technology in pavement analysis, it is desirable to combine limited destructive testing and NDT to achieve the most accurate results. For accurate back-calculation of the layer properties, it is strongly recommended that exact layer thickness be determined by coring in locations where NDT results are going to be used for back-calculation. As a supplement for NDT, the following destructive tests can be used.

1. Coring to determine exact layer thickness.
2. Unified subgrade soil classification in a few representative locations.
3. Visual classification of the base and subbase materials and their conditions in a few representative locations.
4. For asphalt pavements, Marshall stability testing on a few asphalt concrete cores as well as penetration and viscosity (and/or softening point) on extracted asphalt. Based on the NDT results, it may be desirable to perform a few modulus of resilience tests on the asphalt cores to verify the back-calculation.
5. For concrete pavements, indirect tensile strength or compressive strength on a few representative samples.

Some of the following tests may also be used based on field conditions and the results from NDT. If no NDT is performed, a much more extensive destructive testing program is recommended to include the following tests.

1. Field California Bearing Ratio (CBR): The CBR test (Department of the Army 1987) measures the soil resistance to penetration. The test is conducted on the subgrade and granular layers for flexible pavements. A schematic view of the field CBR apparatus is shown in Figure 11–4. The load required to jack the piston to penetrate the soil is measured at various penetration depths up to 0.5 in. The piston has a diameter of 1.95 in. (3 sq. in.). A surcharge weight is also applied around the piston to simulate the weight of the pavement above the soil being tested. The unit load in pounds per square inch (psi) is plotted against depth of penetration in inches. It is sometimes necessary to adjust the zero point of the curve to correct for an initial concave upward shape, which may develop due to surface irregularities. CBR values are obtained by dividing loads at 0.1 in. and 0.2 in. by the standard loads of 1,000 and 1,500 psi, respectively. These loads represent the loads required to penetrate a well graded, minus 3/4 in., crushed limestone. Each ratio is multiplied by 100 to obtain the CBR in percent. The CBR is usually selected at 0.1 in. If the CBR at 0.2 in. is greater, the test is rerun. If check tests give similar results, the CBR at 0.2 inch is used. Figure 11–5 provides typical CBR ranges for different soils.

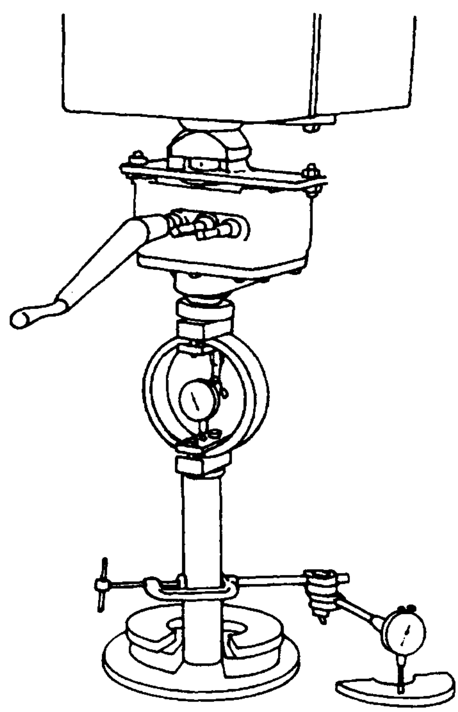

Figure 11-4. Assembled Apparatus, Field In-Place CBR Test (Dept. of the Army 1987).

2. Dynamic Cone Penetrameter (DCP): Similar to the CBR, the DCP measures soil resistance to penetration. The field CBR, however, is much more time consuming to run than the DCP. The DCP, Figure 11–6, consists of a 5/8-in. diameter steel rod with a steel cone attached to one end. The cone is driven into the pavement layers being tested by dropping either an 8-kg (17.6 lb) or 4.6 kg (10.1 lb) sliding hammer from a height of 22.6 in. (575 mm). The depth of cone penetration is measured at selected penetration or hammer drop intervals and the soil strength is reported in terms of DCP index. The cone must penetrate a minimum of 25 mm between recorded measurements. The penetration measurements are recorded to the nearest 5 mm. The test is complete when the cone has been driven to the desired depth (maximum 39 in.). The DCP index is calculated as a ratio in mm per blow for the 8-kg hammer. If the 4.6-kg hammer is used, multiply the ratio by 2 to obtain the DCP index value. Figure 11–7 shows a plot of the correlation of CBR versus DCP index. The correlation was developed by the U.S. Army Corps of Engineers (Webster, Grace, and Williams 1991) based on a database of field CBR vs. DCP index values collected from many sites and different soil types. A useful presentation of the DCP test data is a plot of CBR versus depth as shown in Figure 11–8.

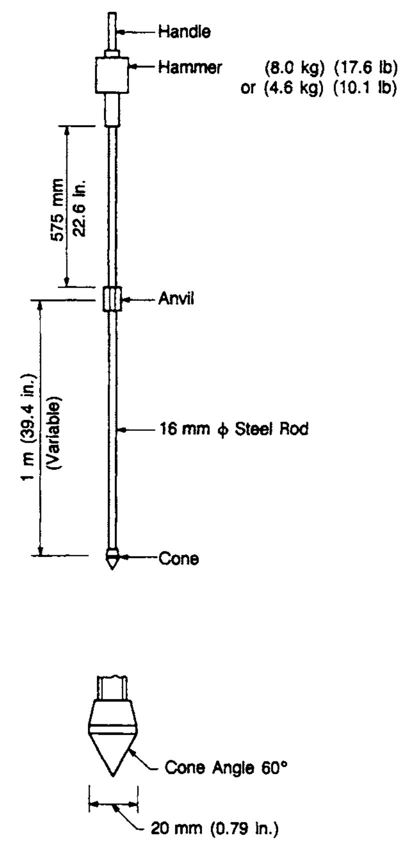

Figure 11-6. Dual Mass Dynamic Cone Penetrometer (Webster et al. 1991).

Figure 11-5.

Major Divisions (1)	(2)	Letter (3)	Name (4)	Value as Foundation When Not Subject to Front Action (5)	Value as Base Directly under Wearing Surface (6)	Potential Front Action (7)	Compressibility and Expansion (8)	Drainage Characteristics (9)	Compation Equipment (10)	Unit Dry Weight (pcf) (11)	Field CBR (12)	Subgrad-e Modulus, k (pci) (13)
		GW	Gravel or sandy gravel, well graded	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	125-140	60-80	300 or more
		GP	Gravel or sandy gravel, poorly graded	Good to excellent	Poor to fair	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	120-130	35-60	300 or more
	Gravel and gravelly soils	GU	Gravel or sandy gravel, uniformly graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	115-125	25-50	300 or more

		GM	Silty gravel or silty, sandy gravel	Good to excellent	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	130-145	40-80	300 or more
Coarse--grained soils		GC	Clayey gravel or clayey, sandy gravel	Good	Poor	Slight to medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120-140	20-40	200-300
		SW	Sand or gravelly sand, well graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	110-130	20-40	200-300
		SP	Sand or gravelly sand, poorly graded	Fair to good	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	105-120	15-25	200-300
	Gravel and gravelly soils	SU	Sand or gravelly sand, uniformly graded	Fair to good	Not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	100-115	10-20	200-300

Continued

Figure 11-5. Continued...

Major Divisions (1)	(2)	Letter (3)	Name (4)	Value as Foundation When Not Subject to Front Action (5)	Value as Base Directly under Wearing Surface (6)	Potential Front Action (7)	Compressibility and Expansion (8)	Drainage Characteristics (9)	Compation Equipment (10)	Unit Dry Weight (pcf) (11)	Field CBR (12)	Subgrad-e Modulus, k (pci) (13)
		SM	Silty sand or silty, gravelly sand	Good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	120-135	20-40	200-300
		SC	Clayey sand or clayey, gravelly sand	Fair to good	Nout suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105-130	10-20	200-300
		ML	Silts, sand silts, gravelly silts, or diatomaceous soils	Fair to poor	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	100-125	5-15	100-200

	Low compres-sibility LL<50	CL	Lean clays, sandy clays, or gravelly clays	Fair to poor	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired equipment, sheepsfoot roller	100-125	5-15	100-200
Fine grained soils		OL	Organic silts or lean organic clays	Poor	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired equipment, sheepsfoot roller	90-105	4-8	100-200
		MH	Micaceo-us clays or diatomac-eous soils	Poor	Not suitable	Medium to very high	High	Fair to poor	Rubber-tired equipment, sheepsfoot roller	80-100	4-8	100-200
	High compres-sibility LL>50	CH	Fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	90-110	3-5	50-100
		OH	Fat organic clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	80-105	3-5	50-100
Peat and other fibrous organic soils		Pt	Peat, humus, and other	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical			

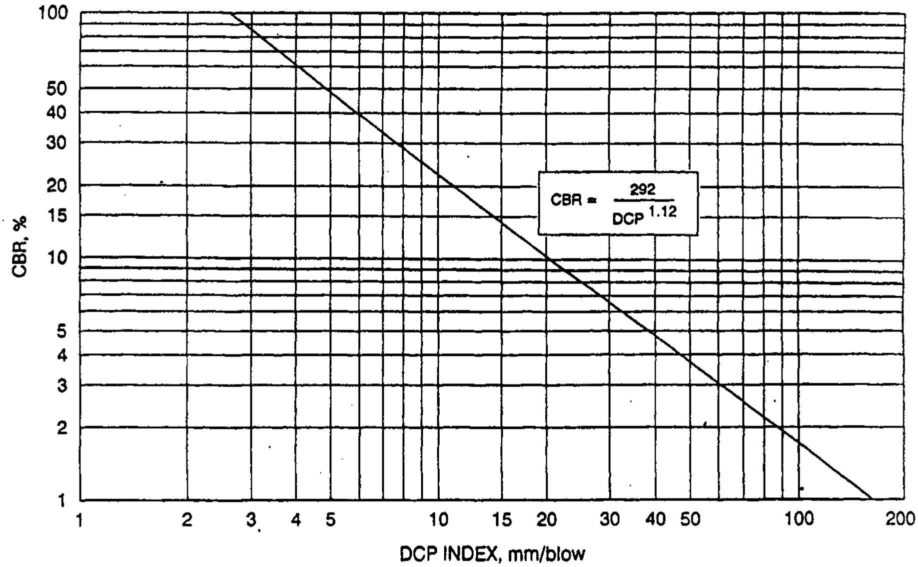

Figure 11-7. Correlation Plot of CBR vs. DCP Index (Webster et al. 1991).

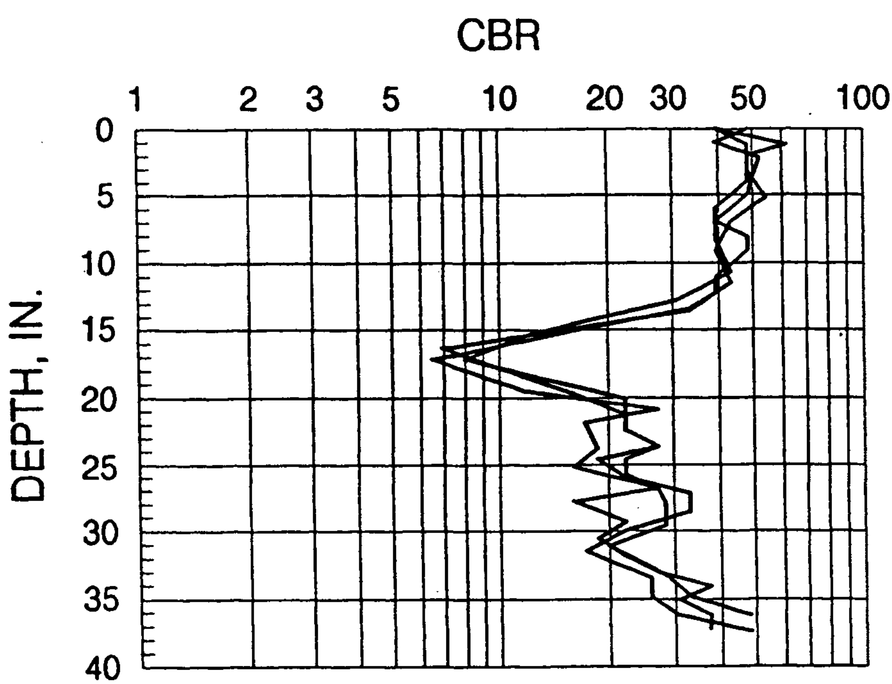

Figure 11-8. Example of DCP Data Plot for Three Tests in Similar Type Soils (Webster et al. 1991).

3. Field subgrade modulus: The modulus of subgrade reaction (K) (Department of the Army 1987), which is determined using the plate load test, is used in the evaluation and design of concrete pavements. The *K* value is the load per unit area per unit deformation of the subgrade expressed in psi per inch of deformation. The test is performed on representative areas, and corrections are made based on the results. The setup for the plate load test is shown in Figure 11–9. Loads are applied by a hydraulic jack working against a jacking frame and through a 30-in. diameter steel bearing plate. A nest of plates (30-in., 24-in., and 18-in. diameter) is used to help center the load and minimize plate bending. The movement of the plate due to the applied loads is measured by three dial gauges (0.0001 in. accuracy) placed on the 30-in. diameter plate 120 degrees apart and 1/4 inch from the rim. The loading plate is seated by applying 1 psi for pavements <15 in. thick and 2 psi for pavements >15 in. thick. An additional 10 psi is applied and held until complete deformation has taken place (rate of deformation <0.0002 in. per minute). A preliminary subgrade modulus, $K'u$, is computed as follows:

$$K'u = 10 \text{ psi/deflection in inches}$$

For cohesive subgrades, where $K'u$ is less than 200 psi, the test is considered complete. For granular subgrades or when $K'u$ is 200 psi or more, the load is applied in

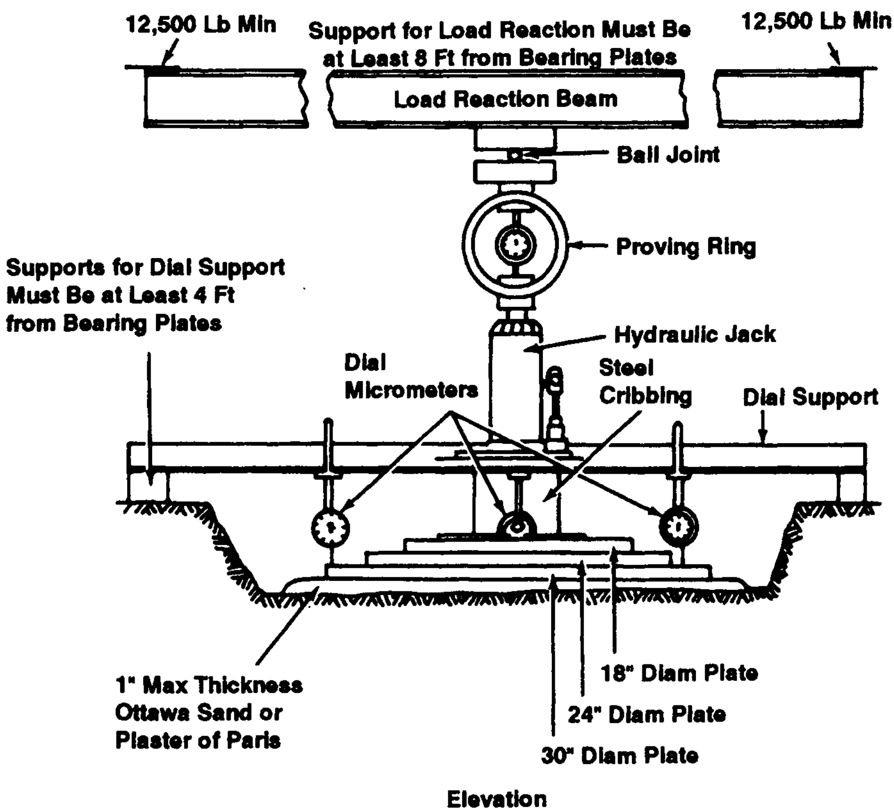

Figure 11-9. Plate Bearing Test Loading (Department of the Army 1987).

successive increments of 5 psi to a maximum of 30 psi. A curve of the unit load vs. deformation is plotted. If the curve approximates a straight line, contains the 10 psi and 30 psi loadings, and goes through the "0" point, no correction is needed. Otherwise, the correction is performed as shown in Figure 11–10. The $K'u$ should be corrected further for plate bending and soil saturation before it is used in pavement evaluation. Since the deflection is measured at the rim, the measurement is smaller than if taken at the center, and $K'u$ is higher. The correction for bending is performed using Figure 11–11, and the resulting subgrade modulus is termed Ku. The correction for saturation is performed using the consolidation test.

The value of the corrected subgrade modulus (K) is determined as follows:

$$K = (d/ds) * Ku$$

where:

K = The subgrade modulus corrected for saturation and bending of the plate

Ku = The subgrade modulus, uncorrected for saturation, but corrected for bending of the plate

d = The deformation of a specimen at field condition under a load of 10 psi

ds = The deformation of a saturated specimen under a load of 10 psi

The value of d/ds is limited to <1. The last two columns of Figure 11–5 show typical K and CBR values based on soil classification.

Destructive testing may also be necessary to investigate special problems such as D cracking in concrete pavements or reflection cracking in asphalt pavements.

11.1.7 Roughness and Skid Resistance

Roughness and skid resistance measurements are not necessary for every project-level evaluation. Roughness measurement is most valuable when the pavement is in very good condition with little or no distress but users have complained about roughness. If reconstruction is imminent, roughness measurements of the existing pavement may not be of any value. Accident records can indicate locations with low skid resistance. However, for pavements such as runways and major highways, skid resistance should be measured regularly to ensure safety. Roughness and skid resistance measurements were presented in Chapter 5 and 6, respectively.

11.2 Pavement Evaluation

The selection of feasible M&R alternatives should be based on the results of the evaluation. Figure 11–12 shows a step-by-step procedure that can be used to summarize the results of an evaluation. This procedure provides a rational basis for identifying feasible alternatives. The following paragraphs describe each step in this procedure and how it should be completed.

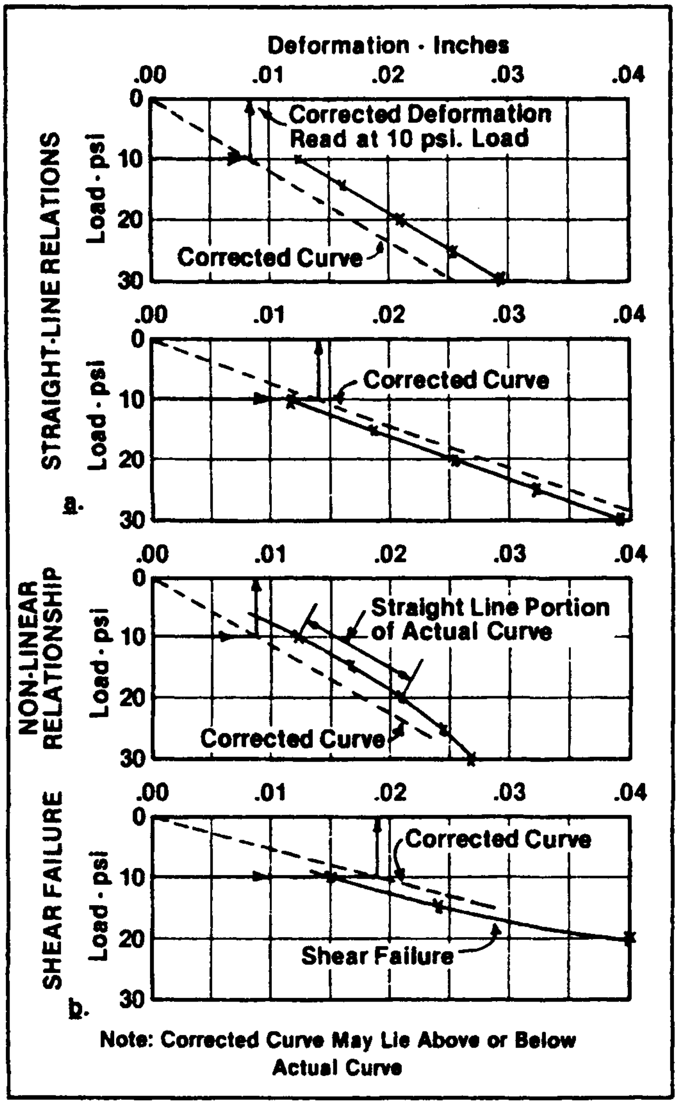

Figure 11-10. Zero Corrections to Load-Deformation Curves (Department of the Army 1987).

Step 1: Overall Condition. The section mean PCI is determined by computing the average of all sample units inspected within the section (adjusted if additional nonrandom units are included—see Chapter 3).

Step 2: Variation of PCI. Variations of materials, construction, subgrade, or traffic loadings may cause certain portions of a given section to show a significantly different condition than the average of the overall section. Areas having a poorer condition are of major concern. Variation within a section occurs on both a localized random basis and a systematic basis. Systematic variation occurs when a large concentrated area of the section has a condition significantly different from the rest. For example, if traffic is

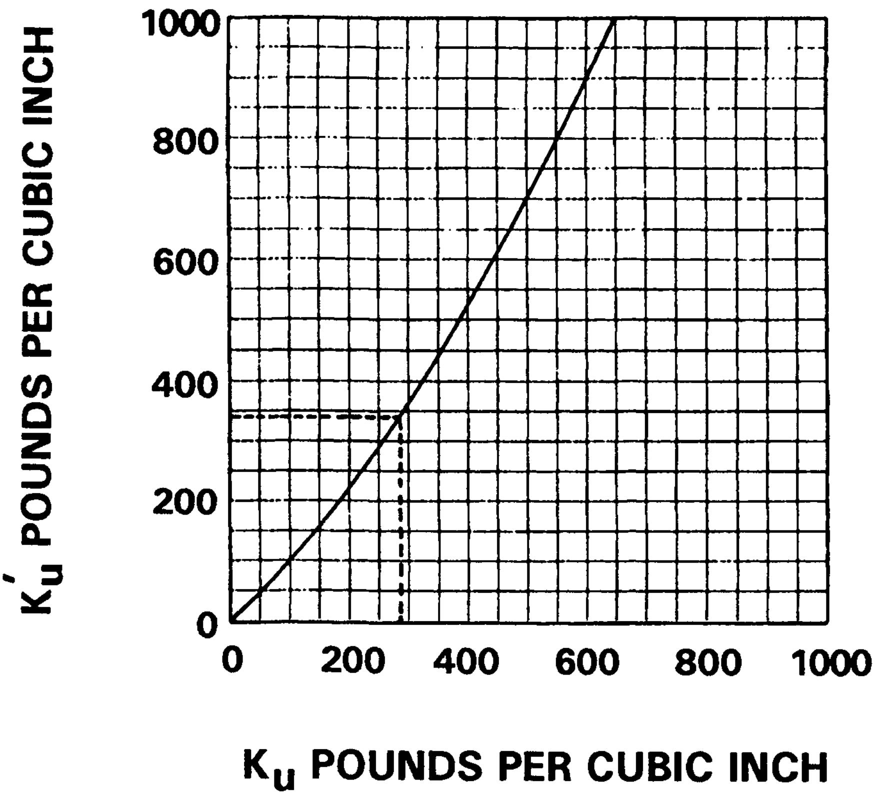

Figure 11-11. Plate Bending Correction for *Ku* (Department of the Army 1987).

channeled into a certain portion of the section, that portion may show much more distress than the other sections. When a significant amount of systematic variability exists within a section, strong consideration should be given to dividing it into two or more sections.

Unlike systematic variation where a change in condition occurs in a relatively large concentrated area (such as one street block or one lane), a localized random variation is one where a smaller area or a few small areas show much worse conditions than the section average. A localized random variation might point to a localized problem, such as a soft subgrade spot or poor compaction around a culvert, which should be corrected.

Step 3: Rate of Deterioration. The long-term rate of deterioration is determined by comparing a section with the deterioration rate of other pavements in the same family. A family of pavements (see Chapter 7) is defined as those with the same surface type [asphalt concrete(AC), Portland cement concrete (PCC), etc.)], pavement use (runway, roadway, etc.), pavement rank (primary, secondary, etc.), level of traffic (trafficked, nontrafficked), and other factors that might affect pavement performance.

A family's rate of deterioration can be analyzed using the PAVER System Family Analysis Report. This report plots PCI vs. age, as shown by the example in Figure 11–13. The figure shows an envelope containing most of the data (95% confidence limits).

1. **Overall Condition Rating - PCI**

Rating -	**Failed,**	**Very Poor,**	**Poor,**	**Fair,**	**Good,**	**Very Good,**	**Excellent**
PCI	**0–10**	**11–25**	**26–40**	**41–55**	**56–70**	**71–85**	**86–100**

2. **Variation of Condition Within Section - PCI**
   a. **Localized Random Variation** — **Yes, No**
   b. **Systematic Variation** — **Yes, No**

3. **Rate of Detioration of Condition - PCI**
   a. **Long-term Period, Since Construction or Last Overall Repair** — **Low, Normal, High**
   b. **Short-term Period, 1 Year** — **Low, Normal, High**

4. **Distress Evaluation**
   a. **Cause**
      Load Associated Distress — ___ **Percent Deduct Value**
      Climate/Durability Associated — ___ **Percent Deduct Value**
      Other Associated Distress — ___ **Percent Deduct Value**
   b. **Moisture, Drainage, Effect on Distress** — **Minor, Moderate, Major**

5. **Deficiency of Load-Carrying Capacity** — **No, Yes**

6. **Surface Roughness** — **Minor, Moderate, Major**

7. **Skid Resistance/Hydroplaning Potential** — **Minor, Moderate, Major**

8. **Previous Maintenance** — **Low, Normal, High**

9. **Comments:**____

____

____

____

____

____

Figure 11-12. Stepwise Procedure for Section Evaluation Summary.

Pavement sections located within that envelope are classified as deteriorating at a normal rate; those above the envelope are classified as having a low deterioration rate; and those below the envelope as having a high deterioration rate.

The pavement's rate of deterioration must also be estimated based on a short-term or yearly loss of PCI. When the mean PCI of a section (assuming that only routine M&R is applied) decreases by four or more PCI points, the rate of deterioration should be considered high. If the loss in PCI is two to three points, the short-term rate of deterioration should be considered normal or average. The Micro PAVER Condition Analysis Report (Figure 11–14) provides a PCI time curve for a specific section, including future projection, which will help determine the rate of deterioration. Engineering judgment should be exercised carefully when evaluating the short-term rate of deterioration.

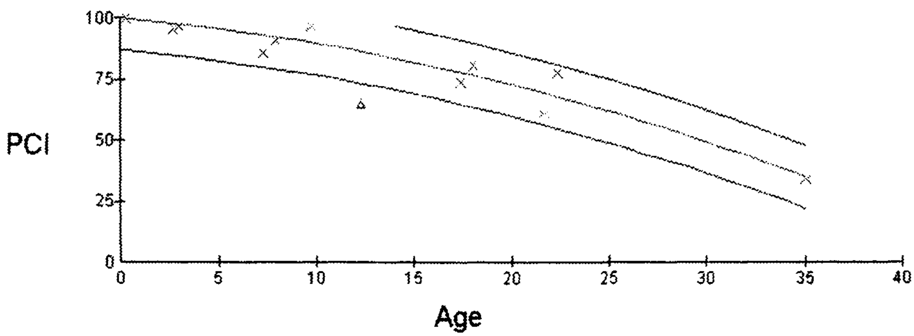

Figure 11-13. Example PCI vs. Age Data with 95 Percent Confidence Limits.

Step 4: Pavement Distress Evaluation. Examination of specific distress types, severities, and quantities provides valuable information used to determine the cause of pavement deterioration, its condition, and eventually its M&R needs. Figures 11–15 and 11–16 generally classify distress types for asphalt and concrete surfaced pavements according to cause and effect on condition. The condition of each pavement will dictate which distresses will be placed into each group.

In the Micro PAVER System, distresses have been classified into three groups based on cause: (1) load-associated, (2) climate-associated, and (3) other factors. Figures 3–23 and 3–24 list distress classification as used in PAVER for paved roads and airfields, respectively.

The percentage of deduct values attributed to each cause is an indication of the cause(s) of pavement deterioration, e.g. distresses caused primarily by load have resulted in 90% of the total deducts, whereas all other causes have produced only 10%. In this example, traffic load is by far the major cause of deterioration for this pavement section. Micro PAVER automatically calculates the percent deduct values attributable to load, climate, and other associated distress for a section when the PCI is calculated.

Pavement drainage should also be evaluated. If moisture is accelerating pavement deterioration, the engineer must determine how it is happening and why (e.g., groundwater, water table, infiltration of surface water, ponding water on the pavement). If moisture is contributing significantly to the rate of pavement deterioration, ways must be found to prevent or minimize this problem.

Step 5: Load-Carrying Capacity Evaluation. The objective of this evaluation is to determine if the existing pavement structure is deficient based on current or expected future traffic. The distress evaluation procedure presented above can be used to determine the pavement structural adequacy with respect to current traffic. Structural analysis for overlay design or analysis for a change in mission can only be done using the results from NDT and destructive testing.

Step 6: Surface Roughness. Three ways are available to evaluate surface roughness. First, user complaints are subjective but highly reliable sources of qualitative roughness information. Second, certain distress types contained in the PCI may be correlated with localized roughness. Third, the roughness can be measured quantitatively using special equipment as described in Chapter 5.

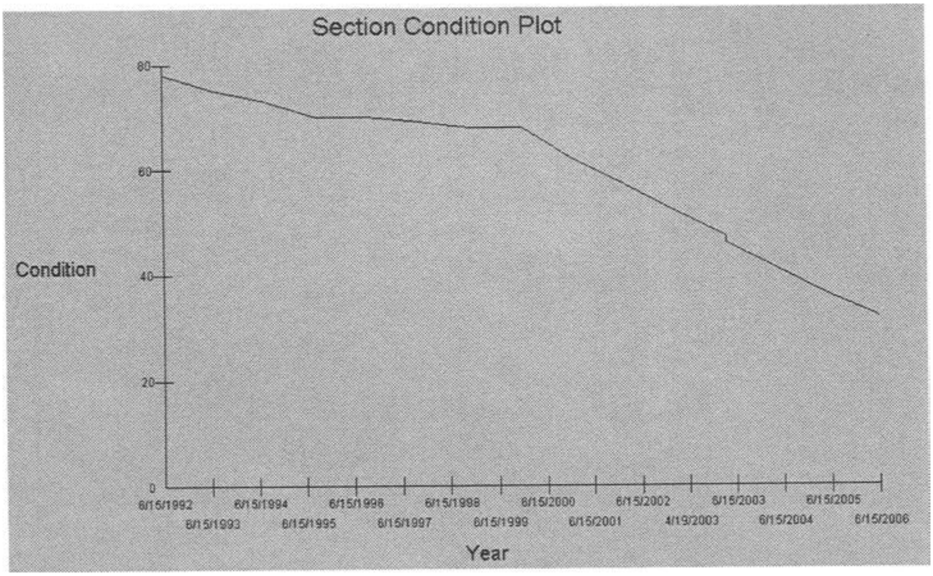

Figure 11-14. Example PCI/Time Curve from the Condition Analysis Report.

Step 7: Skid Resistance and Hydroplaning Potential. Skid resistance can be measured using special equipment, as described in Chapter 6. Also, skid problems can be identified by reviewing accident records.

Step 8: Previous M&R Applied. A pavement section can be kept in operating condition almost indefinitely if extensive M&R is applied continually. However, there are major drawbacks to this maintenance strategy, such as overall cost, downtime of pavement, increase in roughness caused by excessive patching, and limitations of manpower and equipment. The amount and types of previous M&R applied to a pavement section are important factors in deciding what type of M&R is needed. Micro PAVER allows the agency to store records of M&R that have been performed on pavement sections. A pavement for which a large portion has been patched or replaced must have had many previous distress problems that are likely to continue in the future.

Permanent patching of asphalt pavements and large areas of patching (>5 sq ft) or slab replacement of concrete pavement can be used as criteria for evaluating previous maintenance. Patching or slab replacement ranging between 1.5% and 3.5% (based on surface area for asphalt and number of slabs for concrete) is considered normal; >3.5% is considered high, and <1.5% is considered low. Some pavement sections may have received an excessive amount of M&R other than patching. If the engineer finds that a section should be evaluated because of high previous maintenance, this decision should take precedence over evaluation criteria based on only patching and slab replacement.

Step 9: Comments. Any constraints in choosing an M&R alternative should be identified in the comments section.

Figure 11-15.

Classification of Observable Distresses in AC Surfaced Pavement							
Classification by Possible Causes				Classification by Possible Effects			
Load	Climate/ Durability	Moisture/ Drainage	Others	Roughness	Skid/ Hydroplaning Potential	FOD Potential	Rate of Deterioration and Maintenance Requirements
Alligator Cracking	Bleeding	Alligator Cracking	Oil Spillage	Corrugation	Bleeding	Block Cracking	All Distresses
Corrugation	Block Cracking	Depression	Jet Blast Erosion	Depression	Depression	Joint Reflection Cracking	
Depression	Joint Reflection Cracking	Patching of Moisture Caused Distress	Polished Aggregate	Rutting	Polished Aggregate	Long./Trans. Cracking	
Patching of Load Caused Distress	Long./Trans. Cracking	Swelling		Shoving of Ashpalt Pavement	Rutting	Slippage Cracking	
Polished Aggregate	Patching of Climate/Durabilit-y-Caused Distress	Raveling/ Weathering		Swelling		Raveling/ Weathering	
Rutting	Shoving from PCC			Raveling/ Weathering		Jet Blast Erosion	
Slippage Cracking	Raveling/ Weathering					Oil Spillage	
	Swelling						

Figure 11-16.

Classification of Observable Distresses in PCC Surfaced Pavement

Classification by Possible Causes				Classification by Possible Effects			
Load	Climate/ Durability	Moisture/ Drainage	Others	Roughness	Skid/ Hydroplaning Potential	FOD Potential	Rate of Deterioration and Maintenance Requirements
Corner Break	Blowup	Corner Break	Settlement/-Faulting	Blowup	Settlement/Faulting	Corner Break	All Distresses
Shattered Slab	"D" Cracking	Shattered Slab		Corner Break	Spalling, Joint	Long./Trans. Cracking	
Long./Trans. Cracking	Joint Seal Damage	Pumping		Long./Trans. Cracking	Spalling, Corner	"D" Cracking	
Pumping	Popouts	Patching of Moisture-Caused Distresses		Shattered Slab		Joint Seal Damage	
Patching of Load-Associated Distress	Scaling/Map Cracking/Crazing			Settlement/F-aulting		Shattered Slab	
Spalled Key Way Joint	Patching of Climate/Durability Associated Distress			Spalling, Joint		Popouts	
	Shrinkage Cracking			Spalling, Corner		Scaling	
	Spalling, Corner						
	Long./Trans. Cracking						

11.3 Life Cycle Cost Analysis

The selection of feasible M&R alternatives should be based on results of the pavement evaluation discussed earlier. The various types of M&R alternatives were presented in Chapter 8.

Once a list of feasible M&R alternatives has been developed, life-cycle cost (LCC) must be analyzed in order to select the most cost-effective solution.

Several types of costs are used to perform LCC analysis for a given M&R alternative.

1. Initial cost of the alternative (first-year cost)
2. Present value of the alternative (discounted cost of the alternative in present dollars, using interest and inflation rates)
3. Equivalent uniform annual cost (EUAC) of the alternative (present value cost converted to an annuity)
4. EUAC per square yard of pavement

11.3.1 Initial Cost

The initial cost is the present-year cost of the alternative, disregarding any future costs. The initial cost is represented by the symbol CI in this chapter.

11.3.2 Present Value Costs

In economic analysis, the effects of interest and inflation rates are commonly taken into account. The inflation rate is used to adjust the future cost of an M&R alternative according to the following formula:

$$C_{mt} = C_m(1+r)^t \tag{11-1}$$

where:

C_m = the cost of the M&R alternative in present day dollars

r = the annual rate of inflation in decimals

t = the time in the future in years

C_{mt} = the cost of the M&R alternative t years in the future

So that all dollar figures are considered on an equivalent basis, it is common practice to reduce all future costs to their present value by applying an interest rate discount, i. The present value of the future cost, C_{mt}, is:

$$\text{PV} = \frac{C_{mt}}{(1+i)^t} \tag{11-2}$$

where:

PV = the present value; that is, the amount of money that would have to be placed in an interest-bearing account now to be C_{mt} in t years

i = the annual rate of interest in decimals

Combining Equations 11–1 and 11–2, the formula for present value becomes:

$$\text{PV} = C_m \frac{(1+r)^t}{(1+i)^t} \tag{11-3}$$

This formula allows the user to input data in present-day dollars.

In most cases, an M&R alternative consists of a series of M&R activities with associated costs. The present value of a series of M&R costs is found by adding the initial cost, C_I, to the present value of all future costs adjusted for inflation and interest rates. The present value of this series of costs is:

$$\text{PV} = C_I + \sum_{t=1}^{N} \frac{C_{mt}}{(1+i)^t} \tag{11-4}$$

or

$$\text{PV} = C_I + \sum_{t=1}^{N} C_m \frac{(1+r)^t}{(1+i)^t} \tag{11-5}$$

where

N = the number of years in the analysis period

The present-value analysis is a convenient tool in the decision making process because it allows choices to be made in terms of present-day dollars.

11.3.3 Equivalent Uniform Annual Cost

The EUAC is necessary for comparing M&R alternatives with different service lives. To compute the EUAC, the present value is multiplied by the capital recovery factor (CRF). The annual payments over the analysis period are individually discounted and added to obtain the present value (PV).

$$\text{EUAC} = \text{CRF} \times \text{PV} \tag{11-6}$$

where:

$$\text{CRF} = \frac{i(1+i)^N}{(1+i)^N - 1} \tag{11-7}$$

The EUAC is divided by the surface area of the pavement section to determine the cost per square yard.

11.4 Example Project Analysis

11.4.1 Background

A project analysis was conducted for a four-lane urban arterial roadway located in the city of Champaign, Illinois. The project (shown in Figure 11–17) is part of Kirby Avenue. It is 1200 ft long and lies between the general limits of Dodds Drive on the west and Park Haven Drive on the east.

The city initiated the project analysis to assess the load-carrying capacity of the pavement and to determine if structural improvements are required to support projected future traffic. The specific objectives were to answer the following questions:

1. What is the extent and severity of the deterioration?
2. What are the causes of the deterioration?
3. Is a structural improvement needed?
4. What are the feasible rehabilitation alternatives?
5. What is the life cycle cost for each of the rehabilitation alternatives?

11.4.2 Data Collection

The data collection program included construction history, traffic, PCI, NDT, and destructive testing.

11.4.2.1 Construction History.

The road was first constructed in the 1950s with 7 in. of concrete placed at a width of 22 ft. In 1966 the road was widened to four lanes, each 12 ft wide, by adding a 13-ft wide pavement constructed to a depth of 8 in. with contraction joints every 100 ft. A 3-in. asphalt concrete was then placed over the entire width of the roadway.

11.4.2.2 Traffic.

Traffic records showed a two-way average daily traffic (ADT) of 15,400 in 1976 and 18,400 in 1986. This data can be translated into a growth factor of 1.8%.

$$18{,}400 = 15{,}400*(1+0.018)**10$$

This was used to project a 1991 ADT of 20,116. Truck traffic was reported to be 9.5% with 9% single-unit trucks and 0.5% multiple-unit trucks. Using the Asphalt Institute (1981) truck factors for urban areas, single-unit trucks have a truck factor range of 0.04 to 0.21. Thus, one pass of a single-unit truck is equivalent to 0.04 to 0.21 passes of a standard 18-kip single- axle load. Multiple-unit trucks have truck factors ranging from 0.72 to 1.58. Using the upper end of these truck factors, a weighted truck factor was calculated as follows: [{(9*0.21)+(0.5*1.58)}/9.5]=0.28.

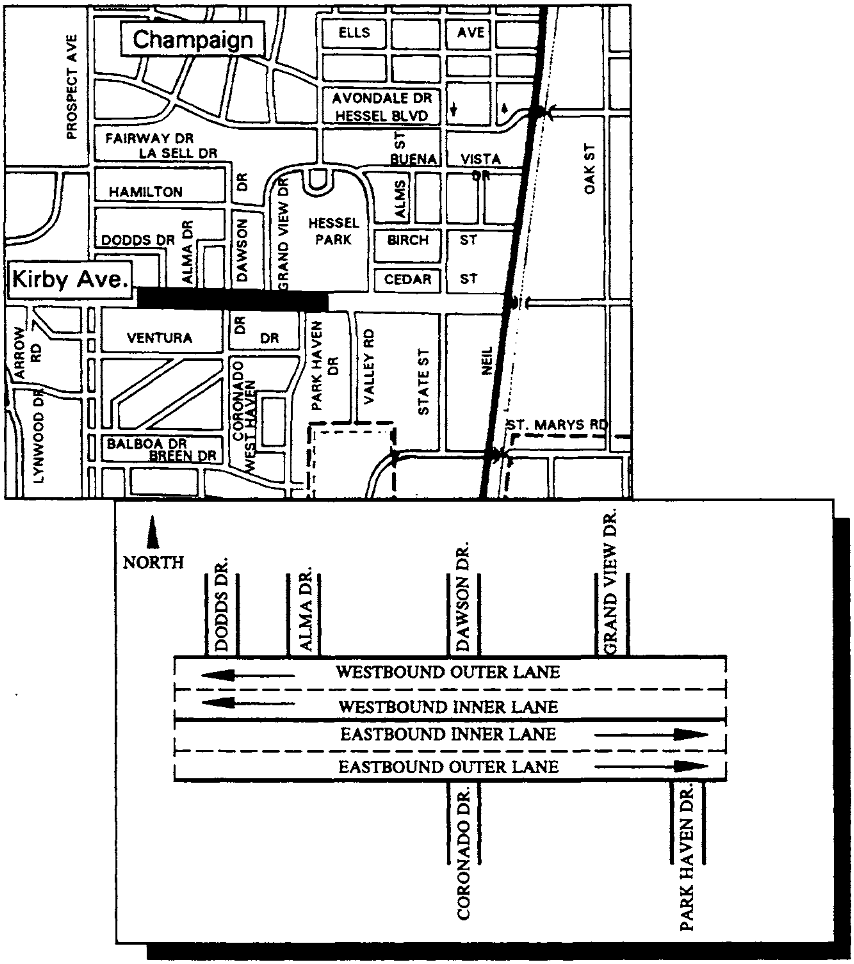

Figure 11-17. Project Location.

Equivalent Single Axle Loads (ESALs) for 1991 were calculated as follows:

$$\text{ESALs/yr} = \text{ADT} * \text{DD} * \text{LD} * \text{ADTT} * \text{TF} * 365 \text{ days}$$

where

ADT = Average Daily Traffic

DD = Directional Distribution factor, assumed 50%

LD = Lane Distribution factor, 0.8 for outer lane, and 0.2 for inner lane.

ADTT = Average Daily Truck Traffic in decimal, 0.095

TF = Truck factor, 0.28.

Thus, the outer lane ESALS/yr = 20,116*0.5*0.8*0.095*0.28*365 = 78,122 and the inner lane ESALs/yr =20,116*0.5*0.2*0.095*0.28*365 =19,530

The total ESALs for a 20-year design life was computed based on a 1.8% growth factor as follows:

$$\text{Total ESALs} = \text{annual ESALS}\,[\{(1+0.018)^{**}20\}-1]/0.018$$
$$= \text{annual ESALs} * 23.82$$

Thus,

the outer lane design ESALs = 78,122 * 23.82 = 1,860,866

the inner lane design ESALs = 19,530 * 23.82 = 465,204

11.4.2.3 PCI Survey.

The pavement was divided into four sections by lane; each lane was divided into six sample units (Figure 11–18). All sample units were surveyed. A plot of the PCI profile for each of the lanes is shown in Figure 11–19. The outer lanes have a higher PCI than the inner lanes. This is in part due to the displaced longitudinal joint in the underlying PCC pavement (13 ft from the outer edge). The inner lanes contained all of the recorded longitudinal joint reflection cracking. The outer lanes exhibited a significant proportion of badly deteriorated transverse joint reflection cracking. The predominant distresses noted during the survey were block cracking, joint reflection cracking, longitudinal and transverse cracking, and utility cut patching. A summary of the distresses by type and severity level is shown in Figure 11–20.

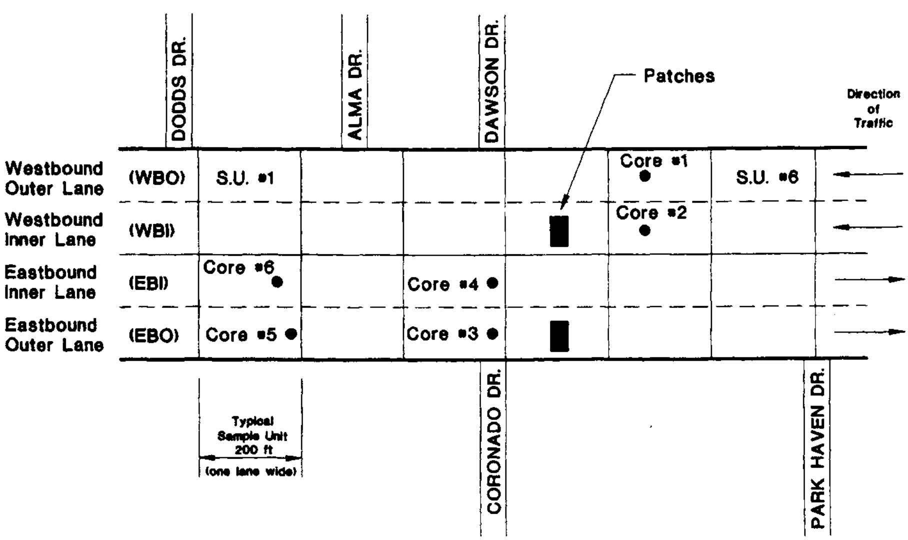

Figure 11-18. Division of Kirby Avenue into Sections and Sample Units.

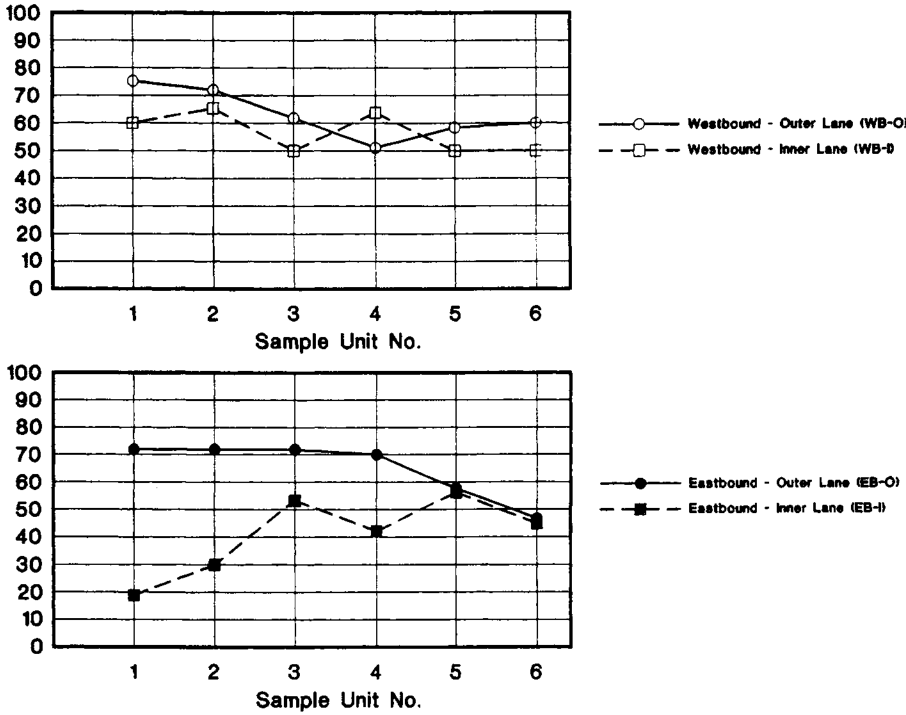

Figure 11-19. PCI Profile for Kirby Avenue.

11.4.2.4 NDT Survey.

The objective of the NDT program was to measure the pavement's structural responses to heavy dynamic loads, such as those produced by moving truck wheel loads. The collected deflection data is used to determine properties of the pavement layer and foundation support, which are then used to determine the pavement structural capacity. The equipment used for the testing on the project was a KUAB two-mass, falling weight deflectometer. Testing was conducted near the outer wheel path of each lane at 50-ft intervals. Figure 11–21 shows the maximum deflection profiles by lane. The inner lanes typically showed higher measured deflections, indicating a weaker pavement system.

11.4.2.5 Destructive Survey.

The purpose of the destructive survey was to verify the pavement layer thicknesses and look for pavement deterioration. Six pavement cores were obtained at locations selected based on the collected NDT data. The extracted cores clearly indicated that the inner lanes (central 22 ft) were significantly deteriorated with multiple cracks at various depths in the PCC layer. An example core log is shown in Figure 11–22. To investigate further the quality of the underlying PCC base slabs, two pavement cuts were made at a high-severity transverse reflection crack which traversed the full width of the pavement. A full-depth saw cut was made through the asphalt surface layer to create a 3 ft by 3 ft opening. The asphalt surface was then removed to expose the underlying PCC slab.

Figure 11-20. Summary of Distress Data for Each Pavement Section.

Distress Type	Severity	Westbound Outer Lane (WB-O)	Westbound Inner Lane (WB-I)	Eastbound Outer Lane (EB-O)	Eastbound Inner Lane (EB-I)
Block Cracking	L	1,770	9,570	1,770	6,258
	M	1,170	1,730	1,140	2,632
	H	--	--	--	960
Reflection Cracking	L	--	864	19	368
	M	--	414	44	555
	H	144	--	48	312
Longitudinal Transverse Cracking	L	398	--	12	66
	M	156	--	15	--
	H	102	--	63	--
Patch and Utility Cut	L	240	216	543	257
	M	--	180	384	384
	H	--	--	--	--
Section PCI (Average of six sample units)		63	57	65	41

Sounding of the PCC slab in the westbound inner lane revealed significant deterioration. Sounding of the pavement in the eastbound outer lane revealed moderate deterioration.

11.4.3 Deflection Data Analysis

The deflection data analysis included materials characterization, load transfer across cracks, and utility cut patching quality.

11.4.3.1 Materials Characterization.

The layer material moduli were back-calculated based on the measured deflections. Figure 11–23 shows a profile of the PCC modulus along each of the sections (lanes). As shown, the inner lanes typically have PCC moduli values significantly lower than the outer lanes. This confirms the observations based on the limited destructive testing

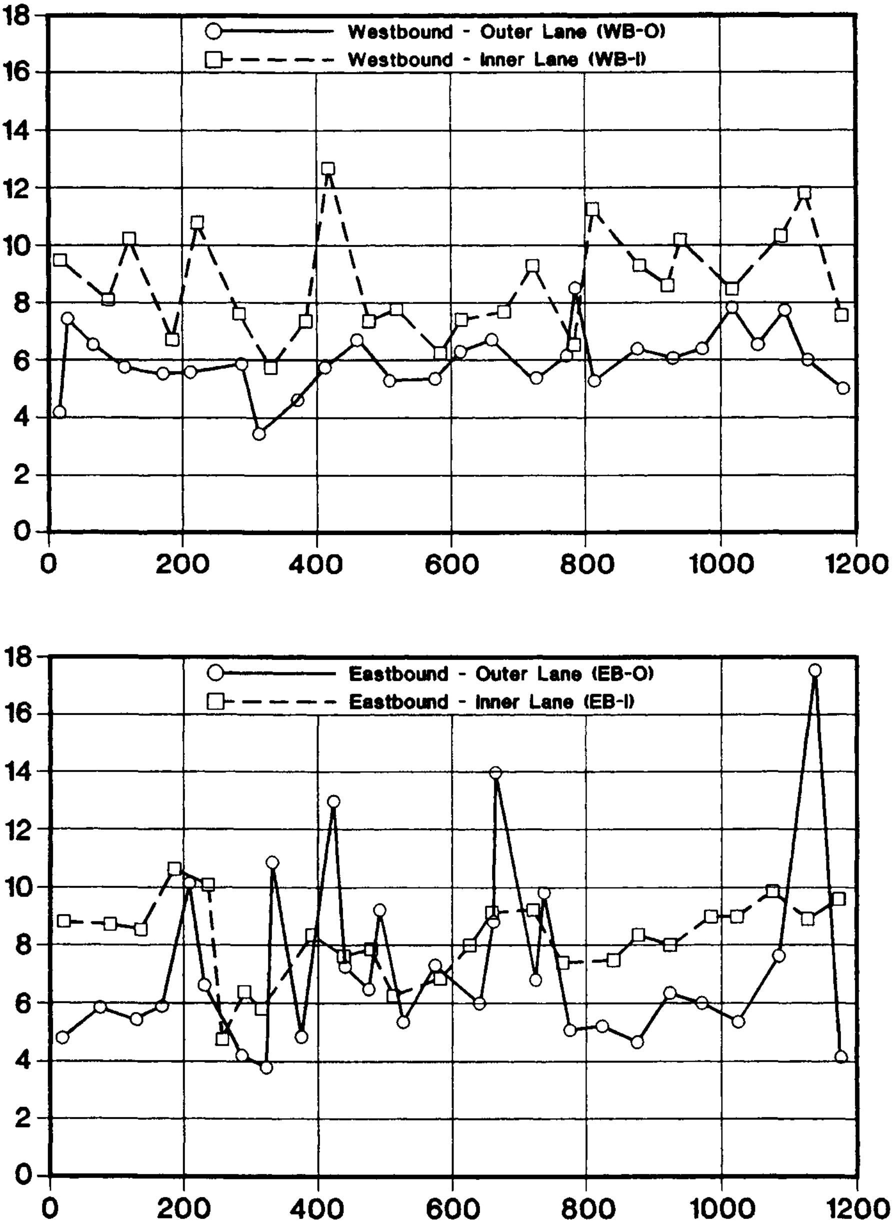

Figure 11-21. Maximum Deflection (D_0) Profile.

that the deterioration of the PCC base layer is much more severe in the inner lanes. Figure 11–24 shows a profile of the back-calculated subgrade moduli. In general, the inner lanes exhibit slightly higher subgrade moduli than the outer lanes. This may be due to the construction history of this particular project. Based on the results of the materials characterizations, the design layer material moduli were selected as shown in Figure 11–25.

ERI

CORE & BORING LOG

Project: City of Champaign #513
Pavement Feature: Asphalt/Concrete
Core #: 21EB-1
Core Location: 1040
Lane: Eastbound Inner Lane
Prepared By:
Date: 05/03/91

Sketch:

Depth (in inches)	Description	Comments
1–4	4" AC	Totally Unbonded with PCC Deteriorated on Edge
5–11	7" PCC	Severely Deteriorated PCC with Multiple Cracks
12	Clayey Subgrade	
1–12		
1–12		
1–4		

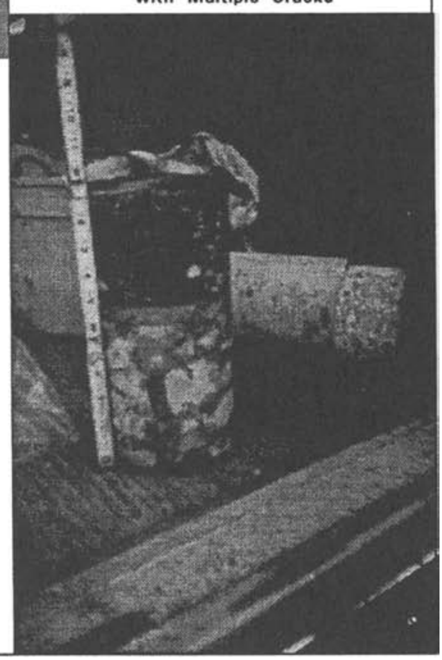

Figure 11-22. Example Core Log (ERI 1991).

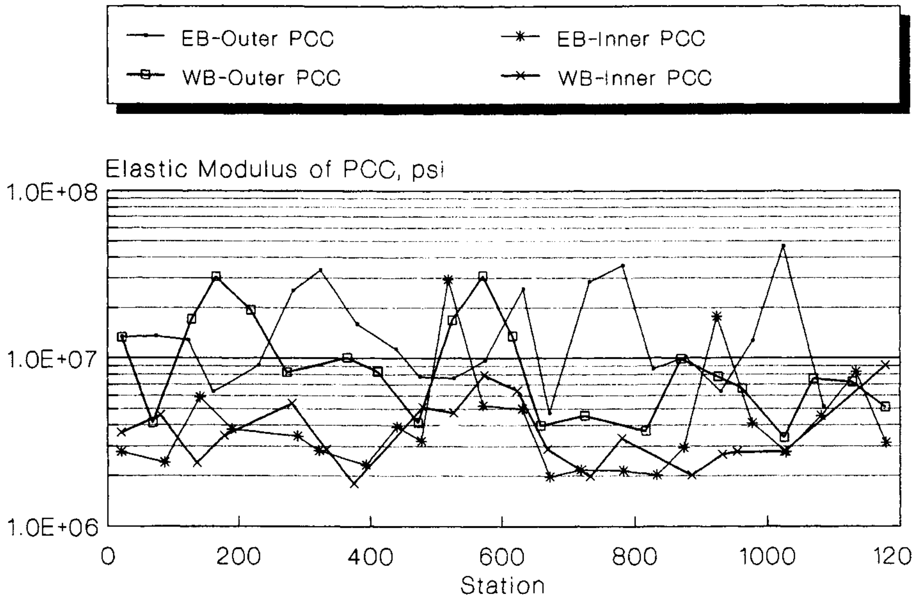

Figure 11-23. Backcalculated PCC Modulus Profile.

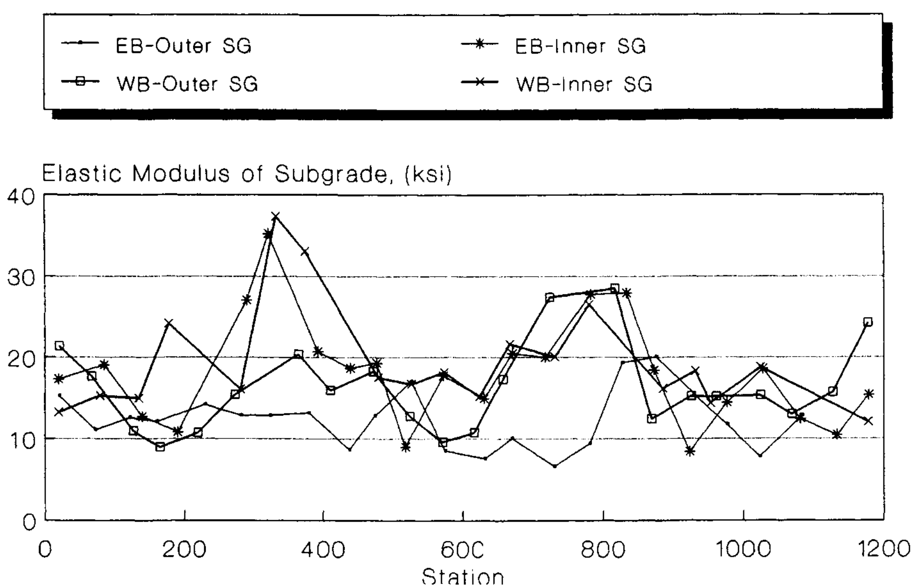

Figure 11-24. Backcalculated Subgrade Modulus Profile.

Table 11-25. Kirby Avenue, Champaign Illinois Design Material Properties, April, 1991.

Lane	E_{PCC} x 10^6 (psi)	E_{AC} (ksi)
Westbound--outer lane (WB-O)	6.20	12.8
Westbound--inner lane (WB-I)	1.10	15.8
Eastbound--outer lane (EB-O)	6.9	10.7
Eastbound--inner lane (EB-I)	1.60	14.6

11.4.3.2 Load Transfer Analysis.

The load transfer concept was presented in Chapter 4. Load transfer measurements were made across six selected surface cracks to represent a wide range of surface conditions. Measured deflection load transfer varied from a low of 40% to a high of 99%. The average value was 76%.

11.4.3.3 Utility Cut Patching Quality.

The quality of the utility cut patches present along the project was investigated by conducting deflection testing in and around three selected patches. The FWD load plate was positioned at the patch boundaries, both on the patch and on the surrounding pavement, and at the center of the patches. In all cases, the deflections obtained at these locations were significantly higher than away from the patches, indicating that the patches are localized zones of weaknesses along the project. Also, patch edge tests were approximately double the deflection at the patch center.

11.4.4 Pavement Evaluation and Selection of Rehabilitation Alternatives

Rehabilitation strategies must be designed to repair the existing pavement distress and prevent their future occurrence. Figure 11–26 shows an example evaluation summary sheet for the eastbound inner lane. A major concern is the deteriorated PCC base and the reflection cracking, which is likely to recur with an application of an overlay. The following major M&R alternatives were considered feasible:

1. Surface milling of the exiting asphalt surface layer followed by an overlay.
2. Surface milling of the existing asphalt surface layer, crack and seating of the existing PCC layer followed by an overlay.
3. Complete reconstruction of the pavements with a conventional flexible pavement design. Reconstruction of the pavement using PCC was deemed cost prohibitive and was therefore not considered.

The unit costs necessary to perform a complete economic analysis were obtained from recent bid tabs available from the city of Champaign and other sources. Figure 11–27 shows the estimated unit costs, in 1991 dollars, for each repair item.

EASTBOUND INNER LANE

1. **Overall Condition Rating - PCI** = 41

Rating -	Failed,	Very Poor,	Poor,	Fair,	Good,	Very Good,	Excellent
PCI	0–10	11–25	26–40	41–55	56–70	71–85	86–100

2. **Variation of Condition Within Section - PCI**
   a. Localized Random Variation — Yes, No
   b. Systematic Variation — Yes, No

3. **Rate of Detioration of Condition - PCI**
   a. Long-term Period, Since Construction or Last Overall Repair — Low, Normal, High
   b. Short-term Period, 1 Year — Low, Normal, High

4. **Distress Evaluation**
   a. Cause
      Load Associated Distress — 0 Percent Deduct Value
      Climate/Durability Associated — 86 Percent Deduct Value
      Other Associated Distress — 14 Percent Deduct Value
   b. Moisture, Drainage, Effect on Distress — Minor, Moderate, Major

5. **Deficiency of Load-Carrying Capacity** — No, Yes

6. **Surface Roughness** — Minor, Moderate, Major

7. **Skid Resistance/Hydroplaning Potential** — Minor, Moderate, Major

8. **Previous Maintenance** — Low, Normal, High

9. **Comments:** Considerable amount of medium and high severity joint reflection cracking. This is caused by a severely deterioriated PCC base.

Figure 11-26. Stepwise Procedure for Section Evaluation Summary.

The advantages and disadvantages of each rehabilitation alternative are discussed in the following sections. Initial construction costs for each alternative were developed based on expected repair quantities. Future maintenance activities were also developed for each repair alternative. Using all of these costs, the equivalent uniform annual cost (EUAC) for each alternative was calculated. This value is a good comparative measure for each alternative in that it takes into account all costs associated with each alternative. The alternative with the lowest EUAC is the most cost effective. For comparison, costs that are expected to be part of all alternatives (i.e., paint markings, sidewalk and driveway repairs, etc.) were excluded.

Figure 11-27. Estimated Unit Costs for 1991.

Item	Unit	Cost ($)[a]
Joint/crack cleaning and sealing	in ft	1.30
Saw cutting	in ft	3.50
Full-depth patching (AC)	sq yd	15.00
Full-depth patching (PCC)	sq yd	100.00
Portland cement concrete base course 8"	sq yd	23.51
Bituminous materials (tack coat)	sq yd	0.63
Aggregate base course (prime coat)	ton	15.00
Bituminous concrete binder course, mixture B. type 2	ton	43.60
Bituminous concrete surface course. mixture D. class I. type 2	ton	45.56
Bituminous concrete surface removal (milling)	sq yd	4.28
Paving fabric (including tack coat)	sq yd	2.56
Crack and seat (equipment mobilization)	lump sum	2,000.00
Crack and seat PCC slabs	sq yd	0.75
Pavement removal	sq yd	35.00

[a]These unit costs were obtained from City of Champaign bid tabs and other sources.

11.4.5 Analysis of Rehabilitation Alternatives

11.4.5.1 Surface Milling and Direct Overlay

This alternative involves the following items:

- Milling off the existing surface layer
- Full-depth patching of deteriorated joints and cracks in the PCC base slabs
- Placing an asphalt concrete overlay with a paving fabric interlayer

Advantages:

- The cold-milling process removes the asphalt without adding heat and will improve the bond between the PCC base slab and the overlay
- The initial cost of this alternative will be comparatively low

Disadvantages:

- Reflective cracking will be expected to continue at a rate at least as great at that experienced between 1980 and 1991. Adding a paving fabric may help retard this crack progression but will not completely eliminate its occurrence.

The required overlay thickness was designed using the Asphalt Institute (MS-1) method. In this method, a full-depth asphalt concrete thickness requirement is developed based on design subgrade properties and expected traffic. The existing PCC base slab is converted to an equivalent thickness of asphalt concrete based on its structural condition. The required overlay thickness is calculated as the difference between the required full-depth asphalt pavement design and the equivalent thickness of the existing PCC base slab.

For this analysis, design subgrade moduli of 11,400 psi and 8,800 psi were selected for the inner and outer lanes, respectively, based on the results of the NDT data analysis. As described in Section 11.4.2.2, design traffic estimates of 0.5 million and 1.9 million ESALs were used for the inner and outer lanes, respectively. Using these values, full-depth asphalt thickness designs of 7.0 in. and 10.0 in. were determined for the inner and outer lanes, respectively.

Based on the results of the materials characterizations, conversion factors for determining the effective thickness of the PCC base slabs were determined as 0.50 and 0.70 for the inner and outer lanes, respectively. Using average slab thicknesses of 7 in. and 8 in. for the inner and outer lanes, the following overlay requirements were determined.

Inner Lane: 7.0 - (0.5*7) = 3.50 in.
Outer Lane: 10.0 - (0.7*8) = 4.40 in.

Review of the 1966 construction drawings indicates the existing 7-in. slab has a surface elevation 1 in. below the 8-in. slab. Thus, the overlay requirement of the inner 22 feet would be increased to 5.4 in. to match the final surface elevation of the outer lanes. Coring records indicate the existing asphalt cover on the outer lanes averages 3.4 in. It is assumed that the difference in present surface elevations and required overlay thicknesses would not result in a requirement to reestablish new curb lines.

The project costs of this rehabilitation alternative are shown on Figure 11–28.

11.4.5.2 Crack and Seat of PCC Base in the Inner Lanes

This alternative involves the following items:

- Milling off the existing surface layer
- Crack and seat of the existing PCC (average 7 in. thick) in the inner lanes
- Full-depth patching of the existing PCC (average 8 in. thick) in the outer lanes
- Placing an asphalt concrete overlay

Advantages:

- The pavement will have a 20-year design life with an increased structural capacity.
- The crack and seat concrete layer will provide a more uniform support condition to the asphalt overlay, thus reducing significant maintenance for several years.
- All reflective distress from the existing surface will be minimized.

Figure 11-28. Cost Estimate for Alternative One. Surface Milling, Full-Depth Patching, and 4.5 in. Asphalt Overlay.

Activity (in 1991)	Quantity	Unit	Unit Price ($)	Total Amount ($)
Mill off existing AC surface	8.533	sq yd	4.28	36,521.00
Full-depth patching (PCC)	400	sq yd	100.00	40.000.00
Tack coat	8.533	sq yd	0.63	5,375.79
Paving fabric	8.533	sq yd	2.56	21,844.48
Binder coarse	1,550	ton	43.60	67,580.00
1.5" Overlay: surface course	672	ton	45.56	30.616.32
Maintenance Activity (Year)	**Quantity**	**Unit**	**Unit Price ($)**	**Total Amount ($)**
Crack sealing (1994)	3,072	in ft	1.30	3,993.60
Crack sealing (1997)	3.072	in ft	1.30	3,993.60
Crack sealing (2000)	3,072	in ft	1.30	3,993.60
Crack sealing (2003)	3,072	in ft	1.30	3,993.60
Crack sealing (2006)	3,072	in ft	1.30	3.993.60
Crack sealing (2009)	3,072	in ft	1.30	3,993.60
Full-depth AC patching (2001)	43	sq yd	15.00	645.00
Full-depth AC patching (2003)	86	sq yd	15.00	1.290.00
Full-depth AC patching (2005)	129	sq yd	15.00	1.935.00
Full-depth AC patching (2007)	172	sq yd	15.00	2,580.00
Full-depth AC patching (2009)	215	sq yd	15.00	3.225.00
			Total Initial Cost	201.937.59
			EUAC/SY	3.05

Disadvantages:

- A higher initial capital investment cost
- Longer closure times for construction

The required overlay thicknesses were again designed using the Asphalt Institute (1981, 1989) methods. The results are the same as for the previous alternative. In this method, a full-depth asphalt concrete thickness requirement is developed based on design subgrade properties and expected traffic. The existing PCC base slabs are converted to an equivalent thickness of asphalt concrete based on their structural condition. The required overlay thickness is calculated as the difference between the required full-depth asphalt pavement design and the equivalent thickness of the existing PCC base slab.

Inner Lane: 7.0 - (0.5*7) = 3.50 in.
Outer Lane: 10.0 - (0.7*8) = 4.40 in.

Review of the 1966 construction drawings indicates the existing 7-in. slab has a surface elevation 1 in. below the 8-in. slab. Thus, an additional 1 in. of aggregate should be placed before the overlay of the inner lanes to match the slab elevation of the outer lanes. Coring records indicate the existing asphalt cover on the outer lanes averages 3.4 in. It is assumed that the difference in present surface elevations and required overlay thicknesses would not result in a requirement to reestablish new curb lines.

The project costs of this rehabilitation alternative are shown in Figure 11–29.

11.4.5.3 Crack and Seat of Entire PCC Base

This alternative involves the following items:

- Milling off the existing surface layer
- Crack and seat of the existing PCC
- Placing an asphalt concrete overlay

Advantages:

- The pavement will have a 20-year design life with an increased structural capacity.
- The crack and seat concrete layer will provide a more uniform support condition to the asphalt overlay, thus reducing significant maintenance for several years.
- All existing surface distresses will be eliminated.

Disadvantages:

- A higher initial capital investment cost
- Longer closure times for construction

The crack and seat PCC slabs were assigned conversion factors of 0.5 and 0.6 for the inner and outer lanes, respectively, for determining their effective thickness. Using the average crack and seat slab thicknesses for the inner and outer lanes, the following overlay requirements were determined:

Inner Lane: 7.0 - (0.5*7.0) = 3.5 in.
Outer Lane: 10.0 - (0.6*8.0) = 5.2 in.

The project costs of this rehabilitation alternative are shown in Figure 11–30.

Figure 11-29. Cost Estimate for Alternative Two. Surface Milling, Full-Depth Patching (Outer Lane), Crack and Seat (Inner Lane) and 4.5 in. Asphalt Overlay.

Activity (in 1991)	Quantity	Unit	Unit Price ($)	Total Amount ($)
Mill off existing AC surface	8,533	sq yd	4.28	36,521.00
Full-depth patching (PCC)	234	sq yd	100.00	23,400.00
Full-depth saw cut	3,200	lf	3.5	11,200.00
Crack and seat equipment mobilization	1	ls	2,000.00	2,000.00
Crack and seat	3,911	sq yd	0.75	2,933.25
Aggregate base course	176	ton	15.00	2,640.00
Tack coat	8,533	sq yd	0.63	5,375.79
Paving fabric	4,267	sq yd	2.56	10,923.52
3" Overlay: binder course	1,344	ton	43.60	58,598.40
1.5" Overlay: surface course	672	ton	45.56	30,616.32
Maintenance Activity (Year)	**Quantity**	**Unit**	**Unit Price ($)**	**Total Amount ($)**
Crack sealing (1994)	2,016	in ft	1.30	2,620.80
Crack sealing (1997)	2,016	in ft	1.30	2,620.80
Crack sealing (2000)	2,016	in ft	1.30	2,620.80
Crack sealing (2003)	2,016	in ft	1.30	2,620.80
Crack sealing (2006)	2,016	in ft	1.30	2,620.80
Crack sealing (2009)	2,016	in ft	1.30	2,620.80
Full-depth AC patching (2001)	23	sq yd	15.00	345.00
Full-depth AC patching (2003)	47	sq yd	15.00	705.00
Full-depth AC patching (2005)	70	sq yd	15.00	1,050.00
Full-depth AC patching (2007)	93	sq yd	15.00	1,395.00
Full-depth AC patching (2009)	116	sq yd	15.00	1,740.00
			Total Initial Cost	184,208.52
			EUAC/SY	2.71

Figure 11-30. Cost Estimate for Alternative Three, Surface Milling, Crack and Seat (All Lanes) and 5.25 in. Asphalt Overlay.

Activity (in 1991)	Quantity	Unit	Unit Price ($)	Total Amount ($)
Mill off existing AC surface	8,533	sq yd	4.28	36,521.00
Full-depth saw cut	3,200	lf	3.5	11,200.00
Crack and seat equipment mobilization	1	ls	2,000.00	2,000.00
Crack and seat	8,533	sq yd	0.75	6,399.75
Aggregate base course	176	ton	15.00	2,640.00
Tack coat	8,533	sq yd	0.63	5,375.79
3.75" Overlay: binder course	1,681	ton	43.60	73,291.60
1.5" Overlay: surface course	672	ton	45.56	30,616.32
Maintenance Activity (Year)	**Quantity**	**Unit**	**Unit Price ($)**	**Total Amount ($)**
Crack sealing (1994)	768	in ft	1.30	998.40
Crack sealing (1997)	768	in ft	1.30	998.40
Crack sealing (2000)	768	in ft	1.30	998.40
Crack sealing (2003)	768	in ft	1.30	998.40
Crack sealing (2006)	768	in ft	1.30	998.40
Crack sealing (2009)	768	in ft	1.30	998.40
			Total Initial Cost	168,044.70
			EUAC/SY	2.37

11.4.5.4 Pavement Reconstruction

This alternative involves the following items:

- Complete removal of the existing pavements
- Placement of a conventional asphalt concrete pavement

Advantages:

- The pavement will have a 20-year design life with an increased structural capacity.
- All existing surface distresses will eliminated.

Disadvantages:

- A higher initial capital investment cost
- Longer closure times for construction

Pavement thickness requirements were developed using mechanistic principles. In this analysis, the new pavement is modeled as a three-layered system with variable thickness of asphalt concrete over an 8-in. aggregate base. The material properties of the pavement layers were varied by season to account for temperature and moisture effects on material properties. Critical load-induced strains in the pavement system, at the bottom of the asphalt layer and at the top of the subgrade, are used to determine allowable load repetitions using fatigue models. Using this analysis technique, the following thickness designs were determined for an 8-in. aggregate base:

Inner Lane: 3.5 in. asphalt concrete surface
Outer Lane: 4.5 in. asphalt concrete surface

The project costs of this rehabilitation alternative are shown in Figure 11–31.

11.4.6 Recommended M&R Alternative

Based on the analysis presented in the previous sections and discussion with city engineers, alternative 3 was recommended. It consists of milling the existing AC surface, crack and seating the underlying PCC pavement, and placing a 5.25-in. AC overlay. This alternative was selected due to its low equivalent uniform annual cost. It was also recommended to let some or all of the other three alternatives be submitted for bids to ensure accurate cost analysis.

Table 11-31. Cost Estimate for Alternative Four. Reconstruction with 8.0 in. Aggregate and 4.5 in. Asphalt Overlay.

Activity (in 1991)	Quantity	Unit	Unit Price ($)	Total Amount ($)
Remove existing pavement	8.533	sq yd	35.00	298,655.00
Place 8" aggregate base	3.100	ton	15.00	46,500.00
3.0" Overlay: binder course	1,344	ton	43.60	58,598.40
1.5" Overlay: surface course	672	ton	45.56	30,616.32
Maintenance Activity (Year)	**Quantity**	**Unit**	**Unit Price ($)**	**Total Amount ($)**
Crack sealing (1994)	768	in ft	1.30	998.40
Crack sealing (1997)	768	in ft	1.30	998.40
Crack sealing (2000)	768	in ft	1.30	998.40
Crack sealing (2003)	768	in ft	1.30	998.40
Crack sealing (2006)	768	in ft	1.30	998.40
Crack sealing (2009)	768	in ft	1.30	998.40
			Total Initial Cost	434,369.72
			EUAC/SY	6.03

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12

Special Application – Impact of Bus Traffic on Pavement Costs

This chapter is based on a study conducted by the author for the City of Los Angeles (Shahin and Crovetti 1999)

City engineers routinely make important decisions regarding the type, location, and frequency of maintenance and repair (M&R) activities. M&R requirements for traffic lanes carrying city bus traffic are frequently greater than those for similar lanes without bus traffic. To properly apportion these increased maintenance costs, city engineers must have an objective method for quantifying the impact of Mass Transit Authority (MTA) bus traffic. This chapter presents a variety of evaluation techniques that may be used to quantify the effect of buses in terms of increased deterioration rates and rehabilitation costs. These techniques are illustrated using pavement data collected from the City of Los Angeles.

12.1 Data Collection Procedure

Ten full-depth hot-mix asphalt (HMA) pavement sections – each section approximately one city block – subjected to bus traffic were selected at random from the City's roadway network. Selected sections ranged from 200 to 600 feet in length. A sample lane and a control lane were identified within each section. The sample lane represents the outer lane that carries bus traffic; the control lane represents the adjacent inner lane (for roadways with two lanes in each direction) or middle lane (for roadways with three lanes in each direction) not subjected to MTA bus traffic. Two pavement cores were taken from each pavement section to determine the HMA thickness. Traffic data was obtained from City engineers, including total two-way average daily traffic, two-way truck traffic, two-way bus passes, and truck distributions by lane. Figure 12–1 provides a summary of data collected for the ten pavement sections.

Pavement performance data, including surface distress and pavement deflections, were collected from the sample and control lanes of each section. Pavement distress data were collected from each lane following the Pavement Condition Index (PCI) survey procedure (Chapter 3). The PCI is an indicator of a pavement section's structural integrity and surface operational condition on a scale of 0 to 100 – with 100 representing a new, distress-free pavement in excellent condition. The PCI has unique qualities that make it a useful visual surveying tool because it agrees closely with the collective judgment of experienced pavement engineers and is repeatable. One representative sample unit, 200 feet long by one lane width, was selected from each of the sample and control lanes. Pavement distress data was collected and used to calculate the representative PCI for each lane.

Surface deflections were collected within each traffic lane using a falling-weight deflectometer (FWD), which produces surface loads similar in magnitude and duration to heavy wheel loadings. Test loads were applied over an 11.81-inch diameter loading plate; resulting surface deflections were recorded by geophones positioned at 0, 8, 12, 18, 24, 36, and 60 inches from the center of the loading. FWD testing was conducted at approximately 50-foot intervals in the inner wheel path of each test lane. Test loads of 9,000 lb. were used at each location.

12.2 Pavement Analysis Techniques

Pavement distress, deflection, and traffic data were used to evaluate the structural condition of the existing pavement and to estimate the remaining life and determine overlay thickness requirements. Remaining life calculations and overlay thickness requirements were determined using three different approaches: (1) a procedure based on American Association of State Highway and Transportation Officials (AASHTO) design guidelines; (2) a mechanistic-based (e.g., fatigue) analysis; and (3) a PCI-based rate of deterioration analysis.

12.2.1 AASHTO Analysis/Design Approach

Following AASHTO guidelines (AASHTO 1993), a comprehensive structural analysis was conducted to assess the impact of City bus traffic on pavement performance for the selected pavement test sections. Pavement response algorithms were developed to estimate key pavement parameters from surface deflections produced by the FWD. The algorithms were then used to determine the additional HMA surface thickness required due to the MTA bus traffic.

Development of Pavement Response Algorithms

A factorial analysis of full-depth HMA pavement cross sections was conducted using the nonlinear elastic computer program KENLAYER (Huang 1993). The HMA layer was modeled as a linear elastic layer with thickness ranging from 10 to 16 inches with elastic moduli ranging from 250 to 1400 ksi. Subgrade soils ranging from very soft to stiff were modeled as nonlinear stress-softening materials. Input values for the breakpoint resilient modulus (E_{ri}), minimum resilient modulus, and maximum resilient modulus for each soil type are provided in Figure 12–2. During all program runs, the HMA layer was assumed to be unbonded from the subgrade layer.

The FWD plate load was modeled as an 11.81-inch circular load with a uniform contact pressure of 82.145 psi. Surface deflections at 0, 12, 24, and 36 inches from the center of loading and maximum tensile strains at the bottom of the HMA layer were calculated for each pavement system. The structural number (SN) for each model full-depth HMA pavement cross section was computed using the following AASHTO equation (AASHTO 1993):

$$SN = (a_1)(h_1) \tag{12-1}$$

where:

a_1 = structural coefficient of the HMA

h_1 = HMA layer thickness, inches

The structural coefficient of the HMA layer was determined based on the assigned elastic modulus using the following equation (Van Til et al. 1972):

$$a_1 = (0.3913)\,\mathrm{Log}\,E_{ac} - 0.601 \tag{12-2}$$

where:

a_1 = structural coefficient of the HMA

E_{ac} = HMA modulus, ksi

Surface deflections generated by KENLAYER were used to compute the area under the pavement profile (AUPP) using the following equation (Thompson 1999):

$$AUPP = (5D_0 - 2D_{12} - 2D_{24} - D_{36})/2 \tag{12-3}$$

where:

D_i = surface deflection at 'i' inches from the load center, mils

A regression analysis was conducted to develop algorithms for estimating pavement layer and system parameters from surface deflections. The following algorithms were developed as part of this study:

$$\mathrm{Log}\,E_{ri} = 2.505 - 2.03\,\mathrm{Log}\,D_0 - 1.52\,\mathrm{Log}\,D_{36} + 1.45\,\mathrm{Log}\,AUPP$$
$$R^2 = 0.9476 \tag{12-4}$$

$$\mathrm{Log}\,E_{sg} = 1.496 + 0.606\,\mathrm{Log}\,D_0 - 1.56\,\mathrm{Log}\,D_{36}$$
$$R^2 = 0.9606 \tag{12-5}$$

$$\mathrm{Log}\,E_{ac}T^3 = 7.132 - 1.31\,\mathrm{Log}\,AUPP + 0.069\,\mathrm{Log}\,D_0$$
$$R^2 = 0.9199 \tag{12-6}$$

$$SN = 0.0566\,(E_{ac}T^3)^{1/3}$$
$$R^2 = 0.9290 \tag{12-7}$$

$$\mathrm{Log}\,\varepsilon_{ac} = 1.075 + 0.977\,\mathrm{Log}\,AUPP$$
$$R^2 = 0.9836 \tag{12-8}$$

where:

E_{sg} = subgrade elastic modulus, ksi

T = HMA thickness, inch

$E_{ac}T^3$ = surface flexural rigidity, kip-inch

ε_{ac} = maximum tensile strain at bottom of asphalt layer, micro units

Full-Depth HMA Thickness Requirements

Following current AASHTO design procedures (AASHTO 1993), the required thickness for a new full-depth HMA pavement is dependent on the level of design reliability, the loss of serviceability due to traffic, the subgrade resilient modulus, and the projected volume of heavy axle loadings. The required SN for a new pavement can be determined using the following AASHTO equation (AASHTO 1993):

$$Log(W_{18}) = (Z_R)(S_O) + (9.36)Log(SN+1) - 0.20 + Log\frac{\left[\frac{\Delta PSI}{(4.2-1.5)}\right]}{\left\{0.40+\left[\frac{1094}{(SN+1)^{5.19}}\right]\right\}} + (2.32)Log(M_R) - 8.07 \qquad (12\text{-}9)$$

where:

W_{18} = total 18-kip equivalent single axle loadings (ESALs)

Z_R = standard normal deviate for design level of reliability

S_O = overall standard deviation

p_i = initial pavement serviceability

p_t = pavement serviceability at end of time 't'

$\Delta PSI = (p_i - p_t)$

M_R = effective roadbed soil resilient modulus

For this analysis, the following design variables were used for each test section: design reliability level = 90% yields Z_R = -1.282; S_O = 0.45; p_i = 4.2; and p_t = 2.5.

For each pavement section, subgrade E_{ri} values were back-calculated at every FWD test location using the algorithms presented above. These values were averaged for each test lane and used as input values for the effective roadbed soil resilient modulus, M_R. The daily 18-kip ESALs for the control and sample lanes of each test section were computed based on the traffic projections provided by City engineers. Based on information provided by the City, average ESAL values of 1.5 and 3.222 were used for each truck and City bus, respectively. Twenty-year total ESAL values were computed for each lane based on truck traffic only, as well as for the combined truck and bus traffic in the sample lanes. The required SN for each lane was then computed using the AASHTO equation. These values were converted to a required HMA thickness using the equation:

$$T_{HMA} = SN / 0.44 \qquad (12\text{-}10)$$

where:

T_{HMA} = required full-depth HMA thickness, in

0.44 = standard layer coefficient for HMA

The additional HMA thickness required due to City bus traffic was computed as the difference between the required HMA thickness in the sample lane due to combined truck and bus traffic and the maximum required HMA thickness in the sample lane due to truck traffic only. Figure 12–3 provides the results of this analysis for each pavement test section. As shown in Figure 12–3, the additional HMA thickness requirements due to City bus traffic range from 0.61 to 2.41 inches, with an average of 1.2 inches.

12.2.2 Mechanistic Analysis/Design Approach

A mechanistic pavement analysis was conducted using collected deflection, coring, and traffic data. The primary focus of this analysis was to determine the additional HMA surface thickness required due to the MTA bus traffic. This was achieved by first determining the fatigue life based on existing bus and truck traffic in the bus lane. The required HMA thickness to achieve the same fatigue life without buses was then determined. The additional HMA thickness was then determined as the difference between existing HMA thickness and required HMA thickness without buses to achieve the same fatigue life. The process is presented in detail for Beverly Boulevard.

Example Mechanistic Analysis

Deflection data collected along Beverly Boulevard was analyzed using the full-depth algorithms previously presented. Initially, the effective SN and critical asphalt tensile strain were back-calculated for each test location. Figure 12–4 provides these results. Pavement cores were obtained from both the sample and control lanes. The core holes were taken at stations where FWD testing was conducted. These stations were designated by "A" for the control lane and "B" for the sample lane. Figure 12–5 provides a summary of pertinent information determined during the coring program for Beverly Boulevard.

Using the deflection and coring data, the elastic modulus of the HMA layer and the subgrade breakpoint resilient modulus were back-calculated for each coring station using the equations presented earlier. Figure 12–6 provides the results of this analysis.

Using traffic data provided by City engineers, representative axle loadings were developed for the City buses and typical trucks using the bus lane. Figure 12–7 provides a summary of these data. Using the data provided in Figure 12–4 and 12–5, the KENLAYER computer program was used to calculate critical HMA tensile strains and subgrade compressive strains for each axle loading within the bus lane. These strain values were then used to estimate the strain repetitions to fatigue failure of the sample lane based on HMA fatigue cracking and subgrade rutting criteria. The following fatigue models used for this analysis were developed by the Asphalt Institute (MS-1 & Research Report 82-2):

HMA Fatigue Cracking: $N_f = (0.0796)\, \varepsilon_{ac}^{-3.291}\, E_{ac}^{-0.854}$ (12-11)

Subgrade Fatigue Rutting: $N_f = (1.365 \times 10^{-9})\, \varepsilon_{sg}^{-4.477}$ (12-12) where:

N_f = strain repetitions to fatigue failure

ε_{ac} = critical HMA tensile strain (micro units)

E_{ac} = HMA layer modulus (psi)

ε_{sg} = critical subgrade compressive strain (micro units)

Combined Traffic Analysis

The yearly fatigue damage induced by each axle loading was determined as the ratio of the yearly-applied axle loadings to the calculated strain repetitions to failure (N_{app}/N_f). The cumulative yearly fatigue damage is calculated as the summation of damage values for each axle loading. The fatigue life of the pavement is calculated as the inverse of the cumulative yearly fatigue damage values. The design fatigue life of the pavement is considered as the minimum fatigue life calculated for each core location. The results of the fatigue analysis are provided in Figure 12–8. As shown, the design fatigue life of Beverly Boulevard is controlled by HMA fatigue cracking at station B2.

Traffic Analysis Excluding MTA Bus Traffic

A second mechanistic fatigue analysis was conducted considering only the axle loadings induced by the truck traffic. The objective of this analysis was to determine the minimum HMA thickness that would yield a fatigue life equal to that calculated during the combined traffic analysis. For each trial HMA layer thickness, critical HMA tensile strains, fatigue repetitions to failure, yearly damages, and calculated fatigue life were determined as before. Figure 12–9 summarizes the results for each core location along the sample lane of Beverly Boulevard.

Impacts of MTA Bus Traffic

Using the results of the mechanistic fatigue analyses discussed previously, the impacts of City bus traffic are represented as the additional HMA thickness required to provide equal pavement fatigue life. Figure 12–10 summarizes the results of this impact analysis. As shown, the additional HMA thicknesses range from 0.5 to 2.8 inches, with an average of 1.565 inches.

12.2.3 PCI Analysis/Design Approach

The results of the PCI survey are shown in Figure 12–11 for the bus and control lanes. Figure 12–12 shows the average rate of deterioration for both the bus and control lanes. The average rate of deterioration for the bus lane was determined to be 3.2 PCI points per year compared to 1.6 PCI points per year for the control lane. This may be interpreted to show that the life of the bus lane is half that of the control lane. For example, if a PCI of 60 is selected to represent the life of the pavement at which major rehabilitation is required, it will take the control lane 25 years to drop from a PCI of 100 to 60, whereas it will take the bus lane only 12.5 years.

The difference between the deterioration rates of the two lanes cannot be completely attributed to buses since the bus lane also receives heavier truck traffic than the control lane. Therefore, it is more appropriate to contribute the difference in performance to the difference in the ESAL of the combined bus and truck traffic. The average ESAL of trucks and buses in the bus lane for all sections was 513 per day as compared to 157 per day for the control lane. The difference in life of 12.5 years, as shown in the previous example, can be represented as a loss in life per ESAL:

(12.5 Years) / (513 - 157 ESALs) = 0.035 years/ESAL

Since the buses alone produce 283 ESALs, the loss of life due to the buses is:

(0.035 years/ESAL) (283 ESALs) = 9.9 Years ≅ 10 years

12.3 Bus Impact on Pavement Life Cycle Costing

Three methods were used to quantify the effect of City buses in terms of increased rate of deterioration and rehabilitation cost: AASHTO, mechanistic, and PCI. The results of the analyses are presented in the following paragraphs.

12.3.1 AASHTO Approach

Average additional thickness due to bus traffic = 1.195 in. ≅ 1.2 in.

Cost per lane mile = {[(1.2 in.) / (12 in./ft.)] (12.5 ft.) (5280 ft./mile)} (0.075 tons/cu. ft.) ($32/ton) = $15,840/lane mile

Assumed total service life = 20 years

Cost per lane mile per year = ($15,840/lane mile) / (20 years) = $792/lane mile/year

12.3.2 Mechanistic Approach

Average additional thickness due to bus traffic = 1.57 in. ≅ 1.5 in.

Cost per lane mile = {[(1.5 in.) / (12 in./ft.)] (12.5 ft.) (5280 ft./mile)} (0.075 tons/cu. ft.) ($32/ton) = $19,800/lane mile

Assumed total service life = 25 years

Cost per lane mile per year = ($19,800/lane mile) / (25 years) = $792/lane mile/year

12.3.3 PCI Approach

Average loss in life = 10 years

Cost to overlay one lane mile based on historical records ≅ $50,000

Assumed total service life = 25 years

Cost to overlay one lane mile/year = ($50,000) / (25 years) = $2,000/year

Extra cost due to buses = ($2,000) (10 years) / (25 years) = $800/year

12.4 Conclusions

A detailed study was conducted on 10 randomly selected asphalt pavement sections subjected to bus traffic for the City of Los Angeles. Pavement distress, deflection, coring, and traffic data were collected and analyzed using three approaches: AASHTO, mechanistic fatigue, and PCI rate of deterioration. All three approaches show that the extra cost associated with City buses is approximately $800/lane mile/year. This cost is conservative since it does not include the extra cost of raising curb and gutter or maintenance hole adjustments.

Roadway	Test Station	Existing HMA Thickness (in.)	Yearly MTA Bus Passes	Yearly Truck Passes
Beverly Blvd.	B1	12.7	46,538	54,020
	B2	11.2		
Cypress Ave.	B3	10.0	33,398	56,356
	B4	10.7		
Fairfax Dr.	B3	7.8	33,580	94,024
	B8	7.3		
Figueroa St.	B6	11.8	32,850	28,434
	B7	7.0		
Hoover St.	B1	10.0	41,245	97,382
	B2	8.2		
Melrose Ave.	B3	11.0	36,318	141,620
	B4	10.5		
Rodeo Rd.	B4	11.1	33,215	15,768
	B6	10.7		
Washington Blvd.	B3	9.6	34,310	24,346
	B6	13.0		
Whiteoak Ave.	B2	13.1		
	B5	9.1	8,213	36,938
	B6	10.0		
3rd St.	B1	7.8	21,353	11,370
	B7	8.3		

Figure 12-1. Pavement and Traffic Data.

Subgrade Type	Eri (ksi)	Minimum Resilient Modulus (ksi)	Maximum Resilient Modulus (ksi)
Very Soft	1.00	1.00	5.66
Soft	3.02	1.83	7.68
Medium	7.68	4.72	12.34
Stiff	12.34	7.61	17.00

Figure 12-2. Input Values for Subgrade Soil Types.

Test Section	Average Eri (ksi)		Daily ESAL Values			Required SN			Additional HMA Thickness Due to Buses (in.)
	SL[1]	CL[2]	SL, T[3] Only	SL, T + B[4]	CL, T Only	SL, T Only	SL, T + B	CL, T Only	
Beverly Blvd.	11.2	18.7	222	633	56	3.23	3.83	2.11	1.36
Cypress Ave.	24.2	15.3	232	526	58	2.42	2.77	2.29	0.80
Fairfax Ave.	5.3	6.2	386	683	97	4.64	5.04	3.53	0.91
Figeuroa St.	2.3	10.3	117	407	351	5.16	6.12	3.60	2.18
Hoover St.	35.5	33.4	400	764	100	2.29	2.55	1.85	0.59
Melrose Ave.	12.3	17.3	582	903	146	3.65	3.92	2.55	0.61
Rodeo St.	5.8	6.1	65	358	194	3.39	4.45	3.97	2.41
Washington Blvd.	26.2	33.0	100	403	300	2.04	2.57	2.24	1.20
Whiteoak Ave.	4.5	4.8	152	224	38	4.27	4.53	3.33	0.59
3rd St.	26.1	16.4	47	235	234	1.79	2.36	2.82	1.30
Average Additional Thickness Required Due to MTA Bus Traffic 1.20									

1. Sample Lane, SL; 2. Control Lane, CL; 3. Truck, T; 4. Bus, B

Figure 12-3. Full-Depth HMA Additional Thickness Requirements.

Test Station	Effective SN		Critical HMA Tensile Strain (microunits)	
	SL[2]	CL[3]	SL	CL
1	5.2	8.4	111	37
2	4.2	6.2	184	76
3	8.6	8.9	36	32
4	7.6	9.3	46	30
5	7.3	8.8	52	33
6	8.7	9.0	34	32
7	9.1	9.5	31	28

1. Values determined from deflections using Full-Depth HMA algorithms
2. Sample Lane, SL
3. Control Lane, CL

Figure 12-4 Effective SN and Critical HMA Tensile Strain[1] for Beverly Boulevard

Corehole Location	HMA Thickness (in.)	Subgrade Class[1]	Liquid Limit	Plasticity Index	Moisture Content
B1	12.7	SM	29	6	15.1
B2	11.2	SM	N/P	0	14.5
A1	14.0	CL	42	21	20.2
A2	11.7	SC	29	9	16.4

1. Based on Unified Soil Classification System

Figure 12-5 Results of Coring Program for Beverly Boulevard

Corehole Location	HMA Layer Elastic Modulus (ksi)	Subgrade Breakpoint Resilient Modulus (ksi)
B1	386	11.3
B2	298	5.1
A1	1181	22.2
A2	805	24.6

Figure 12-6. Back-Calculated Layer Properties for Beverly Boulevard.

Vehicle Type	Axle Location	Axle Configuration	Axle Load (lbs.)	Yearly Loadings
MTA Bus	Front	Single: 2 tires	13,500	46,538
	Rear	Single: 4 tires	23,500	46,538
Typical Truck	Front	Single: 2 tires	12,000	54,020
	Rear	Single: 4 tires	19,300	54,020

Figure 12-7. Summary of Axle Loadings.

Axle		MTA Bus		Typical Truck	
		Front	Rear	Front	Rear
B1	Nf_{ac}	3.2E07	1.4E07	4.5E07	2.5E07
	Nf_{sg}	8.5E10	3.3E10	1.4E11	7.6E10
B2	Nf_{ac}	7.3E06	3.4E06	1.0E07	6.3E06
	Nf_{sg}	6.7E09	2.8E09	1.1E10	6.3E09
	N_{app}	46,538	46,538	54,020	54,020
B1	D_{ac}	4.8E-03		3.3E-03	
	D_{sg}	1.9E-06		1.1E-06	
B2	D_{ac}	2.0E-02		1.4E-02	
	D_{sg}	2.4E-05		1.4E-05	
B1	Life	(ac)	123		
	Life	(sg)	Unlimited		
B2	Life	(ac)	30		
	Life	(sg)	Unlimited		
Pavement Design Life 30 years					

Figure 12-8. Mechanistic Fatigue Analyses Results—Combined Traffic for Beverly Boulevard.

Test Station	HMA Thickness (in.)	Fatigue Life (ac)	Fatigue Life (sg)
B1	10.7	124	Unlimited
B2	9.4	30	Unlimited

Figure 12-9. Mechanistic Fatigue Analysis Results—Trucks Only for Beverly Boulevard.

Roadway	Test Station	HMA Thickness Combined Traffic (in.)	HMA Thickness Trucks Only (in.)	Additional HMA Thickness Due to Buses (in.)
Beverly Blvd.	B1	12.7	10.7	2.0
	B2	11.2	9.4	1.8
		Average Additional HMA Thickness =		1.90
Cypress Ave.	B3	10.8	8.8	2.0
	B4	10.7	9.4	2.3
		Average Additional HMA Thickness =		2.15
Fairfax Dr.	B3	7.8	7.2	0.6
	B8	7.3	6.7	0.6
		Average Additional HMA Thickness =		0.60
Figueroa St.	B6	11.8	9.7	2.1
	B7	7.0	5.6	1.4
		Average Additional HMA Thickness =		1.75
Hoover St.	B1	10.0	9.0	1.0
	B2	8.2	7.4	0.8
		Average Additional HMA Thickness =		0.90
Melrose Ave.	B3	11.0	10.3	0.7
	B4	10.5	9.8	0.7
		Average Additional HMA Thickness =		0.70
Rodeo Rd.	B4	11.1	8.4	2.7
	B6	10.7	8.0	2.7
		Average Additional HMA Thickness =		2.70
Washington Blvd.	B3	9.6	7.6	2.0
	B6	13.0	10.2	2.8
		Average Additional HMA Thickness =		2.40
Whiteoak Ave.	B2	13.1	12.4	0.7
	B5	9.1	8.6	0.5
	B6	10.0	9.4	0.6
		Average Additional HMA Thickness =		0.60
3rd St.	B1	7.8	5.9	1.9
	B7	8.3	6.3	2.0
		Average Additional HMA Thickness =		1.95
Overall Average Additional HMA Thickness = 1.565				

Figure 12-10. Additional Pavement Thickness Required Due to MTA Busses.

Test Section	LCD[1]	Surface Type	Rank	True Area (ft^2)	LID[2]	AI[3]	S1[4] PCI, Bus Lane	S2 PCI, Control Lane
Beverly Blvd.	01/1979	AC[5]	P[6]	13,422.50	01/1999	20	37	96
Cypress Ave.	01/1984	AC	P	13,600.00	01/1999	15	63	92
Fairfax Ave.	10/1992	AC	P	39,680.00	02/1999	7	71	91
Figeuroa St.	10/1994	AC	P	11,220.00	01/1999	5	84	90
Hoover St.	01/1988	AC	P	6,600.00	01/1999	11	85	94
Melrose Ave.	01/1984	AC	P	8,265.00	01/1999	15	34	35
Rodeo St.	09/1997	AC	P	48,000.00	02/1999	2	100	100
Washington Blvd.	11/1987	AC	P	12,250.00	01/1999	12	56	65
Whiteoak Ave.	01/1987	AC	P	97,680.00	06/1999	12	45	67
3rd St.	08/1992	AC	P	31,110.00	02/1999	7	98	100

9. *Last Construction Date, LCD*
10. *Last Inspection Date, LID*
11. *Age at Inspection, AI*
12. *Section 1, S1*
13. *Asphalt concrete, AC*

Figure 12-11. PCI Data.

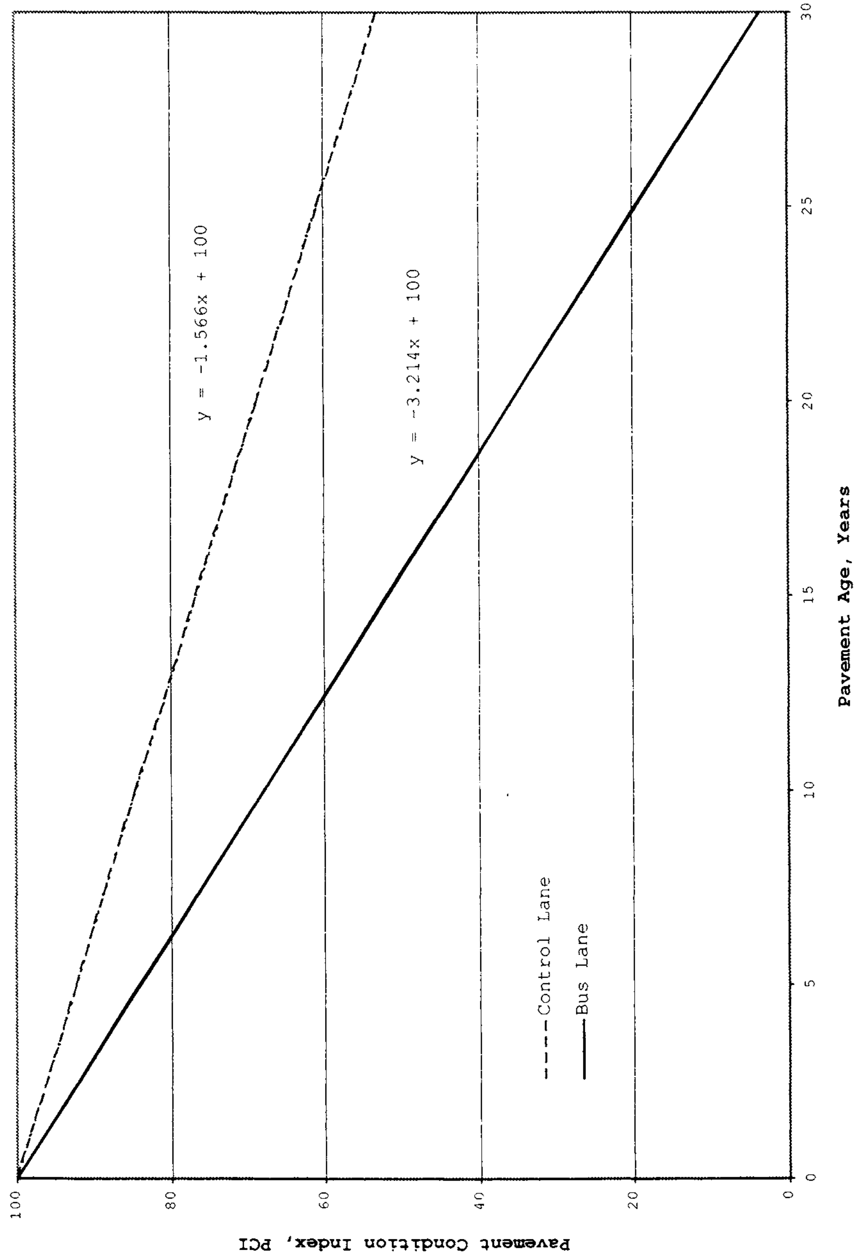

Figure 12-12. Pavement Deterioration Rates.

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13

Special Application - Impact of Utility Cuts on Pavement Life and Rehabilitation Cost

This chapter presents the summary of several studies that were conducted to quantify the effect of utility cut patching on pavement performance and increased Maintenance and Rehabilitation (M&R) cost. For the studies presented in this chapter, the author was the Principal Investigator on the studies for Prince George's County (Maryland), Los Angeles (California), and Burlington (Vermont). The author was a member of a panel for the San Francisco (California) study and served as a reviewer for the Sacramento study (California). Each of the studies is summarized separately. The last part of the chapter presents an overall summary and conclusions from all the studies.

13.1 Prince George's County, MD (Shahin and Crovetti 2002)

13.1.1 Pavement Testing Program

A total of 30 pavement sections were selected and surveyed. Two adjacent inspection units (2500SF + 1000 sf) were selected from each section where one of the units had utility cut patches while the adjacent unit did not (Figure 13-1).

The surface condition was quantified using the Pavement Condition Index (PCI) method. The structural adequacy of the patched and non-patched pavement was evaluated using a Falling Weight Deflectometer (FWD). Pavement deflections were measured inside and outside the patches.

The selection of sections was based on the following criteria:

- The majority of pavements in Prince George's County are asphaltic concrete (AC). Therefore, all selected sections should be AC.
- The total number of sections should be about 30 to provide enough data points for a statistical comparison of the results.
- Sections should cover a wide range of conditions and ages to allow for meaningful analysis.

- Each section should contain adjacent areas for valid PCI calculation. One area should have a utility patch and the other area should not have a patch. A valid PCI inspection area must be 2500 sf ± 1000 sf in size.
- The selected sections should contain a variety of cuts from different utility services (i.e. water, gas, electric). This is to ensure that the results of the study are not limited to cuts from a particular utility service type.
- At least 30 ft on one of the sides of the utility patch should be free from additional utility patches. This is necessary to ensure that the patch has no influence on the adjacent control area.
- The selected sections should allow for safe traffic control during the FWD testing. Sections should not be located in or adjacent to hazardous areas such as busy intersections.
- The selected sections should be representative of the different street functional classifications in the County. This includes commercial and residential streets.

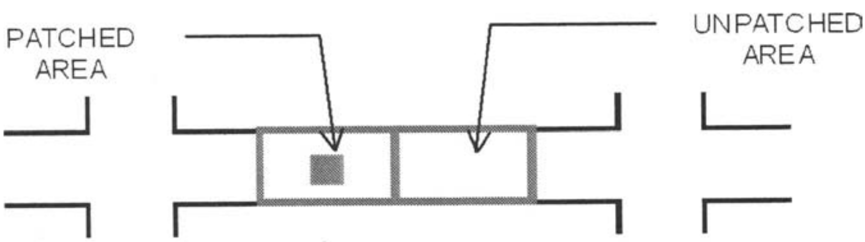

Figure 13-1. Sample Unit Selection.

13.1.2 PCI Data Analysis and Results

Two separate sample units were surveyed in each section. One sample unit included a utility patch, and an adjacent unit contained no patches (classified as No Patch). The unit without patches was surveyed according to the PCI method. The area with patches was surveyed with two different methods. The first method performed was the standard PCI method. This method includes patches as a specific distress (classified as U_Patch). The second method is a modified PCI survey method. Since the PCI lists a patch as a distress, it could be construed that the PCI method unfairly represents the pavement condition in terms of the impact of utility patches. As a result, the PCI survey was performed a second time without including the patch as a distress. However, distresses that were caused by the patch were recorded (classified as U_Distress). The results of the PCI calculation are shown in Figure 13-2. A comparison of the PCI for No Patch and U_Patch is shown in Figure 13-3. A comparison of the PCI for No Patch and U_Distress is shown in Figure 13-4. The figures clearly show that the No Patch PCI is much higher than the PCI's for U_Patch and U_Distress.

Branch ID	PCI		
	No_Patch	U_Patch	U_Distress
PG-01	75	71	68
PG-02	78	46	49
PG-03	43	38	37
PG-04	91	86	90
PG-05	52	43	42
PG-06	74	77	79
PG-08	55	36	24
PG-09	52	41	49
PG-10	87	53	57
PG-11	76	72	71
PG-12	100	84	76
PG-14	86	75	82
PG-15	82	70	72
PG-16	67	40	54
PG-17	56	45	52
PG-18	34	43	37
PG-20	48	47	48
PG-21	69	42	45
PG-22	64	61	61
PG-23	73	45	45
PG-24	49	79	87
PG-25	100	86	91
PG-26	86	79	85
PG-27	86	87	81
PG-28	76	64	66
PG-29	82	69	74
PG-30	75	63	73

Figure 13-2. PCI Survey Results.

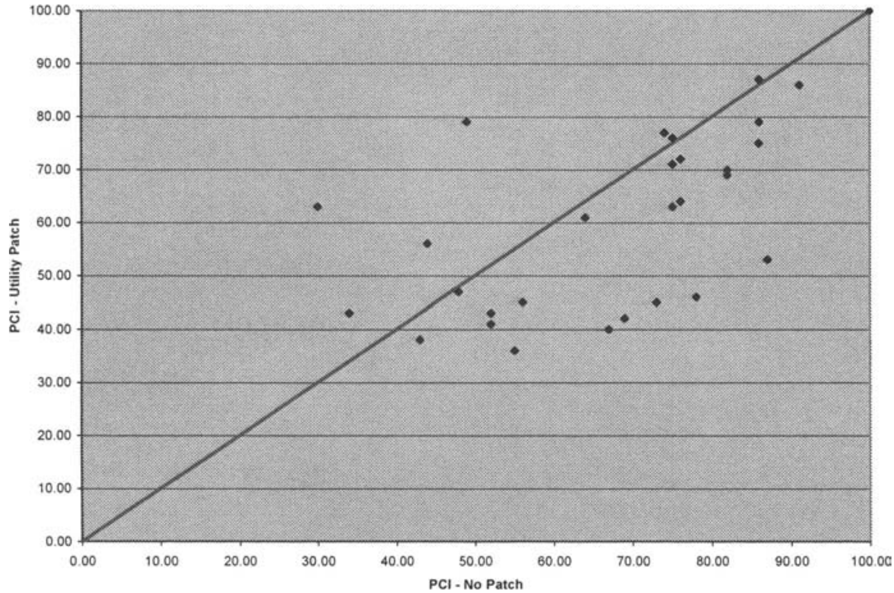

Figure 13-3. Comparison of PCIs from Distresses including Patch vs. Non-patched Area.

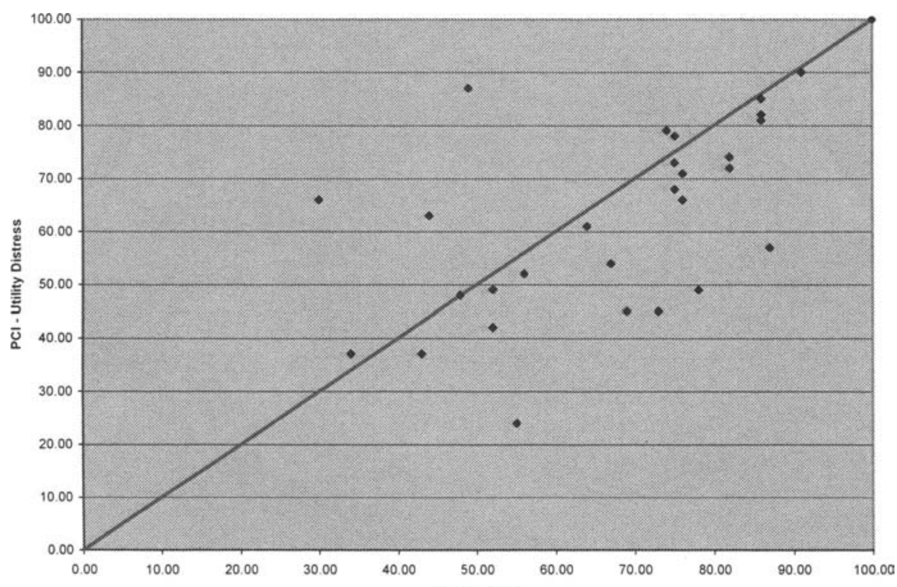

Figure 13-4. Comparison of PCIs from Distresses Caused by Patch vs. Non-patched Area.

In order to determine the impact of the utility patches on pavement life, it is necessary to determine the age of each pavement section at the time of inspection. Since no accurate construction records were available for all the streets, it was decided to estimate the age for all the streets based on the calculated PCI for the No Patch section. To calculate age, a critical PCI of 60 was selected as the value below which major rehabilitation (i.e. overlay) would be required. Four rates of deterioration were selected for analysis that translates to pavement lives of 30, 25, 20, and 15 years (i.e. time required for PCI to drop from 100 to 60). The same pavement age calculated for the No Patch units was also assigned to the U_Patch and U_Distress of the same pavement section. The analysis was performed by creating four separate databases using Micro PAVER, one for each design life (i.e., 30, 25, 20, and 15). The databases were populated with the PCI survey data. A PCI deterioration model was developed in PAVER for each of the U_Patch and U_Distress cases and the age corresponding to a PCI of 60 was calculated. The results are illustrated in Figure 13-5.

	30 Year Design Life			25 Year Design Life		
	Expected Life (Years)	Reduction Factor	Percent Reduction	Expected Life (Years)	Reduction Factor	Percent Reduction
No Patch	30.00	--	--	25.00	--	--
Utility Patch	18.81	1.59	35%	15.62	1.60	35%
Utility Distress	22.53	1.33	25%	18.53	1.36	25%
	20 Year Design Life			15 Year Design Life		
	Expected Life (Years)	Reduction Factor	Percent Reduction	Expected Life (Years)	Reduction Factor	Percent Reduction
No Patch	20.00	--	--	15.00	--	--
Utility Patch	12.50	1.60	35%	9.32	1.61	35%
Utility Distress	14.75	1.36	26%	11.08	1.35	25%

Figure 13-5. Effect of Utility Patches on Pavement Life.

The results of this comparison are consistent among all the databases. The survey data including the patches experienced a 35% drop in pavement life, or a reduction factor of about 1.60. The survey data that included only the distresses caused by the patch experienced a 25% drop in pavement life, or a reduction factor of about 1.36. Even though the second approach is more conservative, it does not include the effect of utility cuts on roughness.

13.1.3 Structural Data Analysis and Results

Deflection testing was conducted using the heavy weight deflectometer (HWD). The HWD has a force range of 3,000 to 55,000 lbs. Four drops producing loads of approximately 9,000, 9000, 11,000 and 15,000 lbs were used at each test location. Seven (7) velocity transducer response sensors were utilized for recording pavement response. The sensors were placed at the center of the loading plate, and at radial offsets of 8, 12, 24, 36, 48, and 60 inches.

For each of the 30 pavement sections, deflection testing was conducted to provide data necessary for a structural assessment of the pavement system. Testing locations were selected in and around existing utility patches as well as within an adjacent control section free of utility patches. The general configuration of test locations is provided in Figure 13-6.

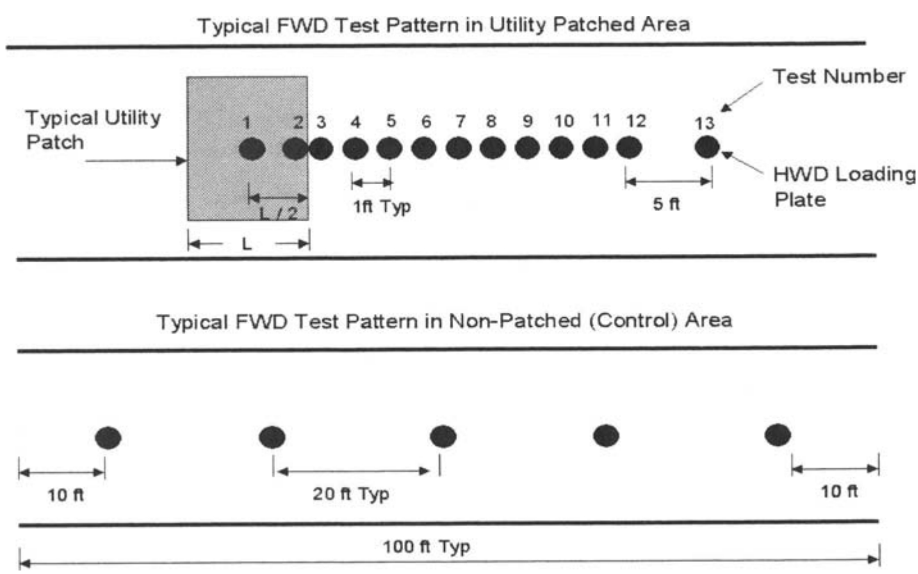

Figure 13-6. FWD Testing Locations.

A comparative analysis of the structural integrity of the pavement system was conducted on four subsets of the test data as follows:

Subset 1: Data collected with the load plate positioned on the pavement with the edge of the plate positioned at distances of 0, 1 and 2 ft from the edge of the utility patch. (Data points 3, 4 & 5 in Figure 13-6)

Subset 2: Data collected with the load plate positioned on the pavement with the edge of the plate positioned at distances of 7, 8 and 9 ft from the edge of the utility patch. (Data points 10, 11, & 12 in Figure 13-6)

Subset 3: Data collected with the load plate positioned on the pavement with the edge of the plate positioned at distances of 8, 9 and 14 ft from the edge of the utility patch. (Data points 11, 12, & 13 in Figure 13-6)

Subset 4: Data collected with the load plate positioned on the un-patched control pavement section.

Air and surface temperature data were recorded during testing to estimate an appropriate maximum deflection temperature adjustment factor (TAF) following guidelines provided by the Asphalt Institute (AI, MS-17). The TAF values were used to provide representative maximum deflections normalized to a common mix temperature of 70 °F as follows:

$$\delta_{adj} = \delta_{meas} \times TAF$$

where

δ_{adj} = temperature adjusted maximum deflections, mils

δ_{meas} = measured maximum deflection, mils

TAF = temperature adjustment factor

Using the results of the second test drop at approximately 9,000 lbs, all deflection data was linearly normalized to 9,000 lbs by the following:

$$\delta_{norm} = (9{,}000\,\delta_{meas})/P$$

where

d_{norm} = normalized deflection, mils @ 9,000 lb

P = applied load, lbs

13.1.3.1 Outlier Analysis

Using the adjusted and normalized maximum deflection data, a data review was conducted to determine if any of the tested sections included data anomalies or outliers that would necessitate the removal of these sections from further analysis.

First, the FWD testing engineer reported that the FWD sensors for Section 7 showed an unexplained reading of zero at some of the load levels. Therefore, it was decided not to include Section 7 in the analysis to reduce the likelihood of error. The remaining 29 sections were analyzed for outliers using a variable defined as the ratio between the measured deflection (δ_{meas}) of the control section and Subset 4. The average ratio was calculated to be 1.0293 and the standard deviation was calculated to be 0.3053. A two standard deviation confidence interval was used (>95%) and sections 13 and 19 were identified as outliers. As a result, sections 7, 13, and 19 were not included in the analysis.

13.1.3.2 Maximum Deflection Response Analysis

A maximum deflection response analysis was conducted following guidelines presented by the Asphalt Institute (MS-17). For this analysis, the average normalized maximum deflection (temperature adjusted) was computed for each of the 4 data subsets within each test section. This average deflection was used to compute a representative rebound deflection (AI, 1983) as follows:

$$RRD = 1.61\ \delta_{ave}$$

where

RRD = representative rebound deflection

δ_{ave} = average normalized maximum deflection (temperature adjusted), mils @ 9,000 lb

Based on Asphalt Institute procedures (AI, MS-17), the RRD for each data subset was used to compute the remaining life equivalent single axle loads to failure ($ESAL_r$) using the following:

$$ESAL_r = (1.0363 / RRD)^{4.1017}$$

A comparative $ESAL_r$ ratio was computed for the patched area (data subset 1) versus each of the three remaining data subsets as follows:

$$ESAL_r\ Ratio = ESAL_{r\text{-}1} / ESAL_{r\text{-}i}$$

where

$ESAL_{r-1}$ = $ESAL_r$ for data subset 1

$ESAL_{r-i}$ = $ESAL_r$ for data subset i = 2 to 4

Figure 13-7 provides a listing of the results of this comparative analysis. As shown, the vast majority of the analyzed test sections have $ESAL_r$ Ratios less than 1 and the overall averages and median values are significantly less than 1 for each subset comparison. The average value based on deflection near the patch as compared to the control is 0.64. This translates to a structural life reduction factor of 1.56. An $ESAL_r$ Ratio less than 1 indicates the un-patched pavement area immediately adjacent to the utility patch has a significantly shorter projected remaining life due to a diminished structural integrity of the pavement system in this area. This is most likely due to structural weakening of the underlying base/subgrade materials during the patching operations.

$ESAL_r$ Ratios			
Section	$ESAL_{r,1}$ / $ESAL_{r,2}$	$ESAL_{r,1}$ / $ESAL_{r,3}$	$ESAL_{r,1}$ / $ESAL_{r,4}$
1	0.44	0.42	0.20
2	0.99	0.69	0.10
3	0.64	0.71	2.23
4	0.31	0.39	0.42
5	0.94	0.93	0.33
6	0.39	0.30	0.19
7			
8	0.44	0.52	0.68
9	0.27	0.25	0.42
10	0.40	0.40	0.68
11	0.31	0.25	0.35
12	0.96	1.51	1.73
13			
14	0.40	0.30	0.21
15	0.12	0.11	0.26
16	0.65	0.64	0.29
17	0.83	0.85	0.49
18	0.61	0.50	0.59
19			
20	0.43	0.47	0.11
21	0.20	0.17	0.12
22	0.80	0.69	0.96
23	1.37	1.41	1.04
24	0.29	0.25	0.84
25	0.61	0.71	1.25
26	1.20	1.31	0.09
27	0.61	0.46	0.45
28	0.09	0.07	0.01
29	1.47	1.29	0.76
30	1.07	1.20	2.50
Average =	0.62	0.62	0.64
Median =	0.61	0.50	0.42

Figure 13-7. Maximum Deflection Based ESAL Ratios.

13.1.3.3 Maximum Deflection Overlay Analysis

The computed RRD values for each data subset were used to estimate the effective modulus of the overall pavement system using the equation (AI, MS-17):

$$E_2 = 672 / RRD$$

where

E_2 = equivalent modulus of the pavement system, ksi

Using this computed equivalent pavement modulus, the required overlay thickness necessary to reduce the RRD for data subset 1 (patched areas) to a design rebound deflection equivalent to the RRD values computed for the other data subsets was computed using the formula (AI, MS-17):

$$\delta_d = (672/E_2)(\{1-[1+.8(h_{ol}/6.4)^2]^{-0.5}\}(E_2/E_1) + \{1+[0.8\,(h_{ol}/6.4)(E_1/E_2)^{1/3}]^2\}^{-0.5})$$

where

d_d = design rebound deflection

E_1 = modulus of HMA overlay, assumed = 500 ksi

h_{ol} = required overlay thickness, inches

Figure 13-8 provides a listing of the required overlay thickness for subset 1 (patched areas) to lower the design rebound deflection to the values in each of the subsets analyzed. Negative overlay thickness is indicative of sections where the RRD of the data subset(s) was greater than the RRD of the patched area and therefore the overlay would be required in the un-patched area (subset 2, 3 or 4). As shown, 66 of the 81 subsets analyzed show the need for an overlay of the patched area, which indicates the pavement areas immediately adjacent to the patched area are predominantly weaker than the un-patched pavement areas. Furthermore, of the 27 sections analyzed, the average overlay thickness required to reduce subset 1 deflections to those in subsets 2, 3 and 4 are 1.50, 1.52, and 1.84 inches, respectively.

13.1.3.4 Deflection Basin Analysis

The analysis of deflection basins allows for the determination of critical load induced tensile strains at the bottom of the HMA layers well as the determination of the fatigue life of the HMA layer. Within this analysis method, surface deflections are first used to compute a deflection basin parameter known as the Area Under the Pavement Profile (AUPP, figure 13-9) as follows (Thompson, 1999):

$$AUPP = \tfrac{1}{2}\,(5\delta_0 - 2\delta_{12} - 2\delta_{24} - \delta_{36})$$

where

d_0 = normalized maximum deflection (temperature adjusted), mils @ 9,000 lb

d_{12} = surface deflection recorded 12 inches from the center of loading, mils @ 9,000 lb

Overlay Thickness for Subset 1, in.			
Section	Subset 2	Subset 3	Subset 4
1	2.50	2.60	4.00
2	-0.90	0.50	4.50
3	1.70	1.50	-4.00
4	2.10	1.90	1.80
5	0.60	0.60	2.70
6	2.50	2.90	3.70
7			
8	2.70	2.40	1.80
9	3.10	3.20	2.40
10	2.40	2.40	1.50
11	2.70	3.00	2.50
12	0.60	-1.80	-2.10
13			
14	2.10	2.50	3.00
15	4.20	4.40	3.00
16	1.50	1.50	2.80
17	1.00	0.90	2.00
18	1.60	1.90	1.60
19			
20	1.60	1.50	3.00
21	2.40	2.50	2.90
22	1.30	1.70	0.60
23	-1.40	-1.50	-0.60
24	2.70	2.90	0.90
25	1.80	1.50	-1.20
26	-1.10	-1.30	5.30
27	1.50	2.00	2.00
28	3.50	3.80	7.00
29	-1.50	-1.20	1.20
30	-0.80	-1.20	-2.60
Average =	1.50	1.52	1.84
Median =	1.70	1.90	2.00

Figure 13-8. Deflection Based Overlay Thickness Requirements.

d_{24} = surface deflection recorded 24 inches from the center of loading, mils @ 9,000 lb

d_{36} = surface deflection recorded 36 inches from the center of loading, mils @ 9,000 lb

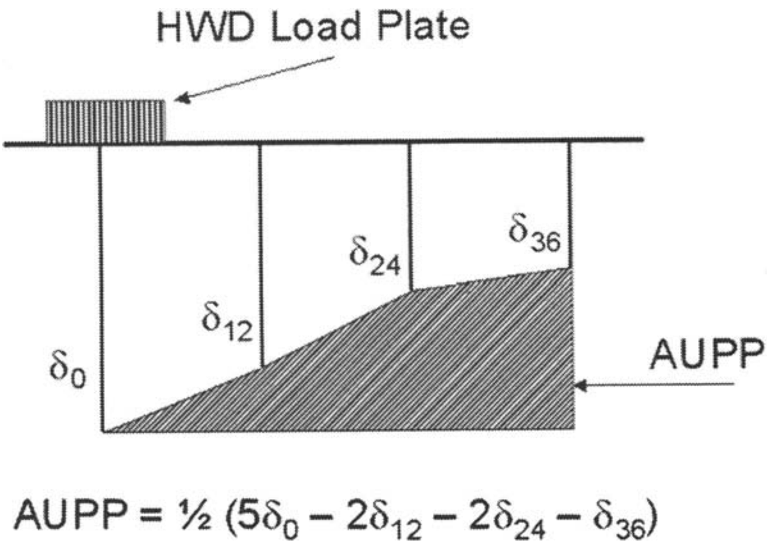

Figure 13-9. Area Under the Pavement Profile (AUPP) Calculation.

The critical tensile strain at the bottom of the HMA layers is then estimated from the computed AUPP as follows (Crovetti, 2002):

$$\varepsilon_t = 14.497\ \text{AUPP}^{0.873406}$$

where

e_t = critical tensile strain at bottom of HMA layer, micro units

The estimated ESALs to fatigue failure of the HMA layer are computed as (Thompson, 1987):

$$\text{ESAL} = 5 \times 10^{-6} (\varepsilon_t)^{-3.0}$$

The average estimated fatigue life of the pavement, based on the data collected within each data subset, is computed as:

$$\text{ESAL}_f = 10^{\text{Ave Log ESAL}}$$

where

ESAL_f = estimated ESALs to fatigue failure

Ave Log ESAL = average of the Log of ESAL computed for each data point within the subset

A comparative $ESAL_f$ ratio was computed for the patched area (data subset 1) versus each of the three remaining data subsets as follows:

$ESAL_f$ Ratio = $ESAL_{f-1}$ / $ESAL_{f-i}$

where

$ESAL_{f-1}$ = $ESAL_f$ for data subset 1

$ESAL_{f-i}$ = $ESAL_f$ for data subset i = 2 to 4

Figure 13-10 provides a listing of the results of this comparative analysis. As shown, the vast majority of the analyzed test sections have $ESAL_f$ Ratios less than 1 and the overall averages and median values are significantly less than 1 for each subset comparison. The average ratio for the control section is computed to be 0.63, which translates to a structural life reduction factor of 1.59. An $ESAL_f$ Ratio less than 1 indicates the un-patched pavement area immediately adjacent to the utility patch has a significantly shorter projected fatigue life due to a diminished structural integrity of the pavement system in this area.

13.1.4 Calculation of Extra Rehabilitation Cost Due to Utility Cut Patching

It has been shown that the presence of the utility patches decreases the life of the pavement by at least 25%. To maintain roads in good condition, Prince George's County must therefore perform rehabilitation practices at a greater frequency, thereby increasing their costs over a given time period. To estimate the extra costs incurred, the following assumptions were made:

- Current overlay practices call for a 2" asphalt overlay
- The expected life without a utility cut is 20 years
- The expected life with a utility cut is 15 years (25% life reduction based on PCI analysis)
- The additional asphalt thickness required to bring the deflection to the same level before the utility cut is 1.5 inches (based on FWD data analysis)
- Prince George's County has a total of 1700 miles with an average width of 26 feet
- Approximately 40% of the miles have patching
- Approximately 1/3 of the miles have manholes and 70% of these miles have utility cut patching
- Manhole realignment cost is $2500/mile
- Approximately 50% of the roads will require cold planing prior to overlay
- The cost of cold planing is $1.5/sy/inch
- The cost per ton of asphalt is $42.60

$ESAL_r$ Ratios			
Section	$ESAL_{r=1}$ / $ESAL_{r=2}$	$ESAL_{r=1}$ / $ESAL_{r=3}$	$ESAL_{r=1}$ / $ESAL_{r=4}$
1	0.41	0.42	0.50
2	1.16	0.66	0.17
3	0.55	0.61	2.32
4	0.12	0.13	0.13
5	0.94	0.98	0.54
6	0.50	0.42	0.27
7			
8	0.53	0.65	0.76
9	0.34	0.34	0.38
10	0.21	0.23	0.79
11	0.33	0.25	0.31
12	0.78	1.15	1.05
13			
14	0.44	0.36	0.22
15	0.07	0.06	0.07
16	0.74	0.71	0.30
17	0.87	0.85	0.49
18	0.69	0.58	0.57
19			
20	0.45	0.53	0.16
21	0.27	0.23	0.14
22	0.96	0.89	0.95
23	1.24	1.33	0.99
24	0.32	0.30	1.24
25	0.58	0.67	1.03
26	1.22	1.38	0.18
27	0.78	0.62	0.64
28	0.12	0.09	0.02
29	1.73	1.59	0.86
30	1.17	1.19	2.04
Average =	0.65	0.64	0.63
Median =	0.55	0.61	0.50

Figure 13-10. Strain Fatigue Based ESAL Ratios.

13.1.4.1 Rehabilitation Costs for Areas without Utility Cuts

The first step in the process is to calculate the rehabilitation costs for areas without utility cuts. The basic cost for overlays (OL) on an annual basis is shown by the following equation:

$$\text{Annual OL Cost} = \frac{\text{Overlay thickness} \times \text{Cost of asphalt} \times \text{Number of miles}}{\text{Frequency of Rehab}}$$

$$\text{Annual OL Cost} = \frac{2''/12\ ft \times 5{,}280\ ft \times 26\ ft \times 0.075\ tons/ft^3 \times \$42.6 \times 1700}{20}$$

$$= \$6{,}213{,}636/yr$$

However, the cost for manhole realignments and cold planing associated with the overlay must also be considered.

$$\text{Annual Manhole Realignment Cost} = \frac{\text{Cost per mile} \times \text{\# of miles with manholes}}{\text{Frequency of Realignment}}$$

$$\text{Annual Manhole Realignment Cost} = \frac{\$2{,}500/mile \times (1/3 \times 1{,}700)}{20}$$

$$= \$70{,}833$$

$$\text{Annual Cold Planing Cost} = \frac{\text{Cost per mile} \times \text{Number of miles}}{\text{Frequency of Cold Planing}}$$

$$\text{Cold Planing Cost per Mile} = \$1.5/yd^2/in. \times 2in. \times (5{,}280 \times 26/9)$$

$$= \$45{,}760 \text{ without utility cut patching}$$

$$\text{Annual Cold Planing Cost} = \frac{\$45760/mile \times (1/2 \times 1{,}700)}{20}$$

$$= \$1{,}944{,}800$$

Total Annual Cost for un-patched areas is \$8,229,269

13.1.4.2 Rehabilitation Costs for Areas with Utility Cuts

The second step in the process is to calculate the rehabilitation costs given that utility cuts do occur. The cost for is more complex. It is assumed that 50% of the roads have areas of utility cut patching:

$$\text{Annual OL Cost}_{cut} = \frac{\text{Overlay thickness} \times \text{Cost of asphalt} \times \text{Number of miles}}{\text{Frequency of Rehab}}$$

$$\text{Annual OL Cost}_{nocut} = \frac{\text{Overlay thickness} \times \text{Cost of asphalt} \times \text{Number of miles}}{\text{Frequency of Rehab}}$$

$$\text{Annual OL Cost}_{cut} = \frac{3.5''/12\,ft \times 5{,}280\,ft \times 26\,ft \times 0.075\,tons/ft^3 \times \$42.6 \times 1{,}700 \times 0.4}{15}$$

$$= \$5{,}799{,}393/yr$$

$$\text{Annual OL Cost}_{nocut} = \$6{,}213{,}636 \times 0.6 = \$3{,}728{,}182/yr$$

Total Annual Overlay Cost = \$9,527,575

The manhole alignment and cold planing calculations were based on the following assumptions: 1/3 of the entire road network has manholes, 70% of the roads with manholes are patched. Based on the greater thickness of overlays on patched areas, the thickness of cold planing will be 3.5 inch for patched areas.

Number of miles with manholes and patches = 70% x (1/3 x 1700) = 397 miles

Number of miles with manholes and no patches = 1700/3 – 397 = 170 miles

$$\text{Annual Manhole Realignment Cost} = \frac{\text{Cost per mile} \times \text{\# of miles with manholes}}{\text{Frequency of Realignment}}$$

Annual Manhole Cost_{nocut} = \$2500/mile x 170 x (1/20) = \$21,250

Annual Manhole Cost_{cut} = \$2500/mile x 397 x (1/15) = \$66,167

Total Annual Manhole Realignment Cost = \$87,417

$$\text{Annual Cold Planing Cost} = \frac{\text{Cost per mile} \times \text{Number of miles}}{\text{Frequency of Cold Planing}}$$

Annual Cold Planing_{nocut} = \$45,760/mile x (1700/2) x 0.6 x 1/20 = \$1,166,880

Cold Planing Cost per Mile for Areas with Utility Cut Patching = \$1.5 x 3.5 in x (5280 x 26/9) = \$80,080

Annual Cold Planing_{cut} = \$80,080/mile x (1700/2) x 0.4 x 1/15 = \$1,815,147

Total Annual Cold Planing Cost = \$2,982,027

Total Annual Cost for areas including utility cuts is \$12,597,019

13.1.4.3 Extra Rehabilitation Cost

Based on the calculations above, the average annual increase in rehabilitation costs is estimated at $4,367,750, or approximately $4.4 million. The costs are further illustrated in Figure 13-11.

	Overlay Cost	Manhole Alignment	Cold Planing	TOTAL
Areas without Utility Patches	$6,213,636	$70,833	$1,944,800	$8,229,269
Areas with Utility Patches	$9,527,575	$87,417	$2,382,027	$12,597,019
			Extra Cost	($4,367,750)

Figure 13-11. Extra Rehabilitation Costs Incurred as a Result of Utility Cuts.

13.1.5 Conclusions

Based on the FWD and PCI analysis presented herein, it is concluded that pavement life is significantly reduced by utility cut patching. The FWD showed a conservative estimate of a structural life reduction factor of over 1.5. The measured maximum deflection is generally much higher near the patch as compared to away from the patch as illustrated in Figure 13-12 for test section number 1. The FWD analysis also showed that the average required overlay thickness to compensate for the weakening of the pavement around the patch is over 1.5 inch. The PCI analysis showed a life reduction factor of over 1.33, i.e. instead of a 20 year pavement life, a pavement with utility cut patching is expected to require overlay in 15 years instead.

The life reduction combined with the increased overlay thickness requirements result in an increased annual rehabilitation cost to the county of approximately $4.4 million.

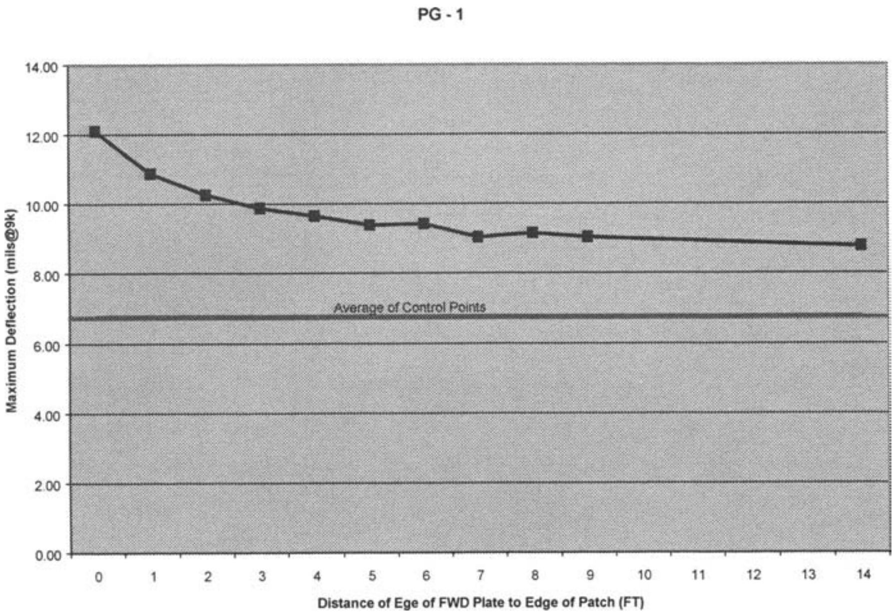

Figure 13-12. Maximum Deflection Values for Section # 1.

13.2 City of Los Angeles, CA (Shahin, Chan, and Villacorta 1996)

13.2.1 Pavement Testing Program

A total of 100 street sections were randomly selected and surveyed to provide a representative data sample. Fifty of these sections were functionally classified as "Local" and the other fifty as "Select" which is the term used for arterial. All selected sections were flexible (asphalt) pavements. Two adjacent inspection units (2500 Sqft ± 40%) were selected from each section where one of the units had utility cut patches while the adjacent unit did not. The surface condition was quantified using the Pavement Condition Index (PCI) method. The structural adequacy of the patched and non-patched pavement was evaluated using a falling weight deflectometer (FWD). Pavement deflections were measured inside and outside the patches. A standard penetration test was also conducted to determine the relative strength of the soil in the patch as compared to the original pavement.

13.2.2 PCI Data Analysis and Results

The PCI results were used to establish four pavement deterioration models (also known as family curves):

1. Select without patching
2. Select with patching

3. Local without patching
4. Local with patching

The PCI deterioration family curves were developed using Micro PAVER. A Critical PCI was estimated for each family. A Critical PCI is the PCI value beyond which the rate of pavement deterioration increases significantly, and the pavement can no longer be economically maintained without the need for major rehabilitation such as overlay. A pavement life span was defined as the pavement age at which the pavement reaches its Critical PCI. Figure 13-13 shows the Critical PCI and corresponding life span for each family. It should be noted that the Critical PCI was kept the same for within the Select and Local networks to allow for comparison of the effect of utility cut patching on pavement life.

Pavement Family	Critical PCI	LifeSpan (years)
Select without patch	55	25.0
Select with patch	55	16.5
Local with patch	65	34.5
Local with patch	65	28.5

Figure 13-13. Critical PCI and Life Span for Each Family.

13.2.3 Structural Data Analysis and Results

The Falling Weight Deflectometer (FWD) was used to determine the deflections of the existing asphalt concrete pavements in the patched and non-patched areas (Figure 13-14). Figures 13-15 and 13-16 show a comparison of the center load plate deflections for both Select and Local networks respectively. The figures show deflections for pavement away from utility cut patching (Avg PAT: average of PVT1 and PVT2), pavement edge next to patching (Avg PVTE: average of PVTE1 and PVTE2), patch center (PATC), and patch edge (Avg. PATE: average of PATE1 and PATE2). As can be seen, on the average, there is a considerable increase in deflection in and around the patching areas and adjacent pavement edges as compared to the original pavement. This translates into weaker structural support for traffic and shorter pavement life span.

Pavement cores were cut to determine the thickness of the pavement structure. A soil investigation of the subgrade was also conducted to determine type of soil (USCS Classification), moisture content, and standard penetration values. The results of pavement coring (Figures 13-17 and 13-18) show that, on the average, the patch asphalt (AC) surface thickness is considerably less than the original pavement asphalt surface thickness. This also translates into weaker support and shorter pavement life span. The deflection results were used to determine the overlay requirement for each area utilizing the 1993 AASHTO Darwin Pavement Design program. Figure 13-19 shows a summary of the average overlay requirements for both Select and Local streets with and without patching.

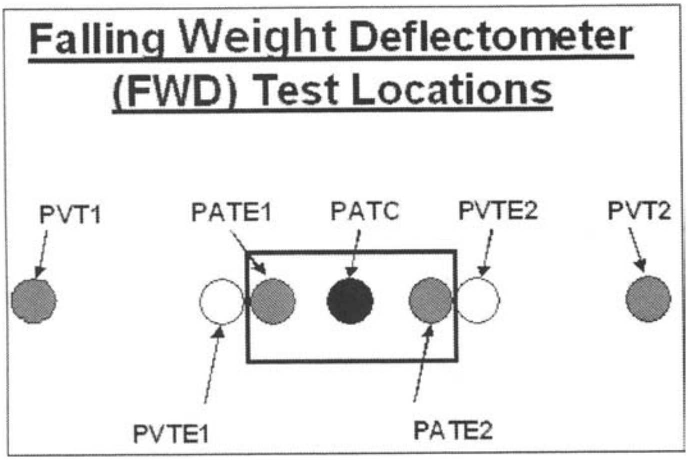

Figure 13-14. FWD Test Locations.

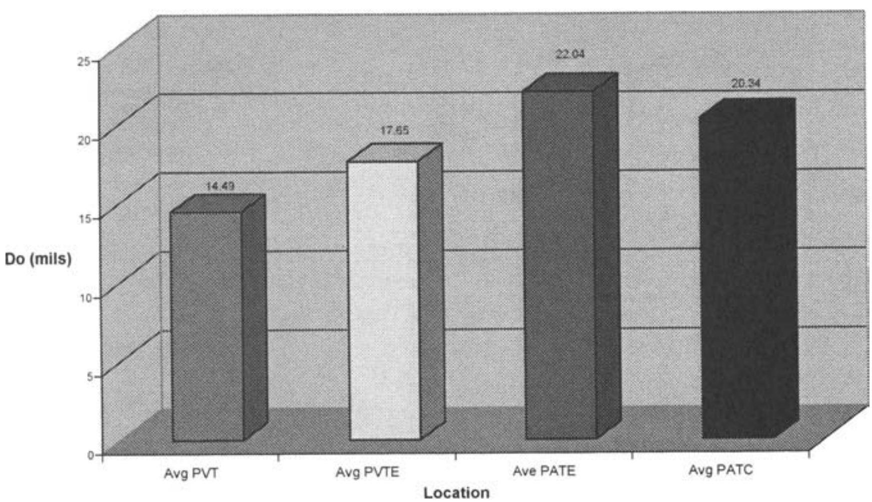

Figure 13-15. Center load plate deflections for Select roads (1 mil = .001 inch).

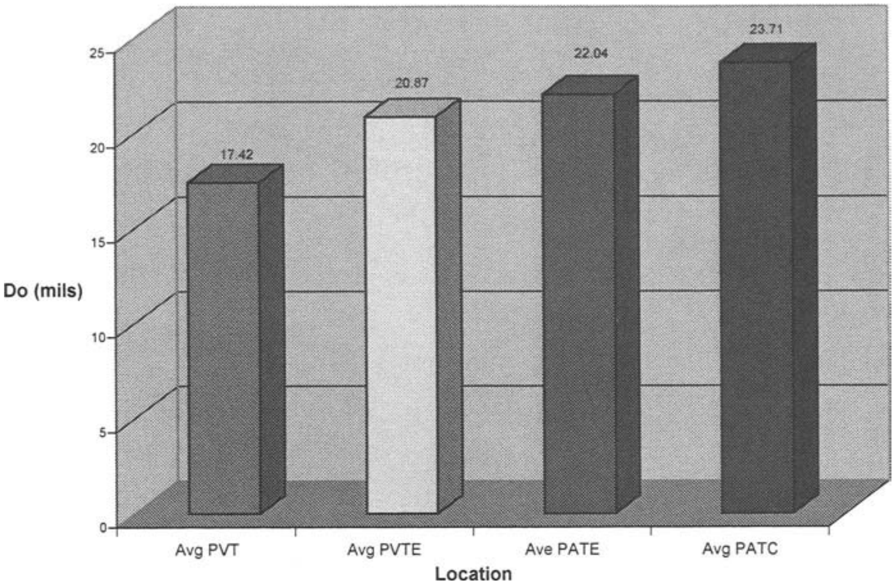

Figure 13-16. Center load plate deflections for Local roads (1 mil = .001 inch).

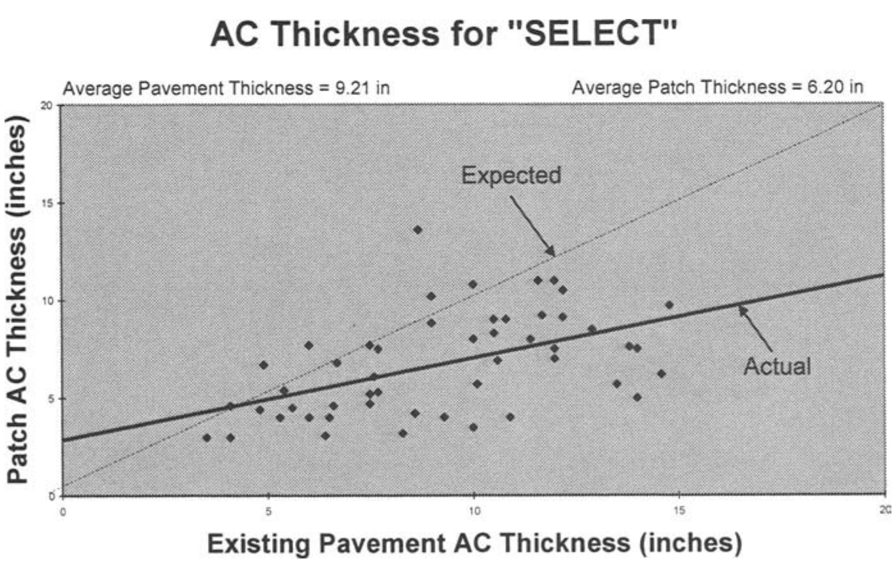

Figure 13-17. Average Asphalt Thickness for Select roads.

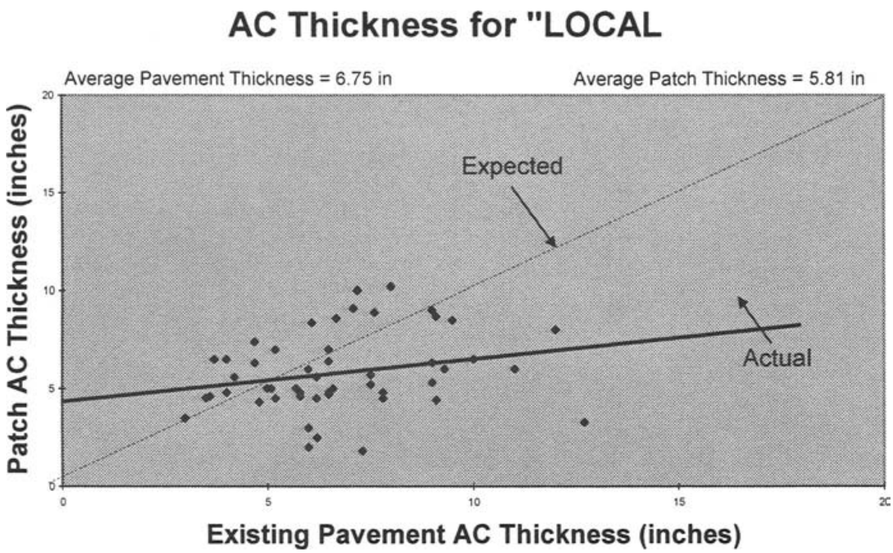

Figure 13-18. Average Asphalt Thickness for Local roads.

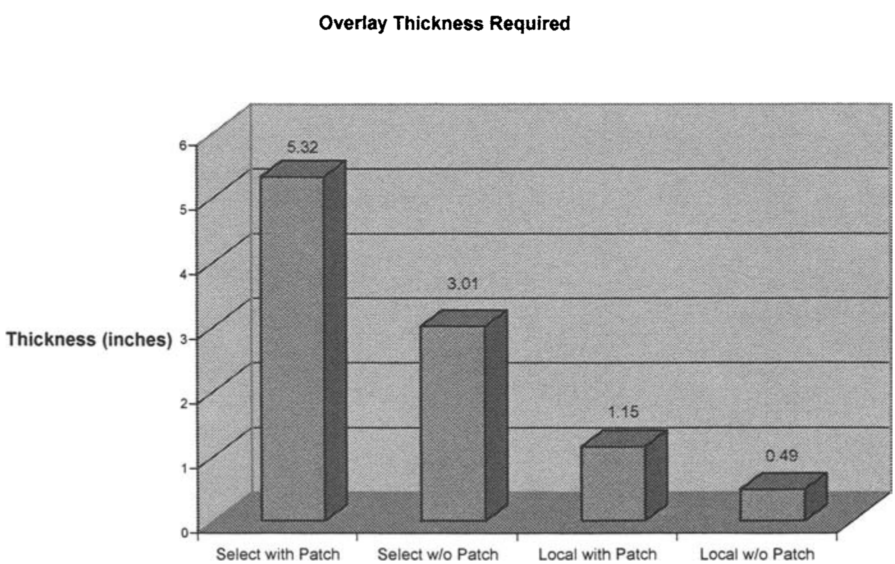

Figure 13-19. Average Overlay Requirements with and without patching.

13.2.4 Calculation of Extra Rehabilitation Cost Due to Utility Cut Patching

The PCI and FWD results were used to calculate the extra annual rehabilitation cost for the Select and Local roads. The Select roads network in the City consisted of 1,469.5 centerline miles with an average width of 53.5 feet. The Local roads network consisted of 3,963.28 centerline miles with an average width of 33.86 feet. Approximately 70% of the Select and Local roads had utility cut patching.

The rehabilitation costs included cold planing, profiling, overlay, and manhole alignment. The cost analysis was performed in 1996 dollars. The annual cost was calculated once assuming no utility cut patching and a second time with 70% of the pavement patched. For Select roads, the annual costs were $11,412,427 and $24,349,378 respectively. In Local roads, the annual costs were $4,180,299 and $7,656,672 respectively. Thus, the extra annual rehabilitation cost was calculated as approximately $12.9 million for Select roads and $3.5 million for Local roads.

13.2.5 Conclusions

Based on the analysis presented, it was concluded that pavement performance is significantly affected by utility cut patching. A life reduction factor of 1.21 for local roads and 1.52 for select roads was determined. The life reduction factor (computed from the PCI survey), and the increased overlay requirements(computed from the FWD analysis), result in significant rehabilitation costs to the city. The increase was calculated at approximately $12.9 million for Select roads and $3.5 million for Local roads. These costs were based on Critical PCI values of 55 for Select roads and 65 for Local roads to avoid excessive reconstruction costs at lower PCI values.

13.3 City of Burlington, VT (Shahin, Crovetti, Franco 1986)

13.3.1 Pavement Testing Program

A representative sample of streets (a total of 50 pavement sections) was randomly chosen from areas throughout the city. A paired experiment was conducted in each section to determine the effect of utility cut patching on the Pavement Condition Index (PCI) for streets of various ages throughout the city. An NDT program was also conducted at positions in and around patched areas using a falling weight deflectometer (FWD) to measure the effects of patching on the structural adequacy of the pavement.

13.3.2 PCI Data Analysis and Results

The PCI survey results were analyzed to determine the effect of utility cut patching on pavement life. Pavement life was defined as the age in years that the street can be economically maintained without the need for major rehabilitation such as an overlay. This was defined as the pavement age at which a PCI value of 70 would be reached. Three methods were used to determine the average pavement life before PCI of 70 would be reached. These methods were the Rate of Deterioration (slope), Best Line fit through PCI vs. Age Data (with PCI = 100 at age of 0), and Best Curve fit through PCI vs. Age Data (with PCI = 100 at age of 0). Figure 13-20 presents a summary of the results obtained from the three analysis methods. As can be seen from this table, the life reduction factor varies from 1.64 to 3.71. For the purposes of follow-up analysis, the 1.64 factor was used as the most conservative assessment of the damage caused as a result of utility cut patching.

Reduction Analysis Method	Average Life of Non-Patched Pavements (years)	Average Life of Patched Pavements (years)	Life Reduction Factor
Method 1: Rate of Deterioration	20.1	11.6	1.73
Method 2: Best-Fit Line Through Data			
*All data	19.8	12.1	1.64
*PCI>40	30.0+	11.9	2.52
Method 3: Best-Fit Curve through data			
*All data	25.9	8.5	3.05
*PCI>40	28.9	7.8	3.72

Figure 13-20. Comparison of Average Pavement Lives.

13.3.3 Structural Data Analysis and Results

The structural strength of pavement sections with and without patching was measured by recording the magnitude (Do) that the surface of the pavement deflected under a load impulse similar in magnitude and duration to a moving truck wheel load. Measured Do data were grouped into four categories based on the test locations illustrated in Figure 13-21. In sound pavement areas (PVT), the average measured Do was 20.43 mils (1 mil = .001 inch). Actual values varied from 9.4 to 50.5 mils. In PVT-E areas, an average value of 29.07 mils was calculated with a range from 13 to 73.9 mils. In PAT-E areas, an average value of 27.68 mils was calculated with a range from 13.6 to 54.3 mils. In central patch areas (PAT), the average value was 25.21 mils with a range from 12.6 to 54.8 mils. The results are presented in Figure 13-22. For comparative purposes, overlay design requirements were calculated for pavements with and without patches. Pavements with patching are those in locations PVT-E, PAT-E, and PAT. Pavements without patches are those in test locations PVT. The overlay thickness requirements were determined using the Asphalt Institute method and the results are shown in Figure 13-23.

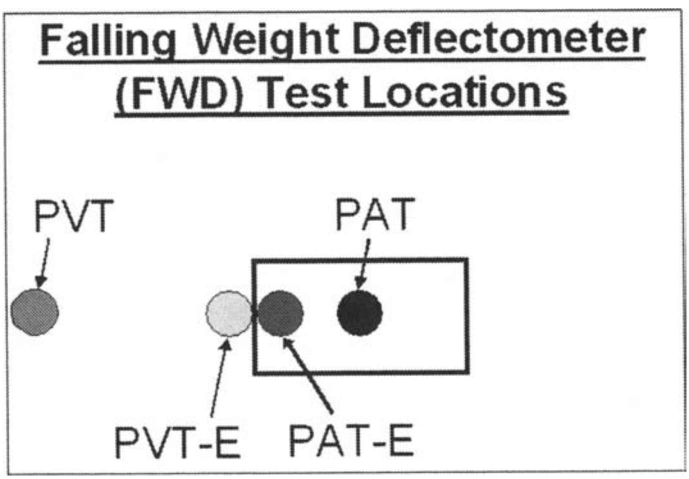

Figure 13-21. FWD Test Locations.

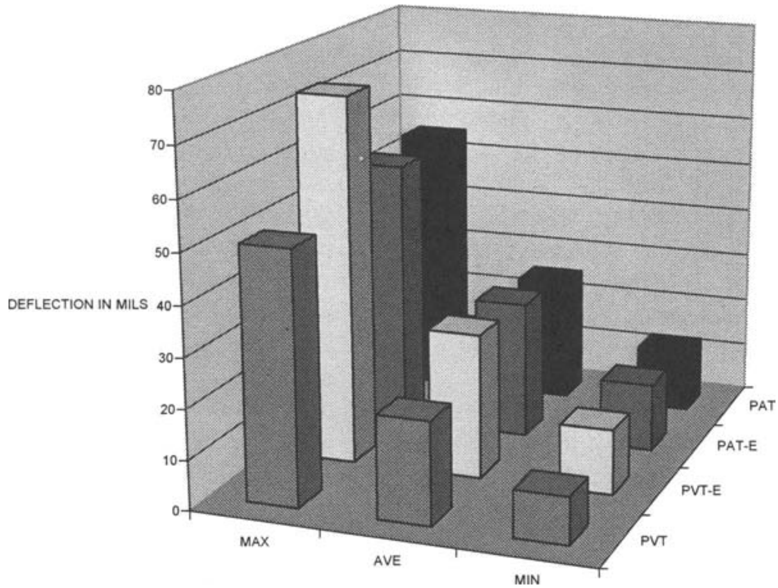

Figure 13-22. Effect of Patching on Deflection (Do) .

Traffic	Non-Patched Areas	Patched Areas
2 ESAL/day	0.00"	1.50"
10 ESAL/day	2.25"	3.00"
20 ESAL/day	3.00"	3.75"

Figure 13-23. Overlay Requirements.

13.3.4 Calculation of Extra Rehabilitation Costs Due to Utility Cut Patching

To compute the extra rehabilitation costs associated with utility cut patching, the results obtained from both the PCI and deflection analyses were utilized. The life reduction factor of 1.64, as determined from the PCI analysis, and the overlay thickness requirements for 10 ESAL/day, as determined from the overlay requirement analysis, were used to compute the overlay costs in 1984 dollars. Inflation and interest rate adjustments were excluded from the cost figures. The average annual rehabilitation cost was calculated as the sum of the overlay cost, curb replacement cost, and manhole alignment cost. The total network of streets in the city contains 87 miles of pavement with an average width of 30 feet. At the time of the study, 88% of this mileage contained utility cut patching. The annual cost was calculated for the total network assuming no utility cut patching and repeated with 88% of the pavement patched. The annual costs from the calculations were $599,443 and $1,113,655 respectively. Thus, the annual extra rehabilitation cost for the network was calculated as the difference between the two costs or $514,212 per year.

13.3.5 Conclusions

Based on the analysis presented, it was concluded that pavement performance is significantly affected by utility cut patching. The life reduction factor, computed from the PCI survey, and the increased overlay requirements, computed from the FWD analysis, result in significant rehabilitation costs to the city. The increase was calculated at approximately $514,000 annually for the paved street system. These costs were based on a minimum acceptable PCI value of 70 to avoid excessive reconstruction costs at lower PCI values. Furthermore, it was concluded that utility patching operations on streets with PCI values below 40 would produce no consequential damage.

13.4 City and County of San Francisco, CA (Blue Ribbon Panel 1998)

In 1992, the City commissioned a study to determine the effects of utility cuts on the service life of City streets. The City retained Dr. Ghassan Tarakji, Ph.D., P.E., of the Engineering Design Center at San Francisco State University (SFSU), to perform this task. Local utility companies commissioned a critical review and expressed concerns over the data and methods used in the analysis. In response, in 1997, the city commissioned an expert panel to reevaluate the results of the original study and expand its scope based on new engineering techniques.

13.4.1 Initial Study

The data used in the City and County of San Francisco study was obtained from their existing Pavement Management and Mapping System (PMMS). The PMMS contained a description of each block in the City, the condition of the pavement, number of utility cuts, and other key information. Pavement Condition is measured by the Pavement Condition Score (PCS). The best score a pavement can get is 100. "Points are deducted based on three factors: RIDE, which represents the smoothness or ride quality of the block; RAVELING, which describes the severity and extent of surface erosion from weathering and traffic; and CRACKING, which considers the amount and severity of cracks in the pavement. Points are not deducted for utility cuts unless cracks form around them."

The SFSU Study took data from the PMMS and compared the PCS for streets with few (0-2), some (3-9), and many (more than 9) utility cuts. The Study concluded that streets with some cuts have lower condition scores than streets with few cuts and that streets with many cuts have lower condition scores than streets with some or few cuts. These conclusions were consistent for every functional class of asphalt street and for concrete streets.

Assuming streets require repaving when they reach a pavement condition score of 65, the SFSU Study concluded that:

Asphalt Streets With:	Have a Service Life of:
Less than 3 cuts	26 years
Between 3 and 9 cuts	18 years
More than 9 cuts	13 years

13.4.2 Follow Up Study

In 1997, the City assembled a panel of experts to provide an objective assessment of whether engineering evidence and statistical analyses supported the SFSU conclusion that excavation reduces the condition and service life of City streets. The panel included five Engineers with expertise in pavements and a Statistician with expertise in analyzing pavement data.

The panel used the data from the PMMS and grouped the pavement blocks based on number of utility cuts: none, few (1-2), some (3-9), and many (10 or more). The 1997 Statistical Study was performed using updated PMMS information and included the variables of functional class, age, number of utility cuts, and area and type of utility cuts. Figure 13-24 shows the sample sizes for the data used in the 1997 Analysis, divided into groups by age and the number of cuts present. The Panel including the Statistician determined that the sample size for the 1997 Analysis was more than sufficient to make statistically significant conclusions regarding the impact of utility cuts on the condition of the City's pavement.

Age	No Cuts (0)	Few Cuts (1 to 2)	Some Cuts (3 to 9)	Many Cuts (10 or more)	Total
0-5 Years	1,248	399	246	53	1,946
6-10 Years	373	271	451	127	1,222
11-15 Years	232	331	669	295	1,527
16-20 Years	75	97	232	105	509
Total	1,928	1,098	1,598	580	5,204

Figure 13-24. Number of Blocks in 1997 Analysis By Age and Number of Cuts.

13.4.3 Follow-up Study Results

The Statistician determined that the most significant variables were age and number of utility cuts – the same variables used in the SFSU Study. Figure 13-25 is a bar chart presentation of the analysis. The chart shows the average of pavement condition scores for streets with zero, few, some, and many cuts in various age groups. As can be seen from the figure, the average condition score of streets with trenches is lower than the average condition score of streets with no trenches. Based on the review of the literature, the findings derived from the 1997 Analysis, and their engineering expertise, the Panelists arrived at the following conclusions regarding the impact of utility cuts on the service life of San Francisco's asphalt streets:

1. On average, pavements with utility cuts have lower condition scores than pavements without cuts.
2. On average, increasing the number of cuts reduces condition scores.
3. A large number of cuts early in the life of a pavement dramatically reduces pavement performance.
4. Conclusions 1-3 remain the same, whether considering the number of cuts per block, number of cuts in an area, or the percentage of area cut.
5. Conclusions 1-3 are statistically supported by data to at least a 95% confidence level.
6. The findings of the 1997 Analysis and other municipal utility cut studies are consistent with the following universally accepted engineering principles:

- Street cuts disrupt surface integrity, creating surface roughness, reducing pavement strength, and allowing for entry of moisture, which accelerates long-term deterioration, Figure 13-26.
- Street cuts disrupt pavement layers and supporting soil in the area surrounding the trench. This disruption can be minimized, but cannot be eliminated. As a result, trenching causes unavoidable damage to the pavement layers and soil supporting the pavement around the perimeter of the utility cut.
- Similar to a protective membrane, pavement layers perform best with no cuts or breaks. Street cuts create joints in the pavement layers that reduce the structural integrity of those pavement layers; and
- Although high quality patching may reduce the structural damage caused by utility cuts, the street will still incur ride quality and cracking damage, and its service life will be diminished.

7. The statistical findings of the 1997 Analysis are consistent with the findings of engineering-based studies which used deflection testing to conclude that utility cuts inevitably and irreparably disrupt the subsurface of a street, and that this damage extends beyond the perimeter of the trench.

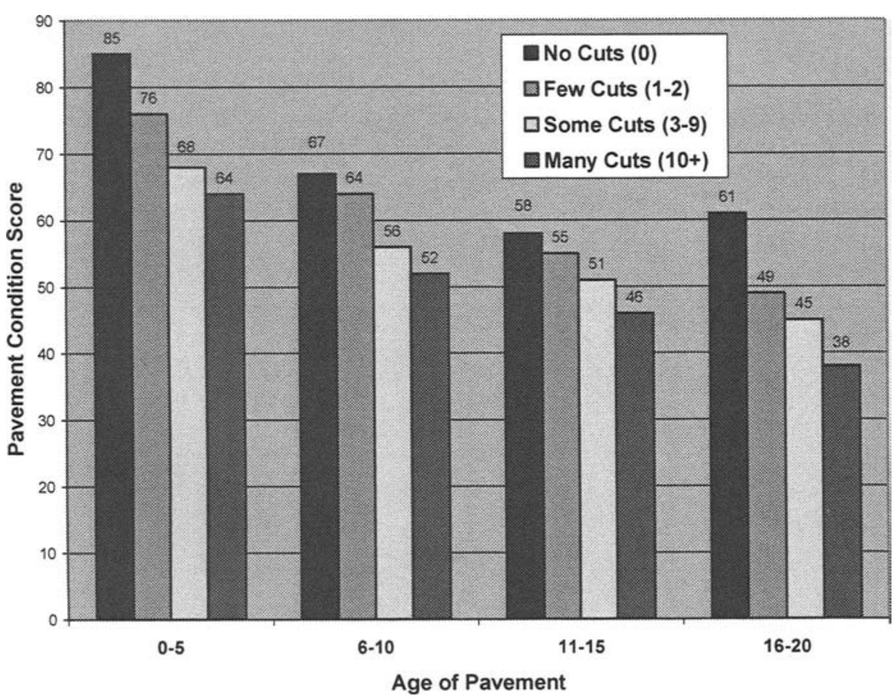

Figure 13-25. Effect of Utility Cuts on Pavement Condition.

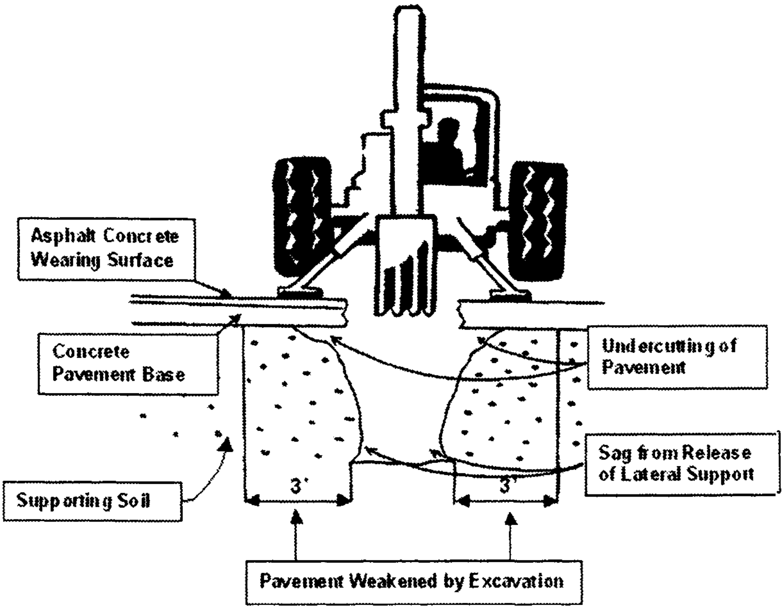

Figure 13-26. Pavement Surface and Soil Disruption from Excavation.

13.4.4 Economic Analysis

The City retained an economist to estimate the extra cost of pavement rehabilitation as a result of utility excavation (Marcus, W. B., 1998). The procedure used to perform the analysis is summarized in the following steps:

1. Identify the total number of blocks that require repaving ("repaving candidates"). This was done using two methods:
   a. Using the decision tree built into the City's PMMS. This method does not consider budgetary constraints.
   b. Using what the economist termed the Excess Failed Street Method (EFS). The EFS method identifies Repaving Candidates as those blocks with a PCS of less than 53. This generally reflects the reality of the City's current repaving practices.
2. Identify the number of "excess" blocks requiring repaving due to excavation. This was determined by comparing the number of repaving candidates with "no cuts" to the number of repaving candidates with "few, some, or many cuts." The analysis presumes that, absent excavation damage, there should be proportionally the same number of repaving candidates in each cut group of the same age. In other words, of streets age 0-5, there should be proportionally the same number of repaving candidates with no cuts, as with few, some, or many cuts.

3. Identify and annualize the total cost to repave excess repaving candidates. The total cost was determined by multiplying the average cost of repaving a block by the total number of excess repaving blocks. The total cost is then annualized over a 26-year paving cycle using a 5.58% discount rate. The 26 years was selected as the City estimated that it repaves the streets at a rate equivalent to 26 years.
4. Convert the annual cost into a square foot cost. This was done by dividing the annual cost by the number of square feet likely to be excavated in a year.

All the above steps were performed twice: once by analyzing only pavement under 20 years, and a second time using all pavements. The 20 years was selected since the City had more confidence in the data for pavement 20 years or less. Figure 13-27 shows a summary of the findings. The City, however, has proposed a conservative fee shown in Figure 13-28.

	EFS Method (Age <= 20 Sample)	PMMS Method (Age <= 20 Sample)	EFS Method (Entire Sample)	PMMS Method (Entire Sample)
Number of Repaving Candidates	1,546	2,653	2,756	3,992
Number of Excess Repaving Candidates Due to Excavation	566	568	1,153	1,000
Total Repaving Cost	$44.5 million	$45.4 million	$69.3 million	$60.1 million
Annual Repaving Cost	$3.3 million	$3.4 million	$5.1 million	$4.4 million
Cost per Square Foot of Excavation	$5.37	$5.49	$8.38	$7.27

Figure 13-27. Economic Analysis Findings.

Age of Street (Years since last repaving)	Fee Amount (Per square foot of excavation)
0-5	$3.50
6-10	$3.00
11-15	$2.00
15-20	$1.00

Figure 13-28. Proposed Restoration Fees.

13.4.5 Conclusions

Based on the results from the 1997 analysis and the subsequent economic report, it was concluded that utility cuts lower pavement condition scores, reduce pavement service life, and increase pavement rehabilitation costs due to accelerated repaving requirements.

13.5 City of Sacramento, CA (1996), (CHEC Consultants, Inc., 1996)

The City of Sacramento, CA contracted with Chec Engineering Consultants to determine the extent and quantify cost associated with utility cuts. Dr. M.Y. Shahin was retained by the City as an independent consultant to review the results of the report prepared by Chec Engineering and to participate in City Council meetings during the discussion of the damage resulting from utility cuts. The following is a summary of the study performed by Chec Engineering.

13.5.1 Pavement Testing Program

The testing program was limited to deflection testing using the Dynaflect device and a coring program to determine asphalt concrete (AC) surface thicknesses. No distress survey or visual condition rating was performed as part of the study. The test sections were grouped in four zones based on soil and traffic conditions.

Separate data collection and analysis were performed for longitudinal and transverse utility cuts. Longitudinal cuts were tested for loss of strength and associated difference in AC overlay requirements. Dynaflect testing was performed on each cut as well as two feet left and right of the trench, if possible. A base line test was also conducted in the same pavement section for comparison purposes.

Transverse cuts were tested for loss of strength and extent of influence from the cut. Each wheel path was tested five feet on either side of the patch, at one-foot intervals. A baseline test was also conducted in the same section for comparison purposes.

13.5.2 Longitudinal Utility Cuts Data Analysis and Results

Each of the pavement sections was analyzed to determine the overlay requirement and the difference between the test area and baseline overlay requirements calculated. The overall average from the four zones (Figure 13-29) showed an extra 1.5 inches of AC overlay is required relative to the baseline.

13.5.3 Transverse Utility Cuts Data Analysis and Results

The purpose of the testing was to determine the distance from the edge of the cut that the pavement is affected. The assumption made is that the only repair needed would be the replacement of the surrounding weak areas prior to overlay. The results of the analysis (Figure 13-30) showed that the average influence from the cut edge is 3.64 feet.

Zone	Average Additional AC Overlay Required (ft)
Zone 1	0.13
Zone 2	0.13
Zone 3	0.11
Zone 4	0.14
Overall	0.13

Figure 13-29. Average Additional AC Overlay Required for Utility Cuts.

Zone	Influence from Cut Edge (ft)
Zone 1	3.35
Zone 2	3.16
Zone 3	3.81
Zone 4	4.24
Overall	3.64

Figure 13-30. Influence of Utility Cuts on Surrounding Pavement.

13.5.4 Calculation of Extra Rehabilitation Costs Due to Utility Cut Patching

For longitudinal cuts, the extra cost was limited to the cost associated with the additional AC thickness required, which was determined to be 1.5 inches. The costs associated with manhole adjustment or cold milling were not included. Using an AC cost of $26/ton, two separate costs were calculated based on whether the extra overlay thickness will be applied to one or two lanes. If the utility cut is within three feet of a lane line, then two lanes will require the extra overlay thickness. For one lane, the extra cost was calculated as $16,068/mile or approximately $3/linear foot. For two lanes, the extra cost was approximately $6/linear foot.

For transverse cuts, only the cost associated with replacing the weakened area prior to overlay was considered. A full depth AC patch (approximately $3/square foot), was assumed for the calculation. Using the 3.64 feet average influence extent, two costs were calculated: one for cuts that go across the entire street width, and another for cuts that cover less than the entire street width. Cuts that cover less than the entire street width exert influence on four sides rather than two. The first cut was assumed to be 2 ft by 24 ft. Therefore, the area to be replaced prior to the overlay is [(2 + 2 x 3.64) x 24] or 223 square feet. At $3/square foot, the cost is $669 or $13.94 per square foot of actual cut. The second cut size was assumed to be 4 ft by 5 ft. Therefore, the area to be replaced prior to the overlay is [(4 + 2 x 3.64) x (5 + 2 x 3.64)] or 139 square feet. At $3/square foot, the cost is $417 or $20.85 per square foot of actual cut.

13.5.5 Conclusions

The study has proven that utility cuts cause a discontinuity in the pavement structure and cause a loss of strength within the adjacent pavement. The approximate extent of influence was also determined.

13.6 Summary and Conclusions

The above studies all indicate that utility cut patches both reduce pavement life and increased costs. These costs are the result of both faster pavement deterioration and increased requirements for overlays. The following conclusions can be drawn based on the results of the previous studies.

- Pavement performance is significantly affected by utility cut patches
- Pavement service life is significantly decreased by utility cut patches
- Overlay thickness requirements are increased by utility cut patches
- Utility cut patches create increased pavement rehabilitation costs for local governments
- Utility cut patches negatively affect pavement outside of the patch area

Figure 13-31 provides a summary of the results from the above studies.

		Life Reduction Factor	Extra Overlay Thickness Required, in.
Prince George's County, MD		1.33	1.50
City of Los Angeles, CA	Local Roads	1.21	0.65
	Select Roads	1.52	2.31
City of Burlington, VT		1.64	1.50
City & County of San Francisco, CA		2.00	NA
City of Sacramento, CA		NA	1.50

$$\text{Life Reduction Factor} = \frac{\text{Pavement Life without Utility Cuts (years)}}{\text{Pavement Life with Utility Cuts (years)}}$$

Figure 13-31. Summary of Results from Relevant Studies.

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14

Special Application – Development of Council District Budget Allocation Methodology for Pavement Rehabilitation

This Application was developed by the author for a large city in the United States.

14.1 Background

Historically, the City used a subjective pavement rating system. The system did not produce meaningful, repeatable condition values usable for budget allocation. As a result, budget allocation was based on the approximate percentage of pavement that required either resurfacing or reconstruction. This was called "percentage need allocation." Many council districts complained about the allocation of funds based on need. Factors such as lower-than-average pavement conditions and/or higher-than-average heavy vehicle traffic were not directly incorporated into the need-based allocation.

14.2 Objective

The objective was to develop an equitable budget allocation methodology based on rational engineering principles and accurate condition data.

14.3 Approach

The approach to achieving the objective included three primary components: collect accurate pavement condition data, assess the collected data with a rational pavement management system, and ensure open communication with council members and the mayor's staff throughout the process.

The first step was to collect accurate pavement data. The City uses Micro PAVER with all streets defined as one entire network and individual streets as branches. Each City block was defined as a section and the section's attribute of zone was used to identify the council district. The Pavement Condition Index (PCI), see Chapter 3, was used for the condition assessment. The distress data collection was performed using a City-owned vehicle equipped with a video camera and three laser devices for measuring rutting and roughness. The surface distress information from the video cameras was viewed on a computer screen, interpreted, and stored in computer data files. Due to the large quantity of area to be surveyed, one-third of the city network was surveyed. The plan was to survey one-third of the roads every year, thus surveying the entire city every three years, which meets the requirements of Government Accounting Standards Board (GASB) Statement No. 34.

The computer data files created from the condition survey were imported into the Micro PAVER system and the PCI was calculated for every pavement section. The area-weighted PCI value for the entire network of the city was 69.6. Detailed PCI values for each district for local and arterial roads are illustrated in Figures 14–1a and 14–1b.

During the budget allocation development process, council members were concerned that the heavier volumes of bus and truck traffic in certain areas should be addressed in the budget allocation model. To alleviate this concern, a traffic study performed by the City Department of Transportation was referenced for data concerning heavy vehicle traffic in each council district. The study counted the frequency of trucks and buses traveling through arterial roads in each district. The study contention was that if truck/bus volumes are high in these areas, it is also indicative of the truck/bus volumes that may penetrate the local streets in those areas.

The traffic data was assessed in two different ways. First, the raw number of trips by trucks and buses was used to calculate a relative volume of traffic for each council district. However, trucks and buses do not have the same impact on pavement, with buses having more impact due to their different weight distribution. As a result, each traffic loading was converted to Equivalent Single Axle Loads (ESAL). This permitted the bus and truck trips to be equated directly. The ESAL values were used in developing the final methodology. The distribution of bus/truck volume and bus/truck ESAL is illustrated in Figures 14–2a and 14–2b.

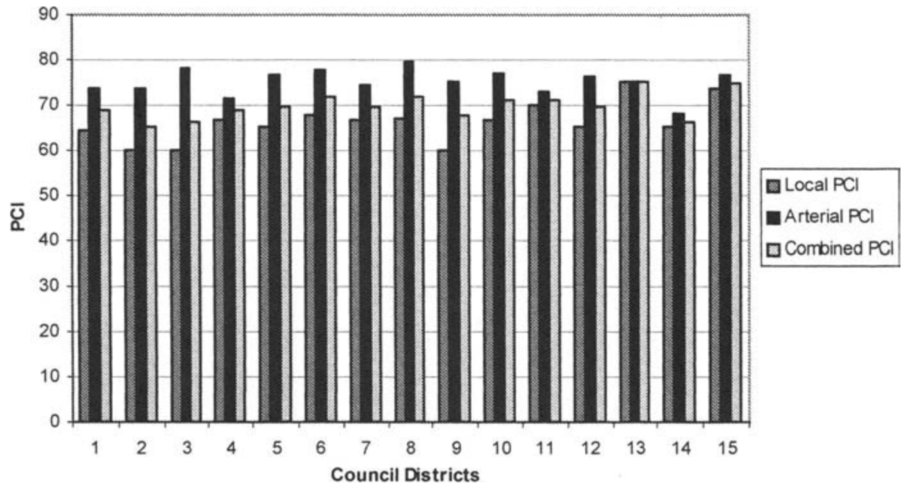


Figure 14-1a. PCI by Council District.

Council District	Local PCI	Arterial PCI	Combined PCI
1	64.28	73.8	68.7
2	60.17	73.7	65.3
3	60.10	78.2	66.4
4	66.80	71.4	68.9
5	65.08	76.7	69.8
6	67.85	77.6	71.9
7	66.61	74.5	69.6
8	67.18	79.8	71.8
9	59.88	75.1	67.9
10	66.68	77.1	71.2
11	69.84	73.1	71.0
12	65.35	76.2	69.5
13	75.04	75.2	75.1
14	65.09	68.1	66.3
15	73.72	76.5	74.8
Area Weighted Avg PCI	**65.8**	**75.2**	**69.6**

Figure 14-1b. PCI by Council District..

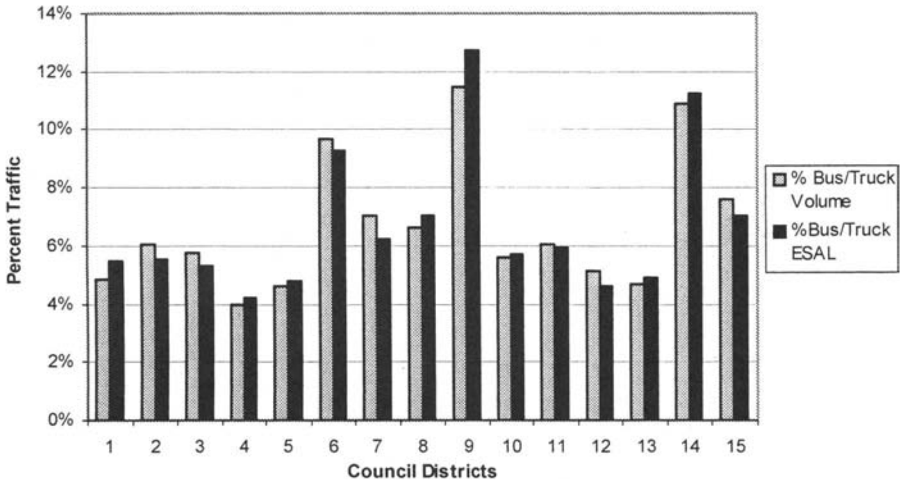


Figure 14-2a. Bus/Truck Traffic by Council District.

Council District	% Bus/Truck Volume	%Bus/Truck ESAL
1	4.81%	5.48%
2	6.04%	5.55%
3	5.77%	5.33%
4	3.98%	4.20%
5	4.58%	4.81%
6	9.67%	9.29%
7	7.04%	6.23%
8	6.65%	7.02%
9	11.49%	12.71%
10	5.59%	5.72%
11	6.05%	5.93%
12	5.15%	4.58%
13	4.68%	4.88%
14	10.88%	11.21%
15	7.61%	7.05%
Total	100.00%	100.00%

Figure 14-2b. Bus/Truck Traffic by Council District.

14.4 Development of Budget Allocation Models

Three factors were considered in developing the budget allocation formula: pavement area, pavement condition, and truck and bus traffic. The area factor was considered due to the varying sizes of the pavement areas (or lane miles) in council districts. The condition factor was included to address the need for pavement resurfacing and reconstruction due to deteriorated conditions in a particular district and to take into account user comfort. The traffic factor was included to address the effects of buses/trucks on increased pavement thickness and thus the cost of resurfacing and reconstruction. The following approach was used in the model development,

14.4.1

The primary objective of the budget allocation formula was to reach an optimum condition level among all council districts. Since the districts had not reached this goal, it was critical to use their current conditions to assess budget allocation to best reach their goals as a unit.

14.4.2

The critical condition value, or the value at which pavement should be economically considered for resurfacing in the City, was determined to be about 60. Therefore, the weighted average PCI for a municipality was about 80 (average condition between 60 and 100). This is due to the fact that not all of the pavements would be 100, but all of them should be above the critical value of 60. As a result, a condition credit would be given based on the weighted average PCI of a district. The Condition Credit (CC) for a district is shown in Figure 14–3 and was computed as follows:

$$CC = 80 - \text{Area Weighted Average PCI for the district} \quad (14\text{-}1)$$

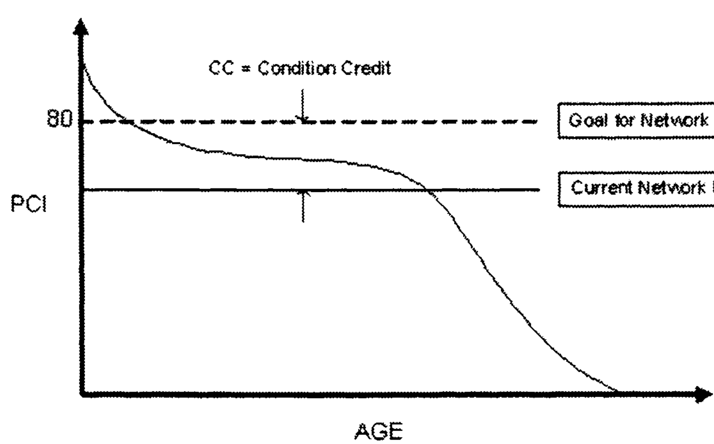


Figure 14-3. Condition Credit Concept.

14.4.3

Another consideration in budget allocation was the pavement area in each district. Two districts with the same condition but different pavement areas should be funded in proportion to their areas (assuming traffic was the same). It was therefore clear that condition and area should interact to provide a rational factor for budget allocation. The Condition Area Factor (CAF) for a district was calculated as:

$$\text{CAF} = \frac{(\text{Condition Credit} + 1) \times (\%\ \text{Area})}{\sum [(\text{Condition Credit} + 1) \times (\%\ \text{Area})]\ \text{for all districts}} \tag{14-2}$$

The reason for adding 1 to the Condition Credit was that if a district had an average PCI of 80 or higher, the Condition Credit would be zero. So in the case where all districts had reached the PCI goal of 80, the above formula would result in budget allocation proportional to the district areas.

14.4.4

A factor for vehicle traffic was considered as well. If all else were equal (i.e., condition and area), pavements subjected to heavier traffic would require more funds to account for higher costs of rehabilitation such as thicker overlays. The traffic effect was considered through the ESAL values computed from the bus/truck volumes. This factor will be referred to here as the Bus/Truck Factor (BTF) which was computed as follows:

$$\text{BTF} = \frac{(\text{ESAL for the district})}{\sum (\text{ESAL for all districts})} \tag{14-3}$$

The effect of traffic would eventually be reflected in the condition/area factor (CAF), but inclusion of the traffic factor minimized the fluctuations until that was achieved. As a result, the traffic factor should have been included, but it deserved a lower proportion in the allocation model.

Three models for allocation of rehabilitation funds were considered in the analysis. The first model was weighted 80% for CAF and 20% for BTF.

$$\text{Model 1: Budget Allocation} = 80\% \times \text{CAF} + 20\% \times \text{BTF} \tag{14-4}$$

Model 1 represented the most aggressive approach to budget allocation. The drawback of this model was that some districts would receive significantly less money than the others due primarily to their relatively good current conditions. To address this problem, two other models were developed. The second model was weighted 60% for CAF, 20% for BTF, and 20% for area.

$$\text{Model 2: Budget Allocation} = 60\% \times \text{CAF} + 20\% \times \text{BTF} + 20\% \times \text{Area Ratio} \tag{14-5}$$

The purpose for introducing the stand-alone area component was to smooth out the sharp adjustment that some districts may have realized with Model 1. However, this model would take longer than Model 1 toward achieving the goal of equal condition among all districts. Model 3 was weighted 40% for CAF, 10% for BTF, and 50% for area. This model gives more credit to the size of each district and further reduces drastic adjustments in budget allocation. This model was the least aggressive among the three models, and thus would take the longest toward achieving the goal of equal condition among all districts.

$$\text{Model 3: Budget Allocation} = 40\% \times \text{CAF} + 10\% \times \text{BTF} + 50\% \times \text{Area Ratio} \tag{14-6}$$

14.5 Budget Allocation Models Analysis

14.5.1 Model 1

In this model 80% of the weight was assigned to the CAF. Therefore, the model is more favorable to council districts with lower PCI values. This is illustrated in Figures 14–4a and 14–4b for arterial and local roads, respectively. For arterial roads, District 8 would be receiving much less of the budget than District 14. This was primarily because District 14's PCI was 68.1 as compared to District 8's PCI of 79.8. For local roads, District 9 would benefit from Model 1 while District 11 would receive much less than its percentage of lane miles.

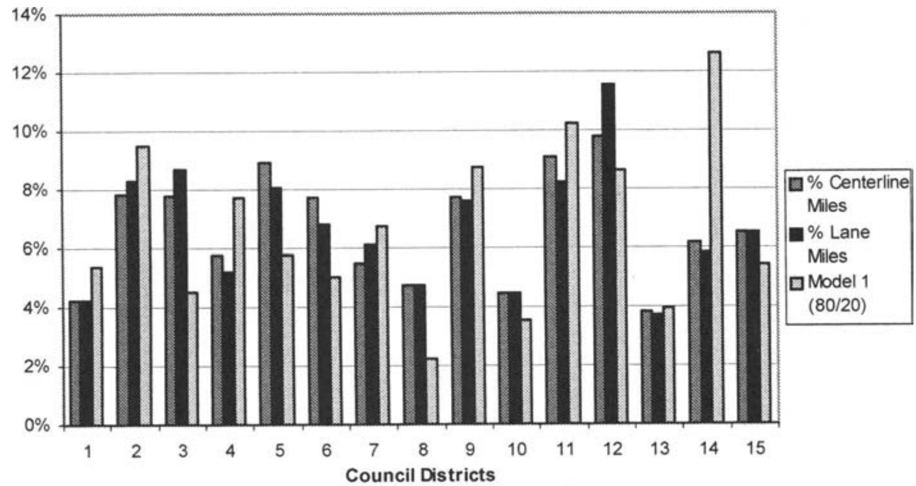


Figure 14-4a. Budget Allocation for Arterial Roads–Model 1.

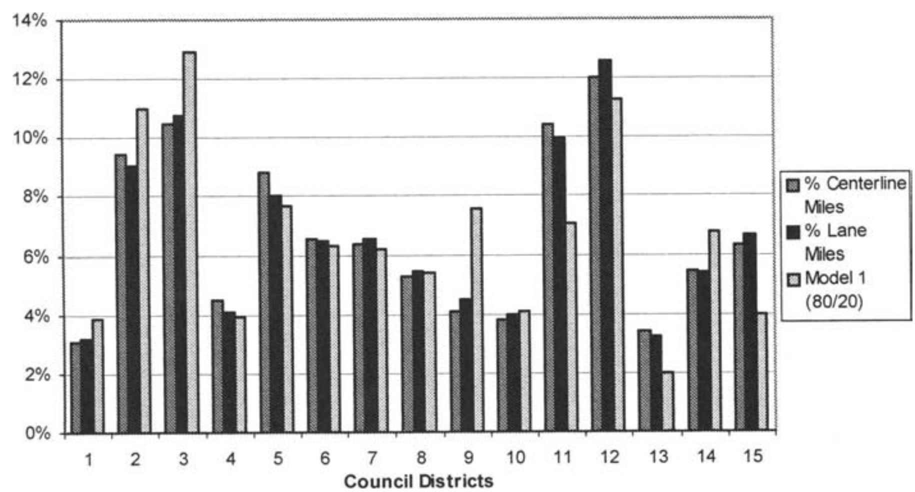


Figure 14-4b. Budget Allocation for Local Roads–Model 1.

14.5.2 Model 2

As indicated before, the objective of adding a stand-alone area component in Model 2 (as compared to Model 1) was to reduce the immediate impact of condition toward achieving the ultimate goal of equal condition among all districts. The results of Model 2 are presented in Figures 14–5a and 14–5b for arterial and local roads respectively. For example, District 8 arterial roads would receive 2.96 percent of the budget as compared to 2.21 percent with Model 1. Similar observations can be made for local roads.

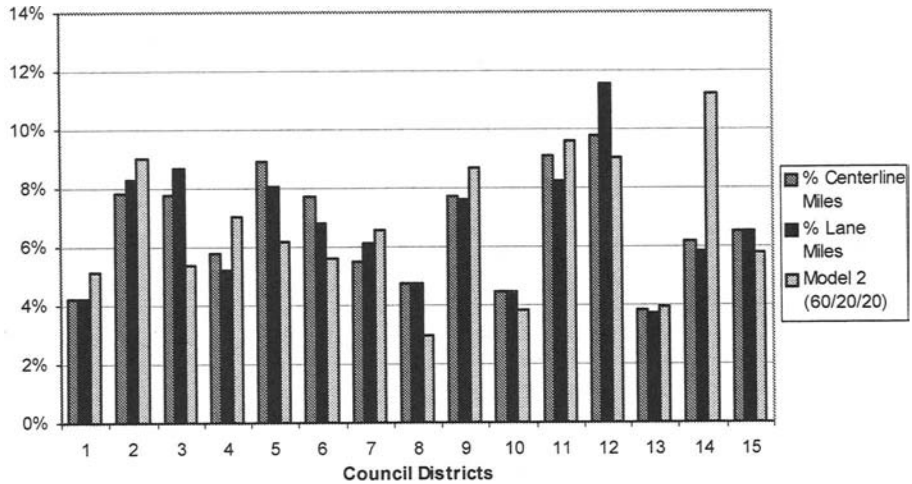


Figure 14-5a. Budget Allocation for Arterial Roads–Model 2.

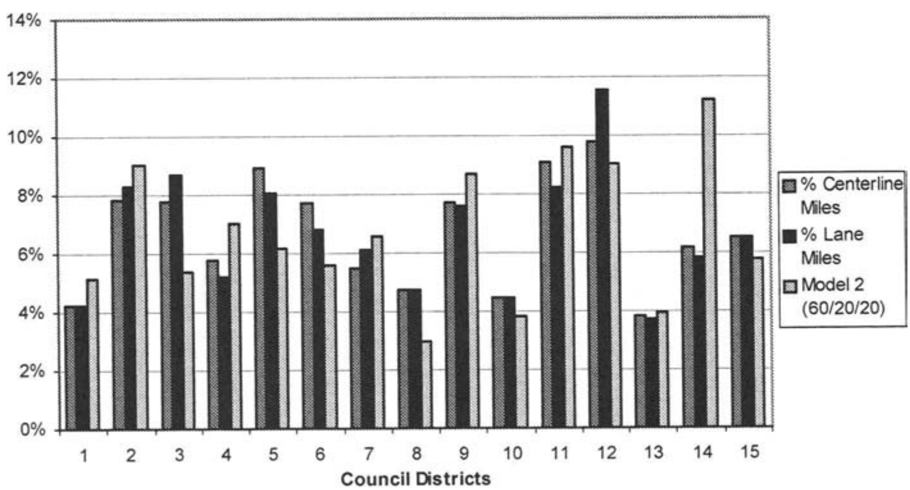


Figure 14-5b. Budget Allocation for Local Roads–Model 2.

14.5.3 Model 3

This model gives 50% weight to area and the other 50% to CAF and BTF. This redistribution further smoothes the districts' budget allocation array toward achieving the goal of equal condition. The results for this model are presented in Figures 14–6a and 14–6b for arterial and local streets, respectively. For example, District 8's budget will be 3.48 percent as compared to 2.96 percent for Model 2 and 2.21 percent for Model 1.

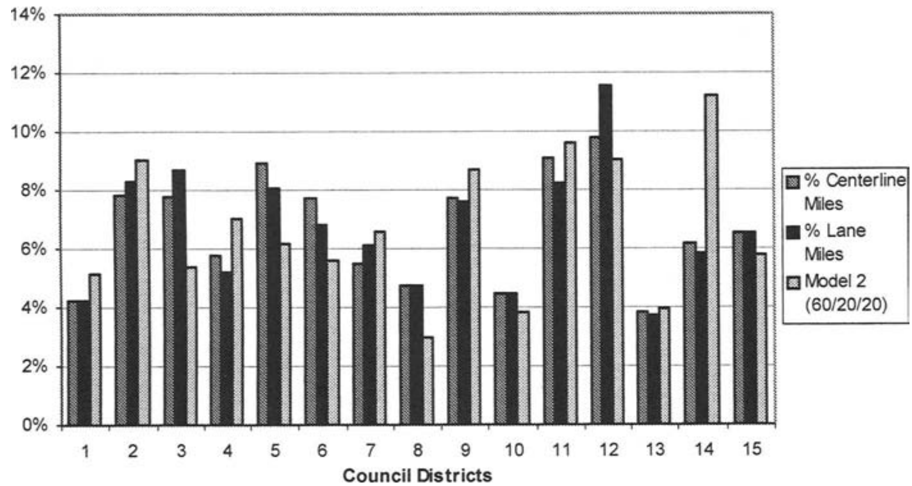


Figure 14-6a. Budget Allocation for Arterial Roads–Model 3.

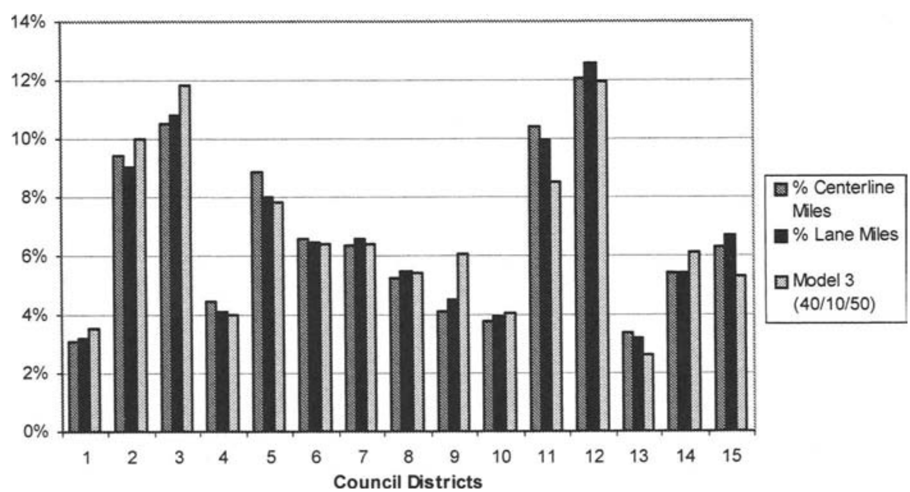


Figure 14-6b. Budget Allocation for Local Roads–Model 3.

14.6 Summary and Conclusions

The developed models included all variables of concern to the council district members. All models worked toward achieving the goal of equal condition among the districts. The percentage of budget allocation among the districts will have to be calculated annually to reflect the change in condition. Since all pavements are not inspected at the same time, the calculations should be based on the projected condition at a selected date. The inspection of all city streets on a 3-year cycle will lead to an acceptable level of accuracy. A comparison of the three models is shown in Figures 14–7a and 14–7b for arterial roads and in Figures 14–8a and 14–8b for local roads. The selection of a model required approval by city council members.

Council District	%Centerline Miles	%Lane Miles	Model 1 (80/20)	Model 2 (60/20/20)	Model 3 (40/10/50)
1	4.25%	4.23%	5.35%	5.13%	4.79%
2	7.83%	8.27%	9.49%	9.05%	8.88%
3	7.76%	8.70%	4.49%	5.37%	6.59%
4	5.80%	5.17%	7.70%	7.02%	6.43%
5	8.91%	8.08%	5.77%	6.18%	6.92%
6	7.71%	6.80%	5.03%	5.59%	5.91%
7	5.46%	6.09%	6.75%	6.60%	6.42%
8	4.72%	4.75%	2.21%	2.96%	3.48%
9	7.73%	7.58%	8.73%	8.70%	8.15%
10	4.45%	4.48%	3.55%	3.85%	4.01%
11	9.07%	8.25%	10.21%	9.61%	9.23%
12	9.75%	11.55%	8.66%	9.03%	10.10%
13	3.84%	3.71%	3.97%	3.96%	3.84%
14	6.20%	5.82%	12.65%	11.21%	9.23%
15	6.53%	6.52%	5.46%	5.75%	5.99%
Total	100.00%	100.00%	100.00%	100.00%	100.00%

Figure 14-7a. Budget Allocation for Arterial Roads.

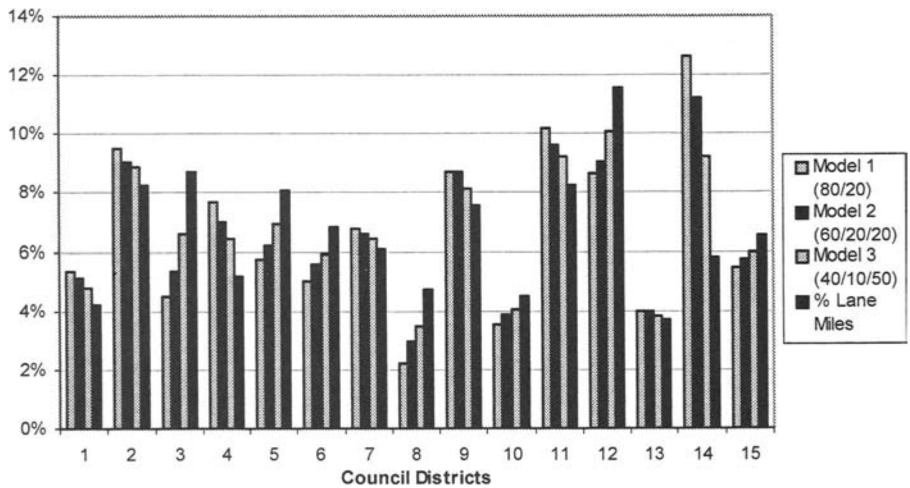


Figure 14-7b. Budget Allocation for Arterial Roads–Combined.

Council District	%Centerline Miles	%Lane Miles	Model 1 (80/20)	Model 2 (60/20/20)	Model 3 (40/10/50)
1	3.09%	3.18%	3.89%	3.82%	3.53%
2	9.45%	9.03%	10.99%	10.33%	10.01%
3	10.49%	10.78%	12.90%	12.10%	11.84%
4	4.47%	4.12%	3.91%	3.97%	4.01%
5	8.83%	8.02%	7.67%	7.60%	7.85%
6	6.57%	6.47%	6.33%	6.51%	6.40%
7	6.35%	6.56%	6.21%	6.28%	6.39%
8	5.28%	5.48%	5.38%	5.49%	5.43%
9	4.09%	4.51%	7.55%	7.20%	6.03%
10	3.80%	3.96%	4.12%	4.17%	4.04%
11	10.40%	9.97%	7.03%	7.56%	8.50%
12	12.03%	12.60%	11.27%	11.20%	11.94%
13	3.40%	3.22%	1.99%	2.38%	2.60%
14	5.45%	5.43%	6.79%	6.74%	6.11%
15	6.31%	6.67%	3.96%	4.66%	5.32%
Total	100.00%	100.00%	100.00%	100.00%	100.00%

Figure 14-8a. Budget Allocation for Local Roads.

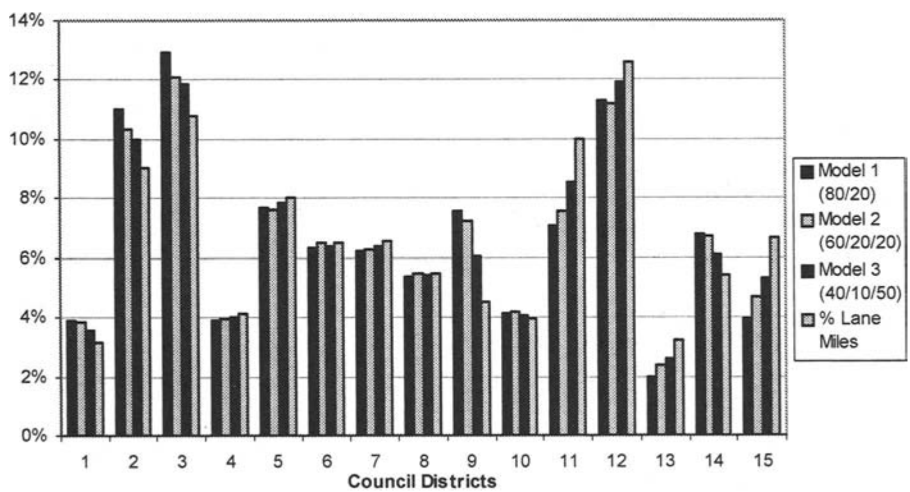


Figure 14-8b. Budget Allocation for Local Roads–Combined.

15

Pavement Management Implementation Steps and Expected Benefits

The information presented in this chapter is directly applicable to the Micro PAVER pavement management system (Micro PAVER 2004). The majority of the information is also applicable to other management systems.

15.1 Pavement Management Implementation Steps

Figure 15–1 shows a summary of the implementation steps. Following is a description of each of the steps.

1. *Obtain map*; a geo-referenced GIS map is best if available. This allows for the use of the same map to show other infrastructure facilities where all facilities will show in correct spatial reference to each other. AutoCad maps can also be used but they are not likely to be geo-referenced.
2. *Define network(s)*; the map is broken into networks, branches, and sections. Agency staff should participate heavily in this process to insure ownership and successful use of the implemented system.
3. *Collect inventory data*; the collected data should be kept to what is essential for the operation of the management system. Such data include:

- Branch use
- Section length, width, and area
- Section surface type
- Section rank (functional classification)
- Section last construction date, LCD (date of last major M&R)

The LCD is one of the most important pieces of information and also the most difficult to obtain. The LCD is essential for developing condition deterioration models and performing condition analysis and work planning. If the LCD is

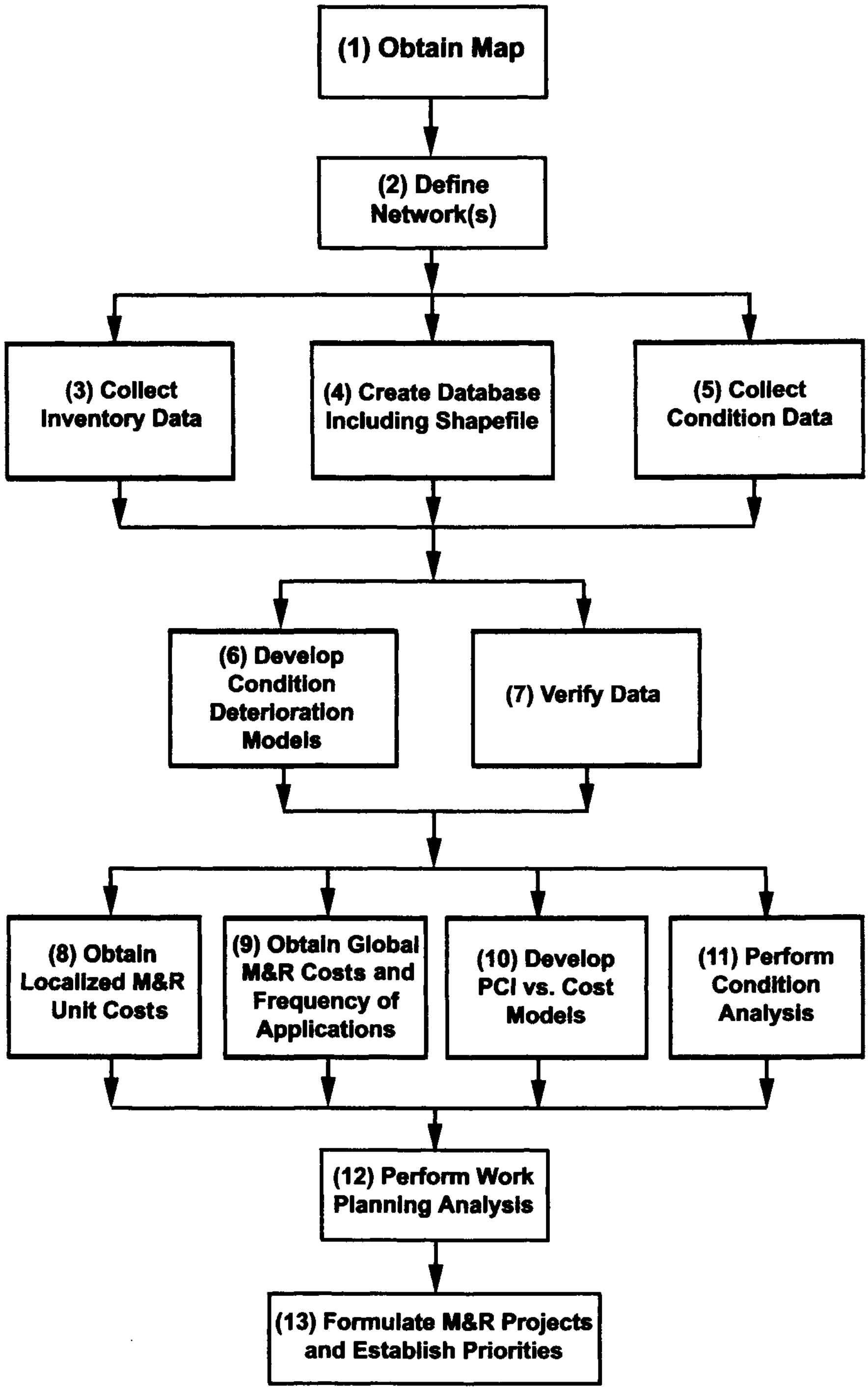


Figure 15-1. Time Sequence of the Implementation Steps.

not known, it can be estimated from the PCI when it is completed. This can be done by assuming a rate of deterioration or a representative condition deterioration curve that is appropriate for the pavement construction, climate, and traffic.

4. *Create database including Shape file*; the defined network(s) and associated inventory data are used to create the system's database. The map showing the pavement sections is converted into a GIS shape file to be used for data entry and presentations.

5. *Collect condition data; at a minimum,* the collected condition data should include distress. Other condition data that may be collected at different intervals include roughness, skid, and structural. Enter the data into the database.

6. *Develop condition deterioration models;* the primary models developed in Micro PAVER are for PCI vs. Age. The quality of the models depends on several factors, the most important of which is the knowledge of the LCD for each pavement section. When the LCD is not known for a section, it can be estimated by assuming a given rate of deterioration or a representative condition deterioration curve.

7. *Verify data*; all entered data should be verified for accuracy and reasonableness. One of the useful tools is a scatter plot of PCI vs. age for each of the pavement families. Fig.15–2 shows a PCI vs. Age scatter plot for asphalt taxiways of a civil aviation airport. From the figure one can observe that at the age of 40, the PCI ranges from 25 to 100. Such a stack of points normally indicates an erroneous LCD.

8. *Obtain localized M&R unit costs*; unit cost of localized repair is needed to develop the first-year localized M&R program and to determine the consequence of applying different levels of localized repair on PCI and cost. Example work types include crack sealing and patching. For some work types, like patching, the unit costs will vary based on pavement type (i.e. asphalt vs. concrete), use (i.e. runway vs. apron), and rank (i.e. arterial vs. residential).

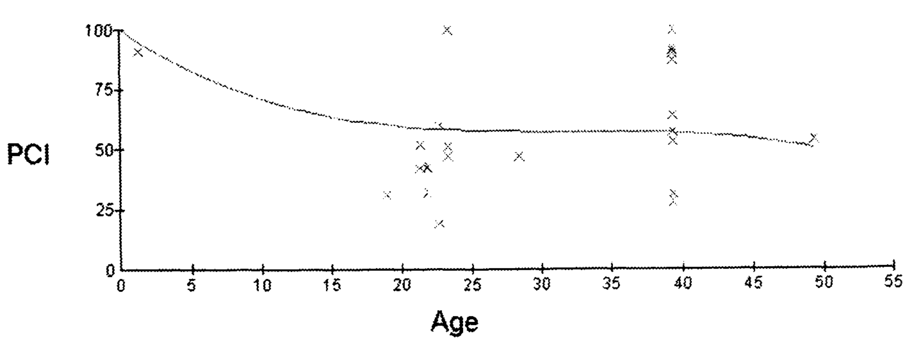


Figure 15-2. Example PCI vs. Age Scatter Plot.

9. *Obtain global M&R unit costs and frequency of applications*; surface treatments are beneficial when applied correctly and at the right time. To be included in work planning, unit costs should be obtained for the different surface treatments to be used. Also to be determined is the application frequency of each of the surface treatment types.

10. *Develop PCI vs. cost models*; the PCI vs. cost curves need to be developed for localized safety M&R, localized preventive M&R, and major M&R. More than one curve of each may be developed to reflect cost dependence on pavement type, use, and rank.

11. *Perform condition analysis*; condition analysis is performed to show past, current and future condition (assuming only stop-gap M&R). The presentation can be made using line graphs, bar charts, and maps.

12. *Perform work planning analysis*; work planning is one of the most important components, if not the most important component, of the pavement management system. It provides the ability to:

- Determine annual localized work requirements, i.e. crack sealing and patching
- Determine optimum M&R category (i.e. localized, global, and major) for each pavement section for each year of the analysis for a given budget.
- Determine the consequence of different budget levels on pavement condition and backlog of major M&R.
- Determine budget requirements to meet specified management objectives. Typical management objectives include maintaining current network condition, reaching a certain condition in x years, or eliminating all backlogs of major M&R in x years.

13. *Formulate M&R projects and establish priorities*; the knowledge generated from the work plan is used to formulate projects where each project is likely to include more than one pavement section and may include more than one work type. Projects can also be generated based on other requirements beside economics as management sees necessary.

15.2 Benefits of Implementing a Pavement Management System

The following benefits were stated by several agencies who have implemented pavement management systems (Shahin et al. 2003):

1. Provide necessary data to legislators and managers for budget determination.
2. Maximize the return on investment from available M&R budget.
3. Create a prioritized 5-year plan.
4. Establish minimum condition requirements.
5. Identify areas in need of maintenance.
6. Justify M&R projects.
7. Criterion for distribution of available budget among various networks (i.e., airports, council districts, etc.)

References

U.S. Army Engineering Research and Development Center-Construction Engineering Research Laboratory (ERDC-CERL). 2004. Micro PAVER Pavement Management System, 2004. e-mail: paver@cecer.army.mil web: www.cecr.army.mil/paver

Shahin, M.Y., Doll, A., Lange G., Nilsson, F.G., and Tasanen, R. 2003. Airport Pavement Management Systems. PIARC World Road Congress, Durban, South Africa. September.

APPENDIX A

Blank Field Survey Sheets

ASPHALT SURFACED ROADS AND PARK. LOTS
CONDITION SURVEY DATA SHEET
FOR SAMPLE UNIT

SKETCH:

BRANCH	DATE
SURVEYED BY	SAMPLE UNIT
SECTION	SAMPLE AREA

1. Alligator Cracking Sq Ft
2. Bleeding Sq Ft
3. Block Cracking Sq Ft
4. Bumps and Sags Ft
5. Corrugation Sq Ft
6. Depression Sq Ft
7. Edge Cracking Ft
8. Jt. Reflection Cracking Ft
9. Lane/Shoulder Drop Off Ft
10. Long & Trans Cracking Ft
11. Patching & Util Cut Patching Sq Ft
12. Polished Aggregate Sq Ft
13. Potholes Count
14. Railroad Crossing Sq Ft
15. Rutting Sq Ft
16. Shoving Sq Ft
17. Slippage Cracking Sq Ft
18. Swell Sq Ft
19. Weathering/Ravelling Sq Ft

DISTRESS SEVERITY	QUANTITY								TOTAL	DENSITY %

Figure A-1. Asphalt Surfaced Roads and Parking Lot Condition Survey Data Sheet for Sample Unit.

CONCRETE SURFACED ROADS AND PARKING LOTS CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT

BRANCH	SECTION	SAMPLE UNIT
SURVEYED BY	DATE	SAMPLE AREA

DISTRESS TYPES

21. Blow up/Buckling	31. Polished Aggregate
22. Corner Break	32. Popout
23. Divided Slab	33. Pumping
24. Durability Crack	34. Punchout
25. Faulting	35. Railroad Crossing
26. Joint Seal	36. Scaling
27. Lane/Shoulder	37. Shrinkage
28. Linear Cracking	38. Spalling Corner
29. Patching (Large)	39. Spalling Joint
30. Patching (Small)	

DIST TYPE	SEV	NO. SLABS	DENSITY %	DEDUCT VALUE

SKETCH:

10
9
8
7
6
5
4
3
2
1

1 2 3 4

Figure A-2. Concrete Surfaced Roads and Parking Lot Condition Survey Data Sheet for Sample Unit.

AIRFIELD ASPHALT PAVEMENT CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT

SKETCH:

BRANCH	DATE
SURVEYED BY	SAMPLE UNIT
SECTION	SAMPLE AREA

41. Alligator Cracking Sq Ft	45. Depression Sq Ft	49. Oil Spillage Sq Ft	53. Rutting Sq Ft
42. Bleeding Sq Ft	46. Jet Blast Sq Ft	50. Patching Sq Ft	54. Shoving from PCC Sq Ft
43. Block Cracking Sq Ft	47. Jt. Reflection (PCC) Ft	51. Polished Aggregate Sq Ft	55. Slippage Cracking Sq Ft
44. Corrugation Sq Ft	48. Long. & Trans. Cracking Ft	52. Raveling/Weathering Sq Ft	56. Swell Sq Ft

DISTRESS SEVERITY	QUANTITY								TOTAL	DENSITY %

Figure A-3. Airfield Asphalt Pavement Condition Survey Data Sheet for Sample Unit.

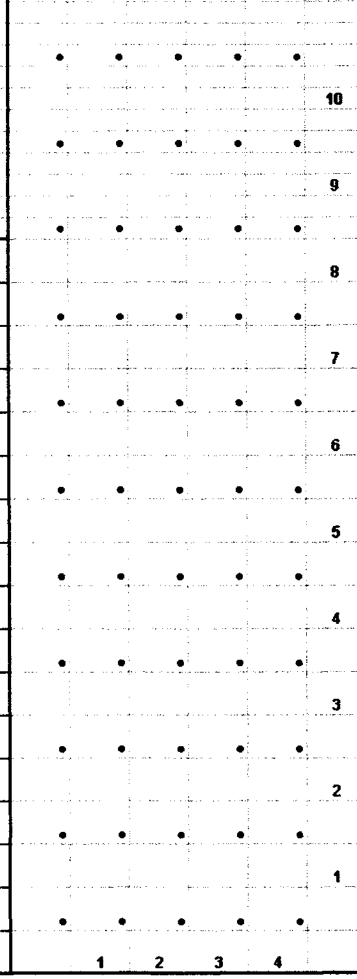
AIRFIELD CONCRETE PAVEMENTS
CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT

BRANCH	SECTION	SAMPLE UNIT
SURVEYED BY	DATE	SAMPLE AREA

DISTRESS TYPES

- 61. Blow up
- 62. Corner Break
- 63. Long/Trans/ Diagonal Crack
- 64. Durability Crack
- 65. Joint Seal Damage
- 66. Patching, Small
- 67. Patching, Large
- 68. Popouts
- 69. Pumping
- 70. Scaling/Map Crack/ Crazing
- 71. Settlement/Fault
- 72. Shattered Slab
- 73. Shrinkage Crack
- 74. Spalling-Joints
- 75. Spalling-Corner

SKETCH:



DIST TYPE	SEV	NO. SLABS	DENSITY %	DEDUCT VALUE

Figure A-4. Airfield Concrete Pavement Condition Survey Data Sheet for Sample Unit.

UNSURFACED ROADS
CONDITION SURVEY DATA SHEET
FOR SAMPLE UNIT

BRANCH	DATE
SURVEYED BY	SAMPLE UNIT
SECTION	SAMPLE AREA

SKETCH:

1. **Improper Cross Section** Linear Ft
2. **Inadequate Roadside Drainage** Sq Ft
3. **Corrugations** Sq Ft
5. **Dust**

13. **Potholes** Count
12. **Ruts** Sq Ft
12. **Loose Aggregate** Linear Ft

DISTRESS SEVERITY	QUANTITY								TOTAL	DENSITY %

Figure A-5. Unsurfaced Road Condition Survey Data Sheet for Sample Unit.

No.	DEDUCT VALUES												TOTAL	q	CDV
1															
2															
3															
4															
5															
6															
7															
8															
9															
10															

Figure A-6. Deduct value calculation sheet.

Ride Quality

When performing the distress survey, ride quality must be evaluated to determine the severity level of some of the distresses such as corrugation and railroad crossing. The following is provided as a general guideline to help establish the degree of severity of the ride quality:

1. *L (low).* Vehicle vibrations (e.g., from corrugation) are noticeable, but no reduction in speed is necessary for comfort or safety, and/or individual bumps or settlements cause the vehicle to bounce slightly, but create little discomfort.
2. *M (medium).* Vehicle vibrations are significant and some reduction in speed is necessary for safety and comfort, and/or individual bumps or settlements cause the vehicle to bounce significantly, creating some discomfort.
3. *H (high).* Vehicle vibrations are so excessive that speed must be reduced considerably for safety and comfort, and/or individual bumps or settlements cause the vehicle to bounce excessively, creating substantial discomfort, and/or a safety and/or high potential vehicle damage.

Ride quality is determined by riding in a standard-sized automobile over the pavement section at the posted speed limit. Pavement sections near stop signs should be rated at the normal deceleration speed used when approaching the sign.

APPENDIX B

Asphalt Concrete Roads: Distress Definitions and Deduct Value Curves

Alligator Cracking (01)

Description

Alligator or fatigue cracking is a series of interconnecting cracks caused by fatigue failure of the asphalt concrete surface under repeated traffic loading. Cracking begins at the bottom of the asphalt surface (or stabilized base) where tensile stress and strain are highest under a wheel load. The cracks propagate to the surface initially as a series of parallel longitudinal cracks. After repeated traffic loading, the cracks connect, forming many sided, sharp-angled pieces that develop a pattern resembling chicken wire or the skin of an alligator. The pieces are generally less than 2ft (0.6 m) on the longest side.

Alligator cracking occurs only in areas subjected to repeated traffic loading, such as wheel paths. Therefore, it would not occur over an entire area unless the entire area was subjected to traffic loading. (Pattern-type cracking that occurs over an entire area not subjected to loading is called "block cracking," which is not a load-associated distress.)

Alligator cracking is considered a major structural distress and is usually accompanied by rutting.

Severity Levels (Fig.B–1)

L—Fine, longitudinal hairline cracks running parallel to each other with no, or only a few interconnecting cracks. The cracks are not spalled[a].

M—Further development of light alligator cracks into a pattern or network of cracks that may be lightly spalled.

H—Network or pattern cracking has progressed so that the pieces are well defined and spalled at the edges. Some of the pieces may rock under traffic.

How to Measure

Alligator cracking is measured in square feet of surface area. The major difficulty in measuring this type of distress is that two or three levels of severity often exist within one distressed area. If these portions can be easily distinguished from each other, they should be measured and recorded separately. However, if the different levels of severity cannot be divided easily, the entire area should be rated at the highest severity present. If alligator cracking and rutting occur in the same area, each is recorded separately as its respective severity level.

Options for Repair

L—Do nothing; Surface seal; Overlay.

M—Partial or full-depth patch; Overlay; Reconstruct.

H—Partial or full-depth patch; Overlay; Reconstruct.

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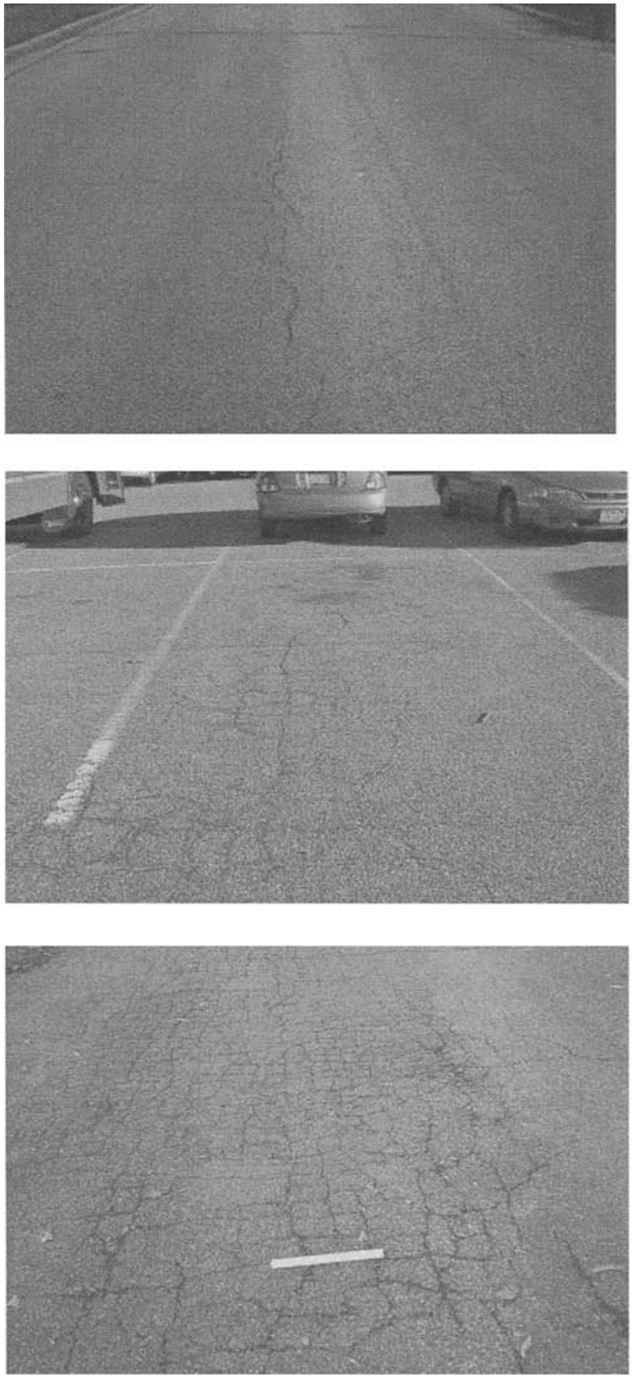
[a]Crack spalling is a breakdown of the material along the sides of the crack.



LOW

MEDIUM

HIGH

Figure B-1. Alligator Cracking.

Bleeding (02)

Description

Bleeding is a film of bituminous material on the pavement surface that creates a shiny, glasslike, reflecting surface that usually becomes quite sticky. Bleeding is caused by excessive amounts of asphaltic cement or tars in the mix, excess application of a bituminous sealant, and/or low air void content. It occurs when asphalt fills the voids of the mix during hot weather and then expands onto the pavement surface. Since the bleeding process in not reversible during cold weather, asphalt or tar will accumulate on the surface.

Severity Levels (Figure B–2)

L—Bleeding has only occurred to a very slight degree and is noticeable only during a few days of the year. Asphalt does not stick to shoes or vehicles.

M—Bleeding has occurred to the extent that asphalt sticks to shoes and vehicles during only a few weeks of the year.

H—Bleeding has occurred extensively and considerable asphalt sticks to shoes and vehicles during at least several weeks of the year.

How to Measure

Bleeding is measured in square feet of surface area. If bleeding is counted, polished aggregate should not be counted.

Options for Repair

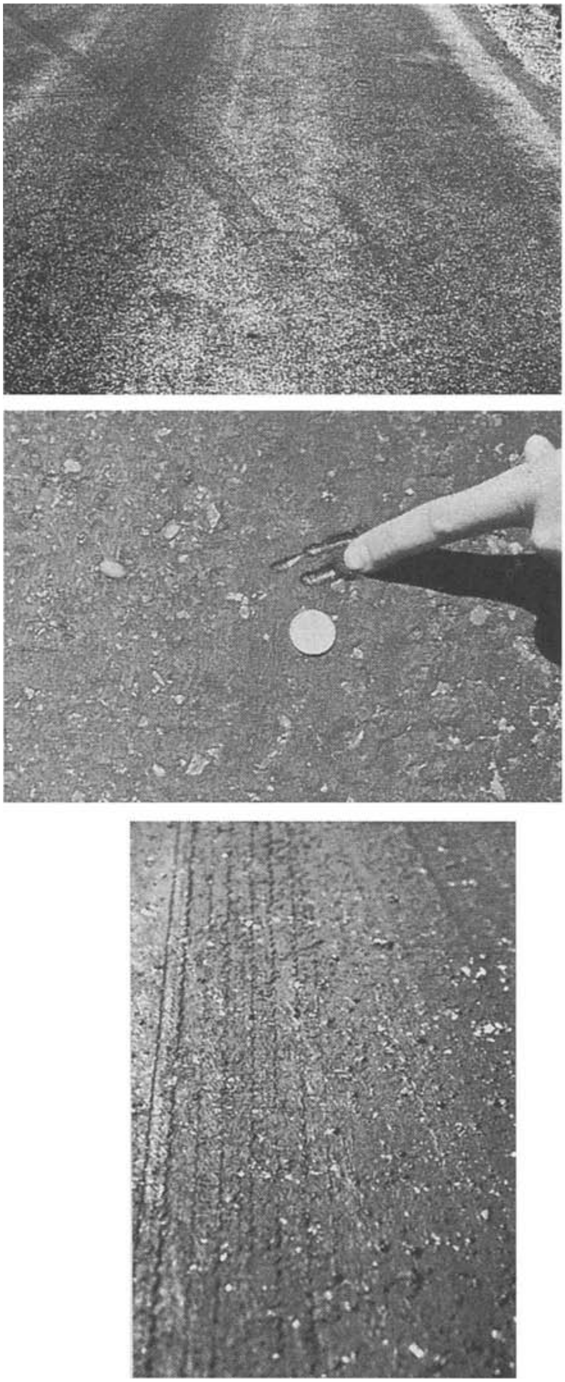
L—Do nothing.

M[a]—Apply sand/aggregate and roll.

H[a]—Apply sand/aggregate and roll.

---

[a]Preheat if necessary.



LOW

MEDIUM

HIGH

Figure B-2. Bleeding.

Block Cracking (03)

Description

Block cracks are interconnected cracks that divide the pavement into approximately rectangular pieces. The blocks may range in size from approximately 1 by 1 ft (0.3 by 0.3 m) to 10 by 10 ft (3 by 3 m). Block cracking is caused mainly by shrinkage of the asphalt concrete and daily temperature cycling (which results in daily stress/strain cycling). It is not load associated. Block cracking usually indicates that the asphalt has hardened significantly. Block cracking normally occurs over a large portion of the pavement area, but sometimes will occur only in non-traffic areas. This type of distress differs from alligator cracking in that alligator cracks form smaller, many sided pieces with sharp angles. Also, unlike block, alligator cracks are caused by repeated traffic loadings, and are therefore found only in traffic areas (i.e., wheel paths).

Severity Levels (Figure B–3)

L—Blocks are defined by low severity[a] cracks.

M—Blocks are defined by medium severity[a] cracks.

H—Blocks are defined by high severity cracks.

How to Measure

Block cracking is measured in square feet of surface area. It usually occurs at one severity level in a given pavement section. However, if areas of different severity levels can be easily distinguished from one another, they should be measured and recorded separately.

Options for Repair

L—Seal cracks over 1/8 in.; Surface seal.

M—Seal cracks; Recycle surface; Heater scarify and overlay.

H—Seal cracks; Recycle surface; Heater scarify and overlay.

---

[a]See definitions of longitudinal transverse cracking.

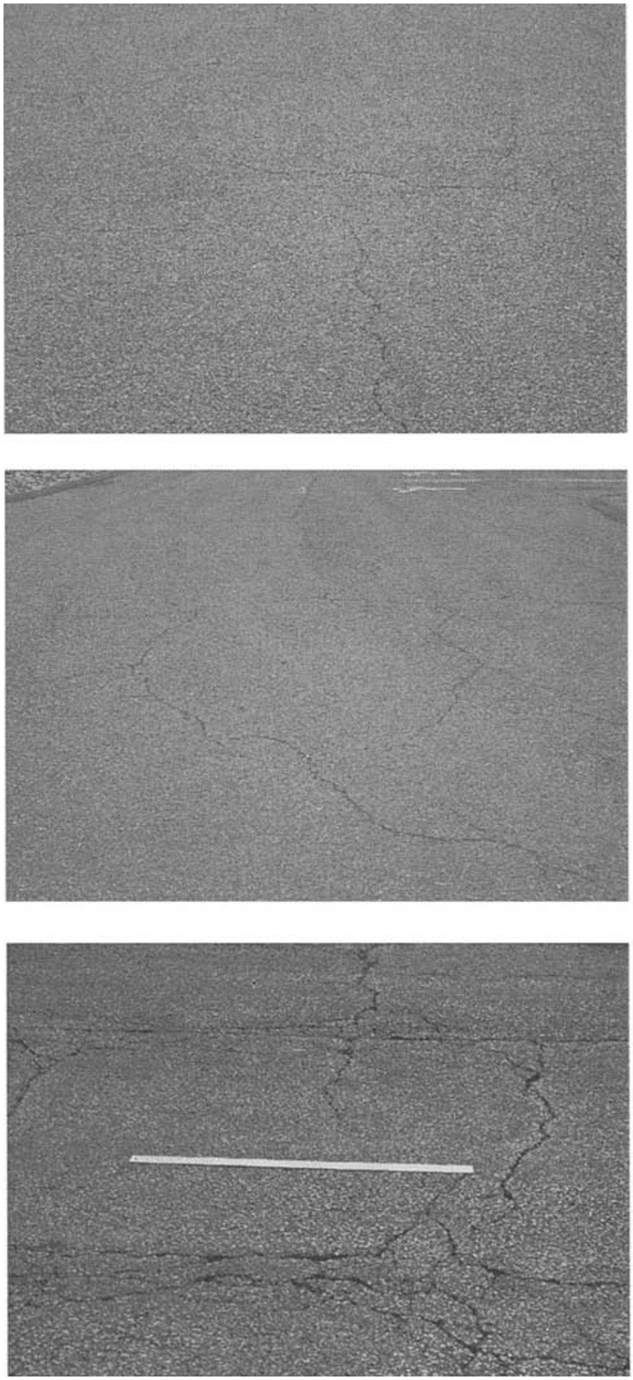


Figure B-3. Block Cracking.

Bumps and Sags (04)

Description

Bumps are small, localized, upward displacements of the pavement surface. They are different from shoves in that shoves are caused by unstable pavement. Bumps, on the other hand, can be caused by several factors, including:

1. Buckling or bulging of underlying PCC slabs in AC overlay over PCC pavement.
2. Frost heave (ice, lens growth).
3. Infiltration and buildup of material in a crack in combination with traffic loading (sometimes called "tenting").

Sags are small, abrupt, downward displacements of the pavement surface. Distortion and displacement that occur over large areas of the pavement surface, causing large and/or long dips in the pavement should be recorded as "swelling."

Severity Levels (Figure B–4)

L—Bump or sag causes low severity ride quality.

M—Bump or sag causes medium severity ride quality.

H—Bump or sag causes high severity ride quality.

How to Measure

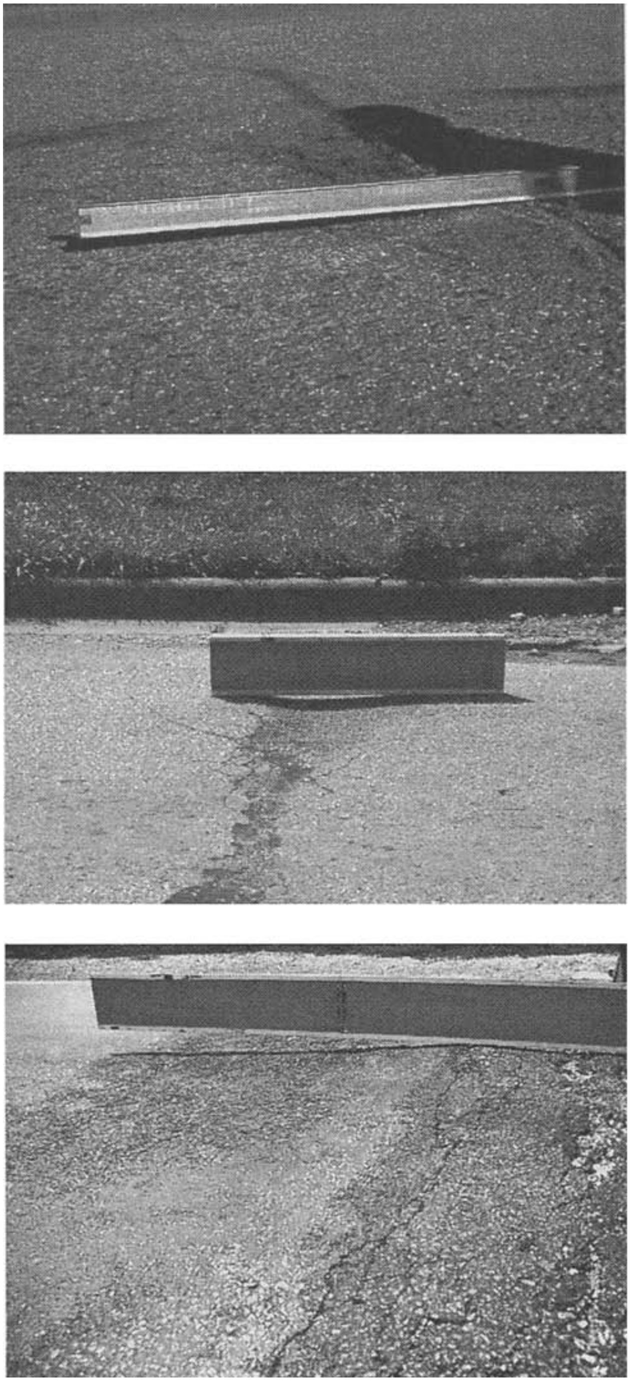
Bumps or sags are measured in linear feet. If bumps appear in a pattern perpendicular to traffic flow and are spaced at <10 ft (3 m), the distress is called corrugation. If the bump occurs in combination with a crack, the crack is also recorded.

Options for Repair

L—Do nothing.

M—Cold mill; Shallow, partial or full-length patch.

H—Cold mill; Shallow, partial or full-depth patch; Overlay.



LOW

MEDIUM

HIGH

Figure B-4. Bumps and Sags.

Corrugation (05)

Description

Corrugation (also known as "washboarding") is a series of closely spaced ridges and valleys (ripples) occurring at fairly regular intervals, usually < 10 ft (3 m) along the pavement. The ridges are perpendicular to the traffic direction. This type of distress is usually caused by traffic action combined with an unstable pavement surface or base. If bumps occur in a series of < 10 ft (3 m), due to any cause, the distress is considered corrugation.

Severity Levels (Figure B–5)

L—Corrugation produces low severity ride quality.

M—Corrugation produces medium-severity ride quality.

H—Corrugation produces high-severity ride quality.

How to Measure

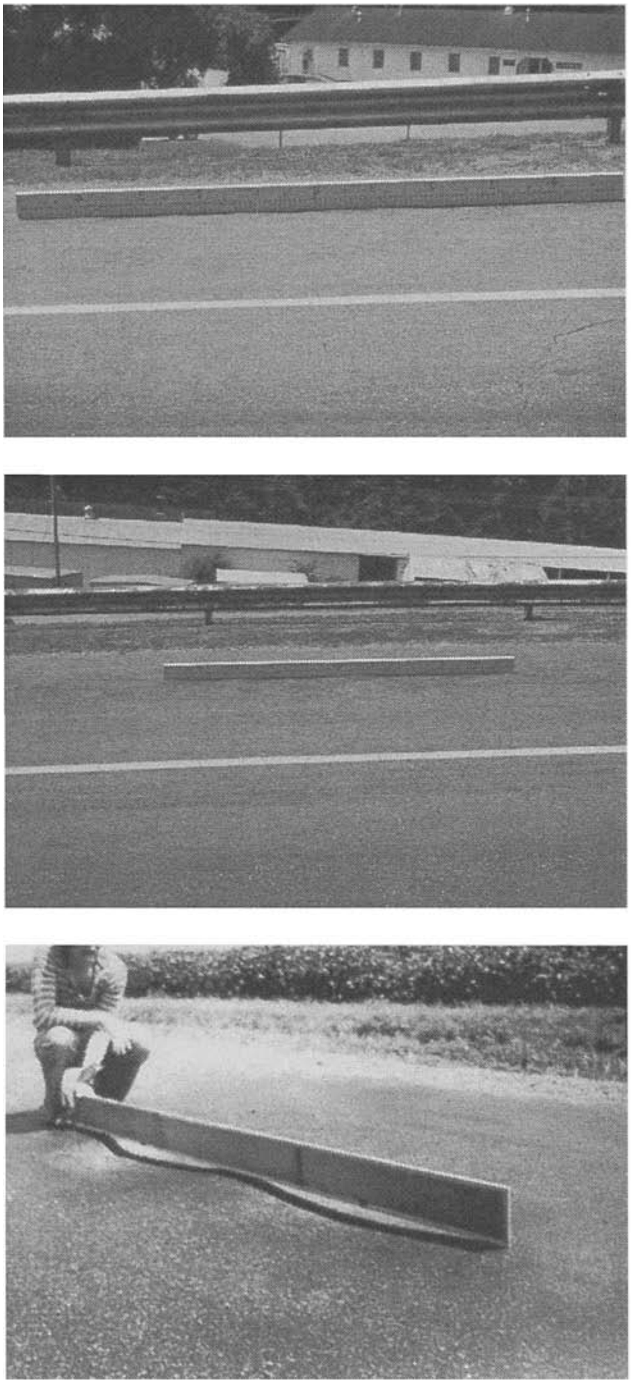
Corrugation is measured in square meters (feet) of surface area.

Options for Repair

L—Do nothing.

M—Reconstruct.

H—Reconstruct.



LOW

MEDIUM

HIGH

Figure B-5. Corrugation.

Depression (06)

Description

Depressions are localized pavement surface areas with elevations slightly lower than those of the surrounding pavement. In many instances, light depressions are not noticeable until after a rain, when ponding water creates a "birdbath" area; on dry pavement, depressions can be spotted by looking for stains caused by ponding water. Depressions are created by settlement of the foundation soil or are a result of improper construction. Depressions cause some roughness, and when deep enough or filled with water, can cause hydroplaning.

Sags, unlike depressions, are abrupt drops in elevation.

Severity Levels (Figure B–6)

Maximum Depth of Depression

L—13 to 25 mm (1/2 to 1 in.).

M—25 to 50 mm (1 to 2 in.).

H—more than 50 mm (2 in.).

How to Measure

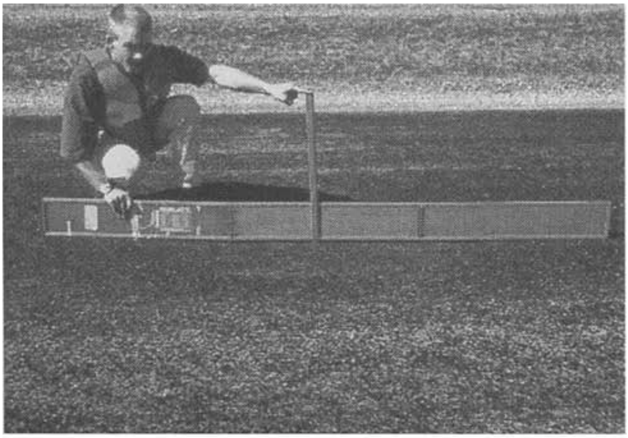
Depressions are measured in square meters (feet) of surface area.

Options for Repair

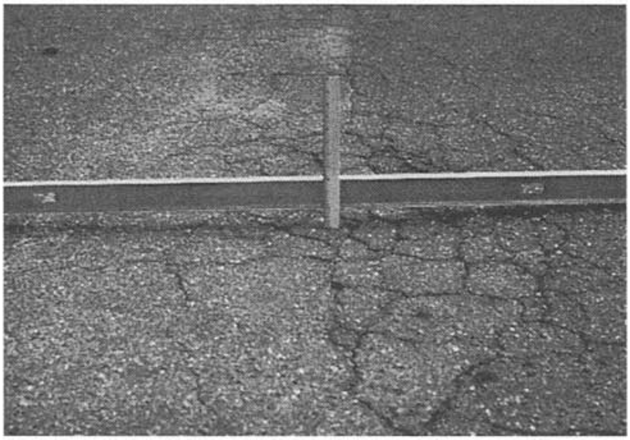
L—Do nothing.

M—Shallow, partial, or full-depth patch.

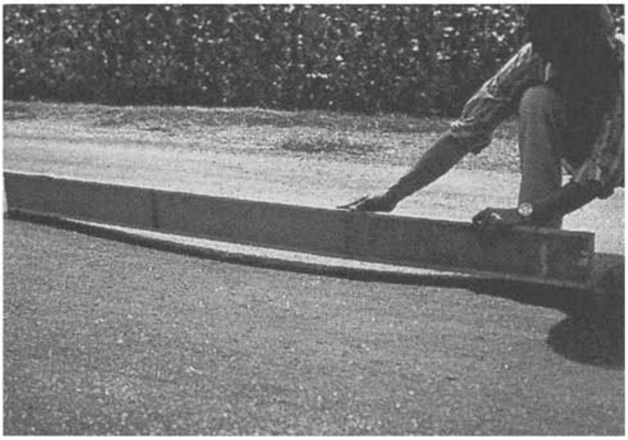
H—Shallow, partial, or full-depth patch.



LOW



MEDIUM



HIGH

Figure B-6. Depression.

Edge Cracking (07)

Description

Edge cracks are parallel to and usually within 1 to 2 ft (0.3 to 0.6 m) of the outer edge of the pavement. This distress is accelerated by traffic loading and can be caused by frost weakened base or subgrade near the edge of the pavement. The area between the crack and pavement edge is classified as raveled if it is broken up (sometimes to the extent that pieces are removed).

Severity Levels (Figure B–7)

L—Low or medium cracking with no breakup or raveling.

M—Medium cracks with some breakup and raveling.

H—Considerable breakup or raveling along the edge.

How to Measure

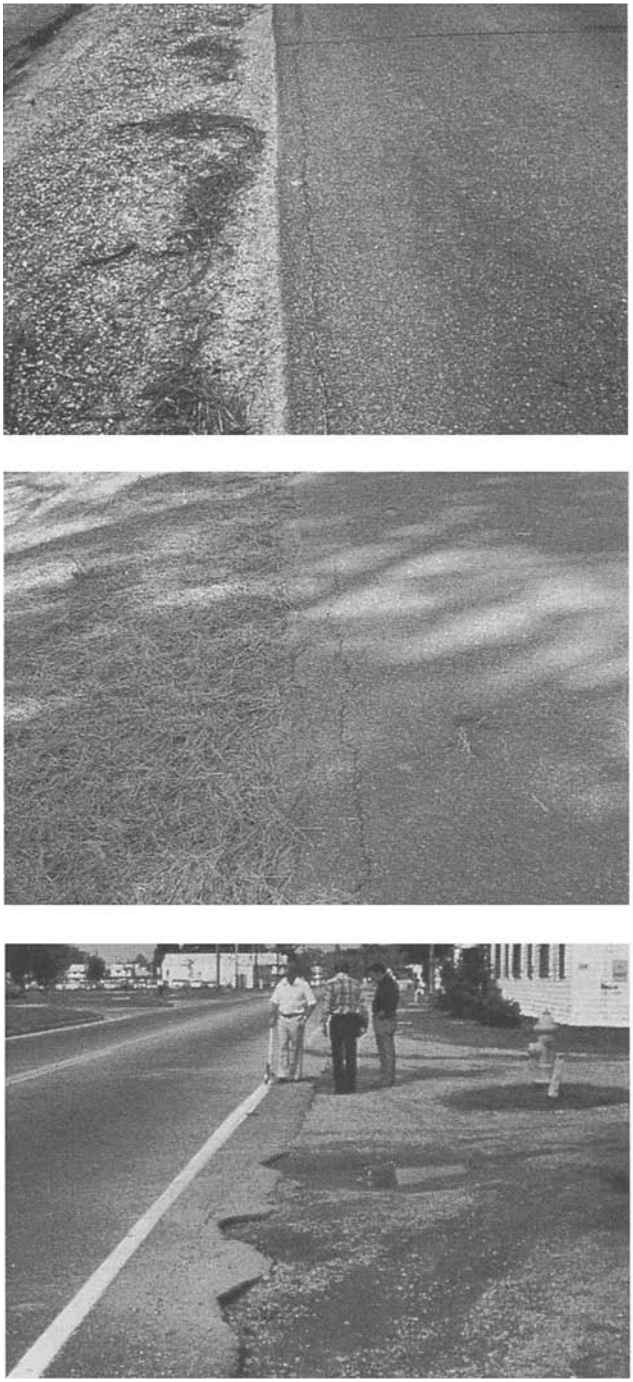
Edge cracking is measure in linear feet.

Options for Repair

L—Do nothing; Seal cracks over 1/8 in. (3 mm).

M—Seal cracks; Partial-depth patch.

H—Partial-depth patch.



LOW

MEDIUM

HIGH

Figure B-7. Edge Cracking.

Joint Reflection Cracking (08) (From Longitudinal and Transverse PCC Slabs)

Description

This distress occurs only on asphalt surfaced pavements that have been laid over a PCC slab. It does not include reflection cracks from any other type of base (i.e., cement or lime stabilized); these cracks are caused mainly by thermal or moisture induced movement of the PCC slab beneath the AC surface. This distress is not load related; however, traffic loading may cause a breakdown of the AC surface near the crack. If the pavement is fragmented along a crack, the crack is said to be spalled. A knowledge of slab dimension beneath the AC surface will help to identify these distresses.

Severity Levels (Figure B–8)

L—One of the following conditions exists:

1. Non-filled crack width is less than 3/8 in. (10 mm).
2. Filled crack of any width (filler in satisfactory condition).

M—One of the following conditions exists:

1. Non-filled crack width is 3/8 to 3 in. (10 to 76 mm).
2. Non-filled crack of any width up to 3 in. (76 mm) surrounded by light random cracking.
3. Filled crack of any width surrounded by light random cracking.

H—One of the following conditions exists:

1. Any crack filled or non-filled surrounded by medium or high severity random cracking;
2. Non-filled cracks over 3 in. (76 mm).
3. A crack of any width where a few inches of pavement around the crack are severely broken (Crack is severely broken).

How to Measure

Joint reflection cracking is measured in linear feet. The length and severity level of each crack should be identified and recorded separately. For example, a crack that is 50 ft long may have 10 ft of high severity cracks; these are all recorded separately. If a bump occurs at the reflection crack, it is also recorded.

Options for Repair

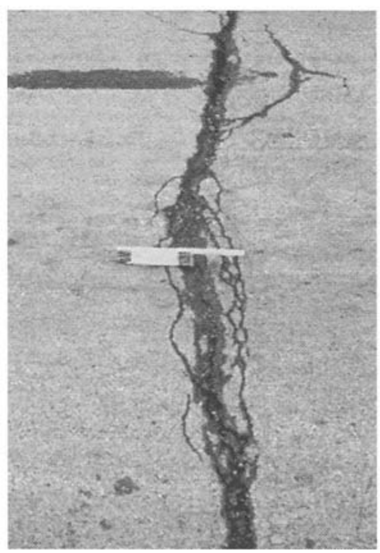
L—Seal is over 1/8 in. (3 mm).

M—Seal cracks; Partial-depth patch.

H—Partial-depth patch; Reconstruct joint.

LOW

MEDIUM



HIGH

Figure B-8. Join Reflection Cracking.

Lane/Shoulder Drop Off (09)

Description

Lane/shoulder drop off is a difference in elevation between the pavement edge and the shoulder. This distress is caused by shoulder erosion, shoulder settlement, or by building up the roadway without adjusting the shoulder level.

Severity Levels (Figure B–9)

L—The difference in elevation between the pavement edge and shoulder is 1 to 2 in. (25 to 51 mm).

M—The difference in elevation is > 2 to 4 in. (51 to 102 mm).

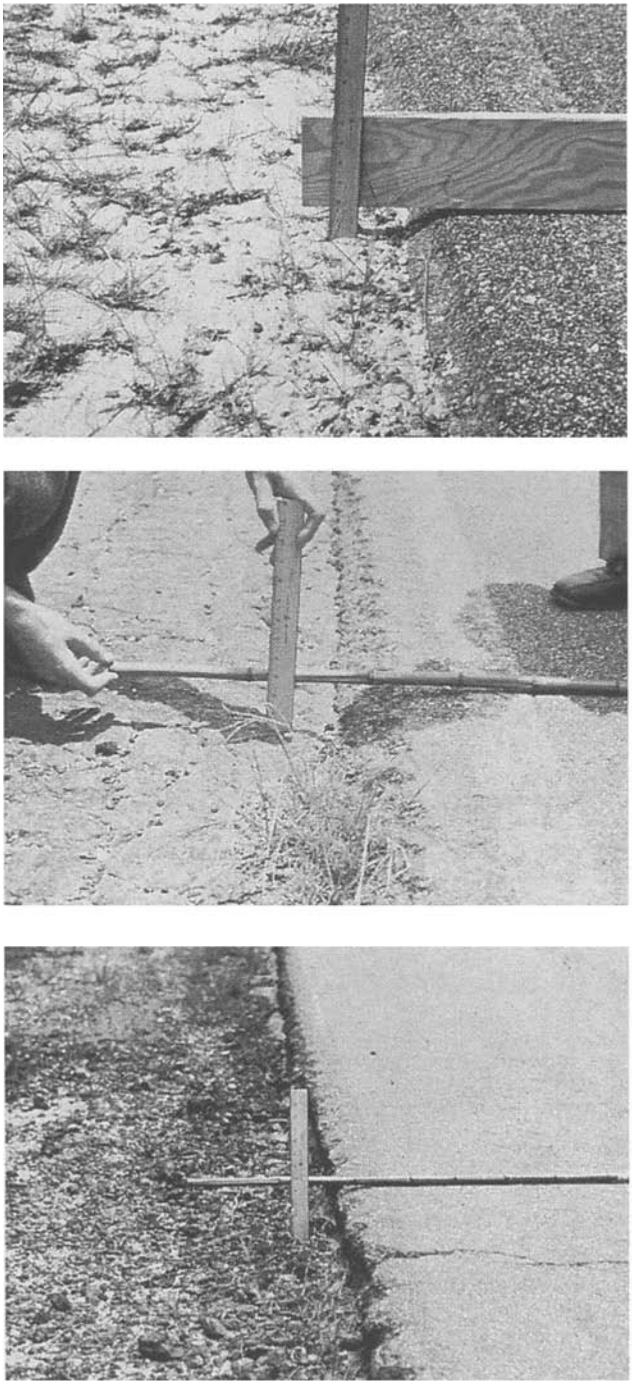
H—The difference in elevation is > 4 in. (102 mm).

How to Measure

Lane/shoulder drop off is measured in linear feet.

Options for Repair

L, M, H—Regrade and fill shoulders to match lane height.



LOW

MEDIUM

HIGH

Figure B-9. Lane/Shoulder Drop-Off.

Longitudinal and Transverse Cracking (10) (Non-PCC Slab Joint Reflective)

Description

Longitudinal cracks are parallel to the pavement's centerline or laydown direction. They may be caused by:

1. A poorly constructed paving lane joint.
2. Shrinkage of the AC surface due to low temperatures or hardening of the asphalt and/or daily temperature cycling.
3. A reflective crack caused by cracking beneath the surface course, including cracks in PCC slabs (but not PCC joints).

Transverse cracks extend across the pavement at approximately right angles to the pavement centerline or direction of laydown. These types of cracks are not usually load-associated.

Severity Levels (Figure B–10)

L—One of the following conditions exists:

1. Non-filled crack width is less than 3/8 in. (10 mm).
2. Filled crack of any width (filler in satisfactory condition).

M—One of the following conditions exists:

1. Non-filled crack width is 3/8 to 3 in. (10 to 76 mm).
2. Non-filled crack is up to 3 in. (76 mm) surrounded by light and random cracking.
3. Filled crack is of any width surrounded by light random cracking.

H—One of the following conditions exists:

1. Any crack filled or non-filled surrounded by medium or high severity random cracking.
2. Non-filled crack over 3 in. (76 mm).
3. A crack of any width where a few inches of pavement around the crack is severely broken.

How to Measure

Longitudinal and transverse cracks are measured in linear feet. The length and severity of each crack should be recorded after identification. If the crack does not have the same severity level along its entire length, each portion of the crack having a different severity level should be recorded separately. If a bump or sag occurs at the crack, it is also recorded.

LOW

MEDIUM

HIGH

Figure B-10. Longitudinal and Transverse Cracking.

Options for Repair

l—Do nothing; Seal cracks > 1/8 in. wide.

M—Seal cracks.

H—Seal cracks; Partial-depth patch.

Patching and Utility Cut Patching (11)

Description

A patch is an area of pavement that has been replaced with new material to repair the existing pavement. A patch is considered a defect no matter how well it is performing (a patched area or adjacent area usually does not perform as well as an original pavement section). Generally, some roughness is associated with this distress.

Severity Levels (Figure B–11)

L—Patch is in good condition and satisfactory. Ride quality is rated as low severity or better.

M—Patch is moderately deteriorated and/or ride quality is rated as medium severity.

H—Patch is badly deteriorated and/or ride quality is rated as high severity. Needs replacement soon.

How to Measure

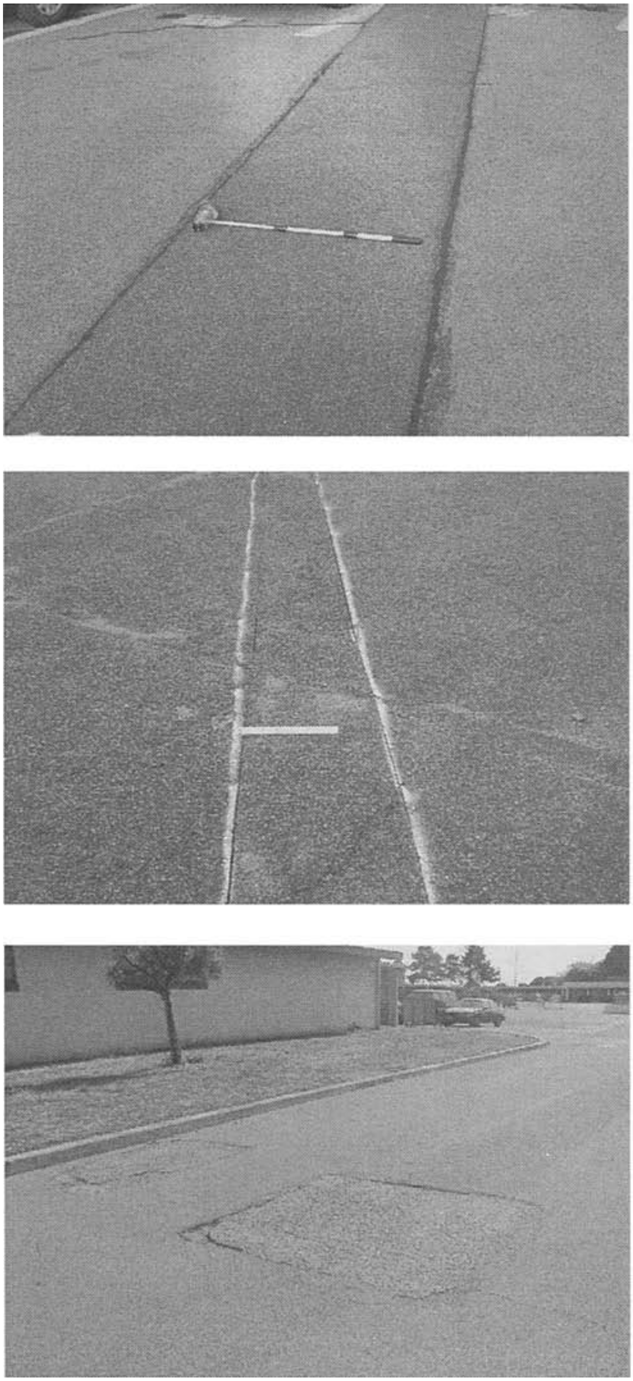
Patching is rated in square feet of surface area. However, if a single patch has areas of differing severity, these areas should be measured and recorded separately. For example, a 25 sq ft (2.32 m^2) patch may have 10 sq ft (0.9 m^2) of medium severity and 15 sq ft (1.35 m^2) of low severity. These areas would be recorded separately. No other distresses (e.g., shoving and cracking) are recorded within a patch; even if the patch material is shoving or cracking, the area is rated only as a patch. If a large amount of pavement has been replaced, it should not be recorded as a patch, but considered as new pavement (e.g., replacement of a complete intersection).

Options for Repair

L—Do nothing

M—Do nothing; Replace patch.

H—Replace patch.



LOW

MEDIUM

HIGH

Figure B-11. Patching and Utility Cut Patching.

Polished Aggregate (12)

Description

This distress is caused by repeated traffic applications. When the aggregate in the surface becomes smooth to the touch, adhesion with vehicle tires is considerably reduced. When the portion of aggregate extending above the surface is small, the pavement texture does not significantly contribute to reducing vehicle speed. Polished aggregate should be counted when close examination reveals that the aggregate extending above the asphalt is negligible, and the surface aggregate is smooth to the touch. This type of distress is indicated when the number on a skid resistance test is low or has dropped significantly from a previous rating.

Severity Levels (Figure B–12)

No degrees of severity are defined. However, the degree of polishing should be clearly evident in the sample unit in that the aggregate surface should be smooth to the touch.

How to Measure

Polished aggregate is measured in square feet of surface area. If bleeding is counted, polished aggregate should not be counted.

Options for Repair

L, M, H—Do nothing; Surface treatment; Overlay; Mill and Overlay.

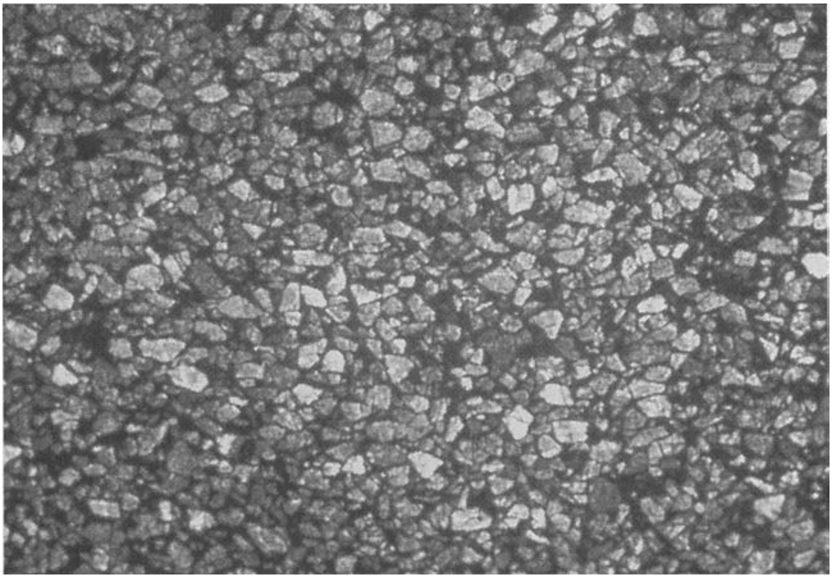


Figure 12. Polished Aggregate.

Potholes (13)

Description

Potholes are small—usually less than 3 ft (0.9 m) in diameter—bowl shaped depressions in the pavement surface. They generally have sharp edges and vertical sides near the top of the hole. Their growth is accelerated by free moisture collection inside the hole. Potholes are produced when traffic abrades small pieces of the pavement surface. The pavement continues to disintegrate because of poor surface mixtures, weak spots in the base or subgrade, or because it has reached a condition of high severity alligator cracking. Potholes most often are structurally related distresses and should not be confused with raveling and weathering. When holes are created by high severity alligator cracking, they should be identified as potholes, not as weathering.

Severity Levels (Figure B–13)

The levels of severity for potholes less than 30 in. (762 mm) in diameter are based on both the diameter and the depth of the pothole, according to Table B-1.

If the pothole is more than 30 in. (76 mm) in diameter, the area should be determined in square feet and divided by 5 sq ft (0.47 m^2) to find the equivalent number of holes. If the depth is 1 in. (25 mm) or less, the holes are considered medium severity. If the depth is more than 1 in. (25 mm), they are considered high severity.

Table B-1. Levels of Severity for Potholes.

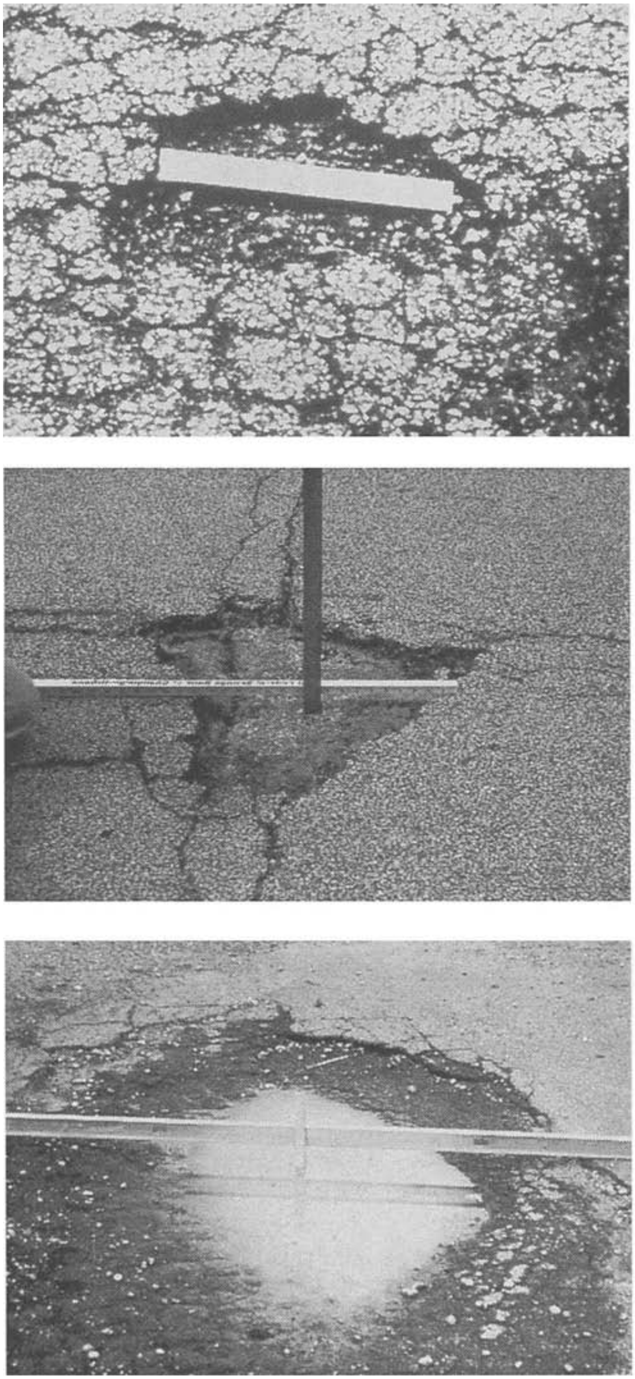
	Average Diameter, in. (mm)		
Maximum Depth to Pothole	4 to 8 in. (102 to 203 mm)	8 to 18 in. (203 to 457 mm)	18 to 30 in. (457 to 762 mm)
1/2 to 1 in. (12.7 to 25.4 mm)	L	L	M
>1 to 2 in. (25.4 to 50.8 mm)	L	M	H
>2 in. (50.8 mm)	M	M	H

How to Measure

Potholes are measured by counting the number that are low, medium, and high severity and recording them separately.

Options for Repair

L—Do nothing; Partial or full-depth patch.
M—Partial or full-depth patch.
H—Full-depth patch.



LOW

MEDIUM

HIGH

Figure B-13. Pothole.

Railroad Crossing (14)

Description

Railroad crossing defects are depressions or bumps around and/or between tracks.

Severity Levels (Figure B–14)

L—Railroad crossing causes low severity ride quality.

M—Railroad crossing causes medium severity ride quality.

H—Railroad crossing causes high severity ride quality.

How to Measure

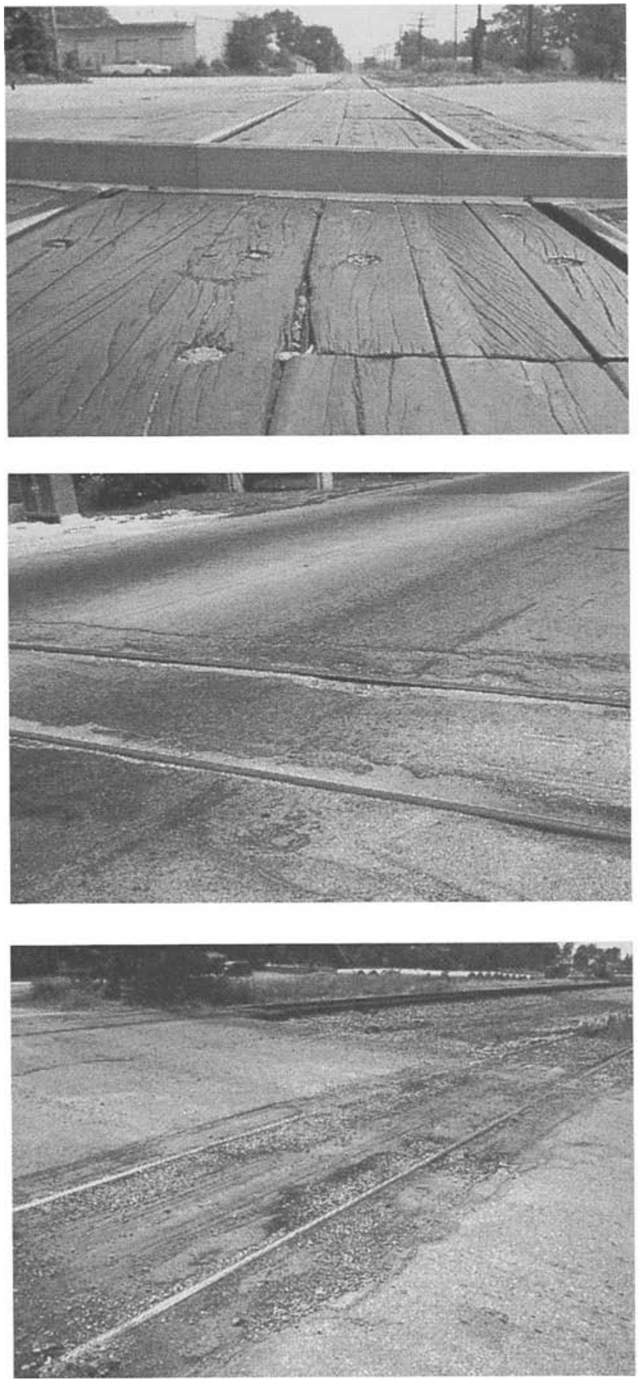
The area of the crossing is measured in square feet of surface area. If the crossing does not affect ride quality, it should not be counted. Any large bump created by the tracks should be counted as part of the crossing.

Options for Repair

L—Do nothing.

M—Shallow or partial-depth patch approach; Reconstruct crossing.

H—Shallow or partial-depth patch approach; Reconstruct crossing.



LOW

MEDIUM

HIGH

Figure B-14. Railroad Crossing.

Rutting (15)

Description

A rut is a surface depression in the wheel paths. Pavement uplift may occur along the sides of the rut, but, in many instances, ruts are noticeable only after a rainfall when the paths are filled with water. Rutting stems from a permanent deformation in any of the pavement layers or subgrades, usually caused by consolidated or lateral movement of the materials due to traffic load. Significant rutting can lead to major structural failure of the pavement.

Severity Levels (Figure B–15)

Mean Rut Depth

L—1/4 to 1/2 in.

M— Greater than 1/2 in. up to 1 in.

H—Greater than 1 in.

How to Measure

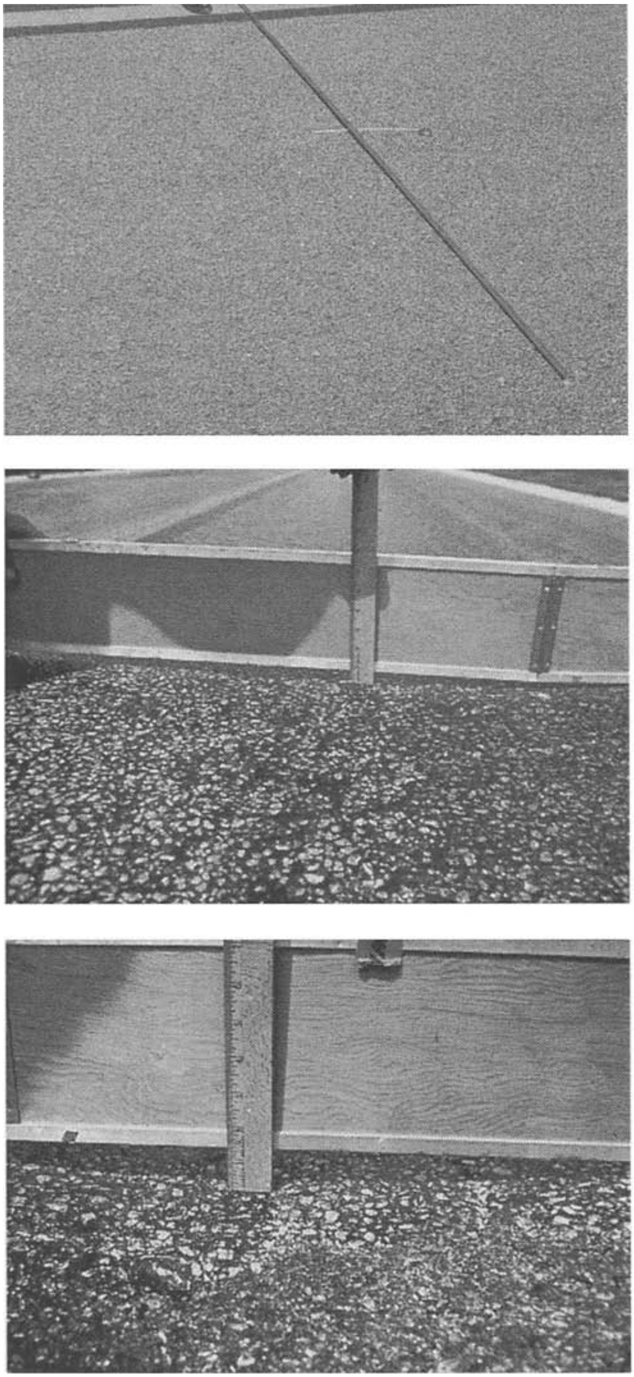
Rutting is measured in square feet of surface area and its severity is determined by the mean depth of the rut (see above). The mean rut depth is calculated by laying a straight edge across the rut, measuring its depth, then using measurements taken along the length of the rut to compute its mean depth in inches.

Options for Repair

L—Do nothing; Mill and overlay.

M—Shallow, partial, or full-depth patch; Mill and overlay.

H—Shallow, partial, or full-depth patch; Mill and overlay.



LOW

MEDIUM

HIGH

Figure B-15. Rutting.

Shoving (16)

Description

Shoving is a permanent, longitudinal displacement of a localized area of the pavement surface caused by traffic loading. When traffic pushes against the pavement, it produces a short, abrupt wave in the pavement surface. This distress normally occurs only in unstable liquid asphalt mix (cutback or emulsion) pavements.

Shoves also occur where asphalt pavements abutt PCC pavements; the PCC pavements increase in length and push the asphalt pavement, causing the shoving.

Severity Levels (Figure B–16)

L—Shove causes low severity ride quality.

M—Shove causes medium severity ride quality.

H—Shove causes high-severity ride quality.

How to Measure

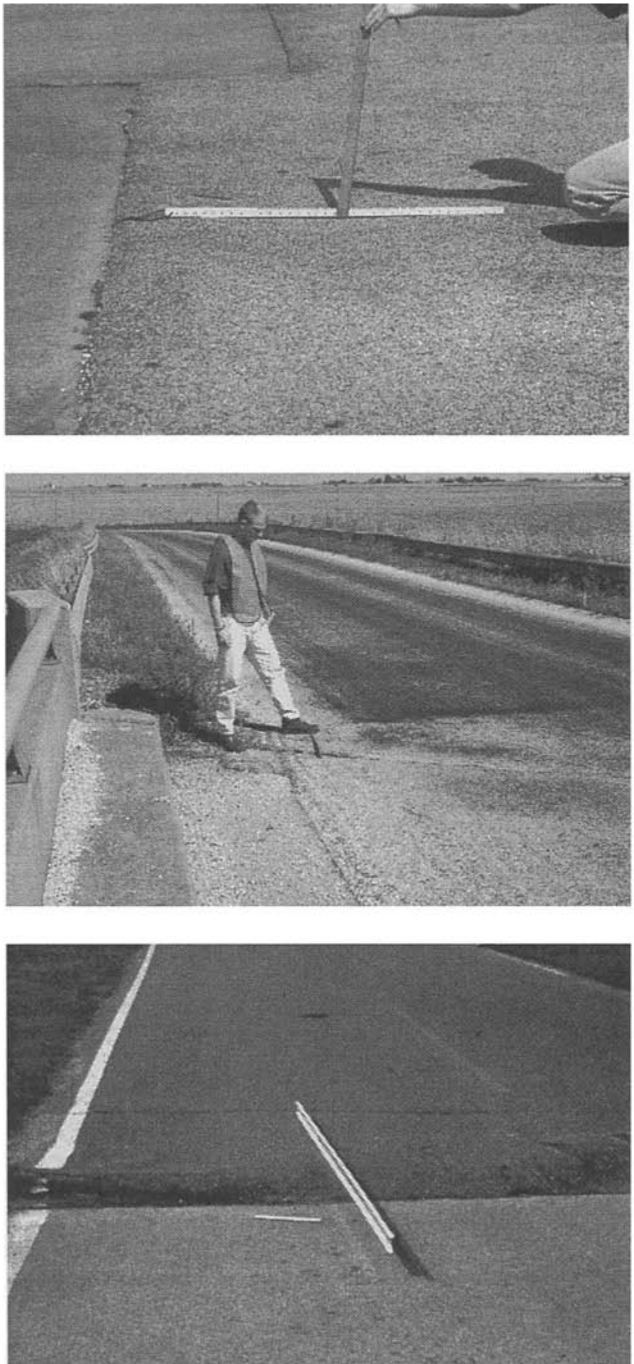
Shoves are measured in square feet of surface area. Shoves occurring in patches are considered in rating the patch, not as a separate distress.

Options for Repair

L—Do nothing; Mill.

M—Mill; Partial or full-depth patch.

H—Mill; Partial or full-depth patch.



LOW

MEDIUM

HIGH

Figure B-16. Shoving.

Slippage Cracking (17)

Description

Slippage cracks are crescent or half-moon shaped cracks. They are produced when braking or turning wheels cause the pavement surface to slide or deform. This distress usually occurs when there is a low-strength surface mix or poor bond between the surface and the next layer of the pavement structure.

Severity Level (Figure B–17)

L—Average crack width is < 3/8 in. (10 mm).

M—One of the following conditions exists:

1. Average crack width between 3/8 in. and 1-1/2 in. (10 mm and 38 mm).
2. The area around the crack is broken into tight-fitting pieces.

H—One of the following conditions exists:

1. The average crack width is greater than 1-1/2 in. (38 mm).
2. The area around the crack is broken into easily removed pieces.

How to Measure

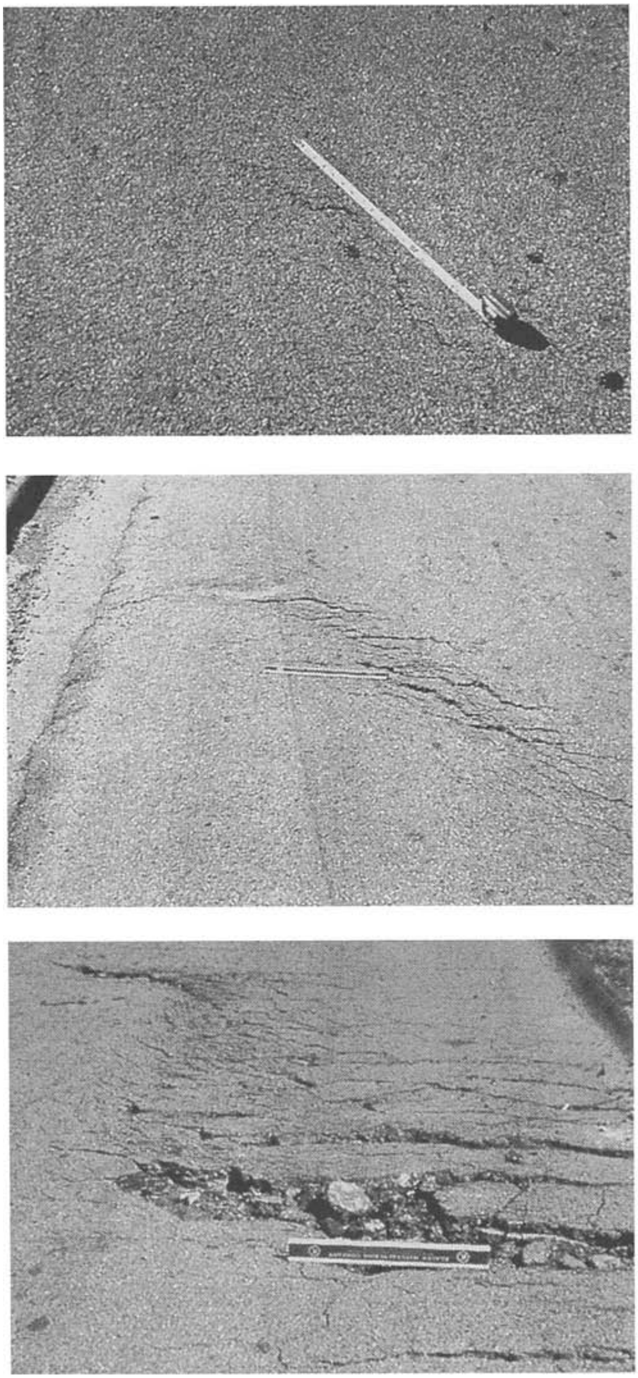
The area associated with a given slippage crack is measured in square feet and rated according to the highest level of severity in the area.

Options for Repair

L—Do nothing; Partial-depth patch.

M—Partial-depth patch.

H—Partial depth patch.



LOW

MEDIUM

HIGH

Figure B-17. Slippage Cracking.

Swell (18)

Description

Swell is characterized by an upward bulge in the pavement's surface—a long, gradual wave greater than 10 ft (3 m) long. Swelling can be accompanied by surface cracking. This distress is usually caused by frost action in the subgrade or by swelling soil.

Severity Level (Figure B–18)

L—Swell causes low-severity ride quality. Low-severity swells are not always easy to see, but can be detected by driving at the speed limit over the pavement section. An upward motion will occur at the swell if it is present.

M—Swell causes medium severity ride quality.

H—Swell causes high severity ride quality.

How to Measure

The surface area of the swell is measured in square feet.

Options for Repair

L—Do nothing.

M—Do nothing; Reconstruct.

H—Reconstruct.

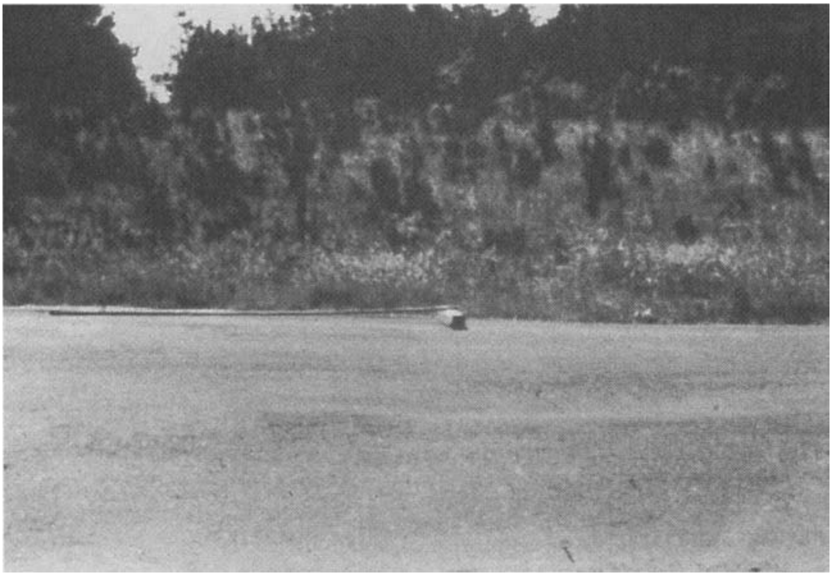


Figure B-18. Example Swell. Severity Level is Based on Ride Quality Criteria.

Weathering and Raveling (19)

Description

Weathering and raveling are the wearing away of the pavement surface due to a loss of asphalt or tar binder and dislodged aggregate particles. These distresses indicate that either the asphalt binder has hardened appreciably or that a poor quality mixture is present. In addition, raveling may be caused by certain types of traffic, for example, tracked vehicles. Softening of the surface and dislodging of the aggregates due to oil spillage are also included under raveling.

Severity Levels (Figure B–19)

L—Aggregate or binder has started to wear away. In some areas, the surface is starting to pit. In the case of oil spillage, the oil stain can be seen, but the surface is hard and cannot be penetrated with a coin.

M—Aggregate or binder has worn away. The surface texture is moderately rough and pitted. In the case of oil spillage, the surface is soft and can be penetrated with a coin.

H—Aggregate or binder has been worn away considerably. The surface texture is very rough and severely pitted. The pitted areas are less than 4 in. (10 mm) in diameter and less than 1/2 in. (13 mm) deep; pitted areas larger than this are counted as potholes. In the case of oil spillage, the asphalt binder has lost its binding effect and the aggregate has become loose.

How to Measure

Weathering and raveling are measured in square feet of surface area.

Options for Repair

L—Do nothing; Surface seal; Surface treatment.

M[a]—Surface seal; Surface treatment; Overlay.

H[a]—Surface treatment; Overlay; Recycle; Reconstruct.

---

[a]If localized, that is, due to oil spillage, then partial-depth patch.

LOW

MEDIUM

HIGH

Figure B-19. Weathering and Raveling.

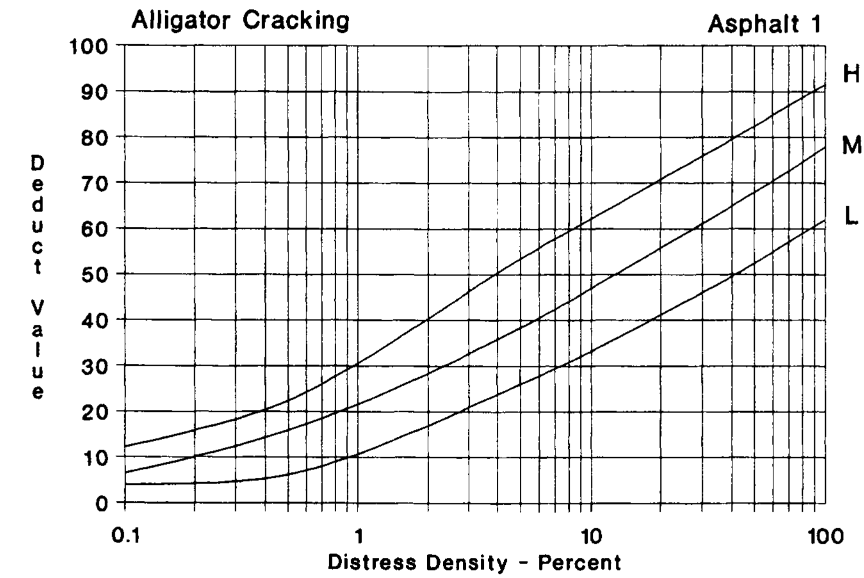


Figure B-20. Alligator Cracking.

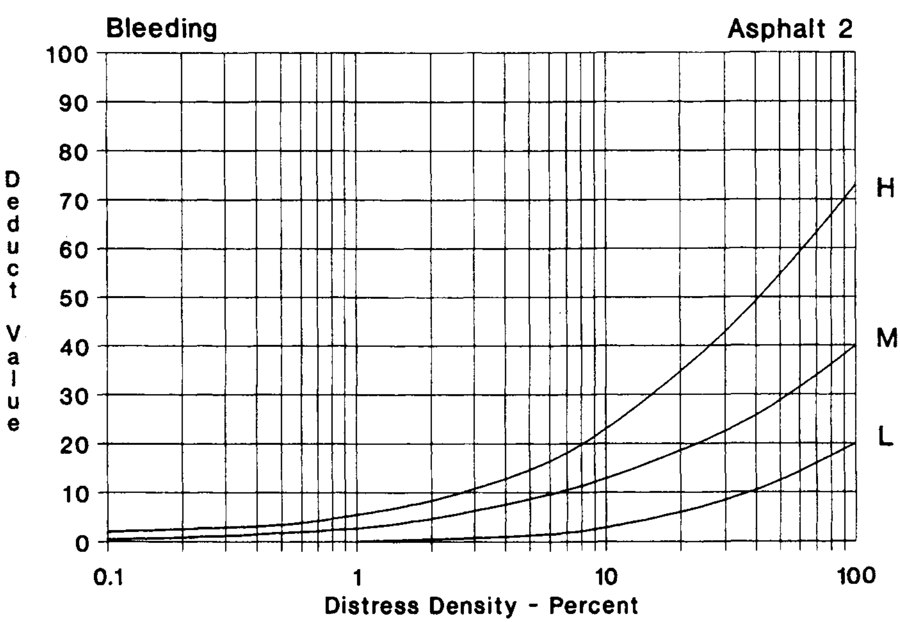


Figure B-21. Bleeding.

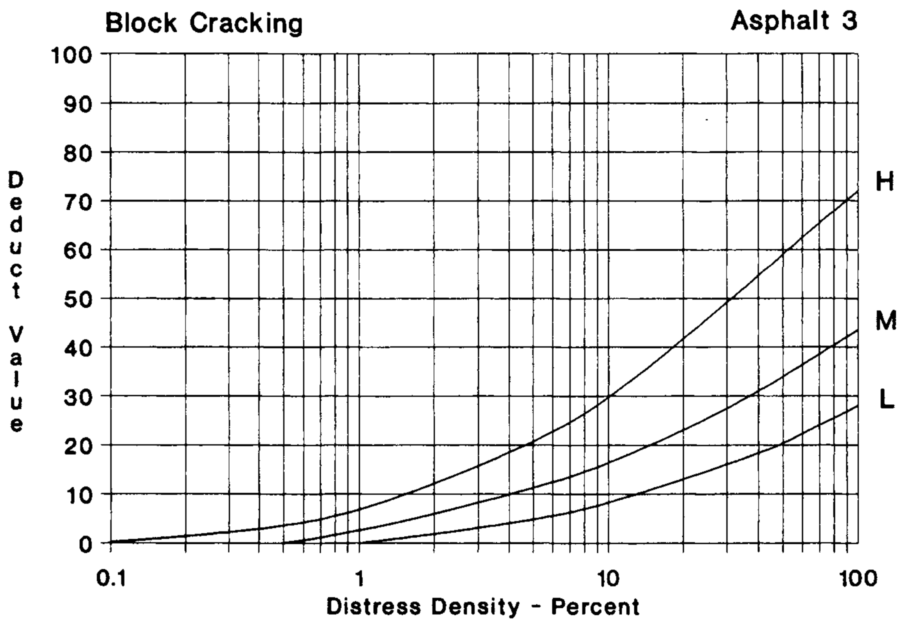


Figure B-22. Block Cracking.

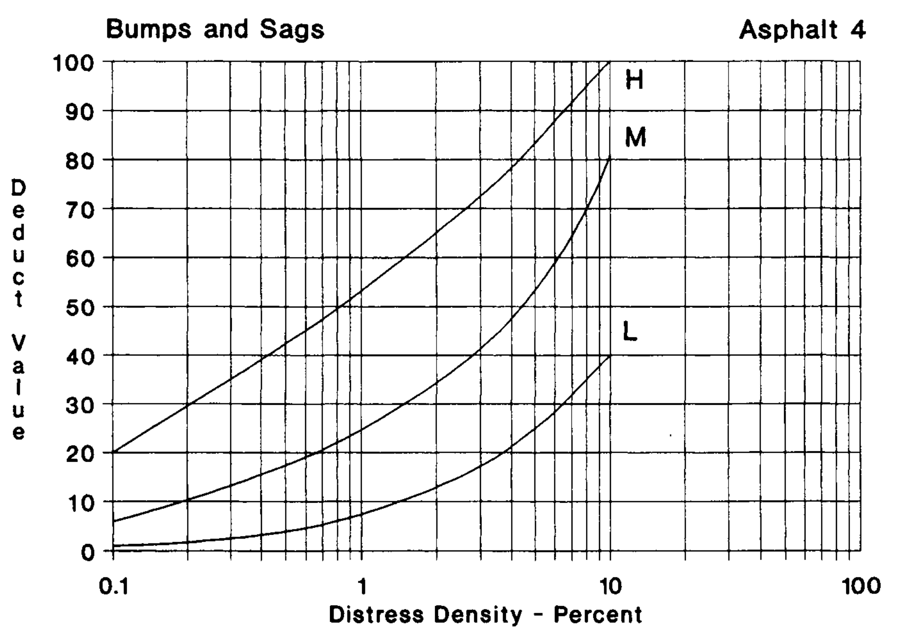


Figure B-23. Bumps and Sags.

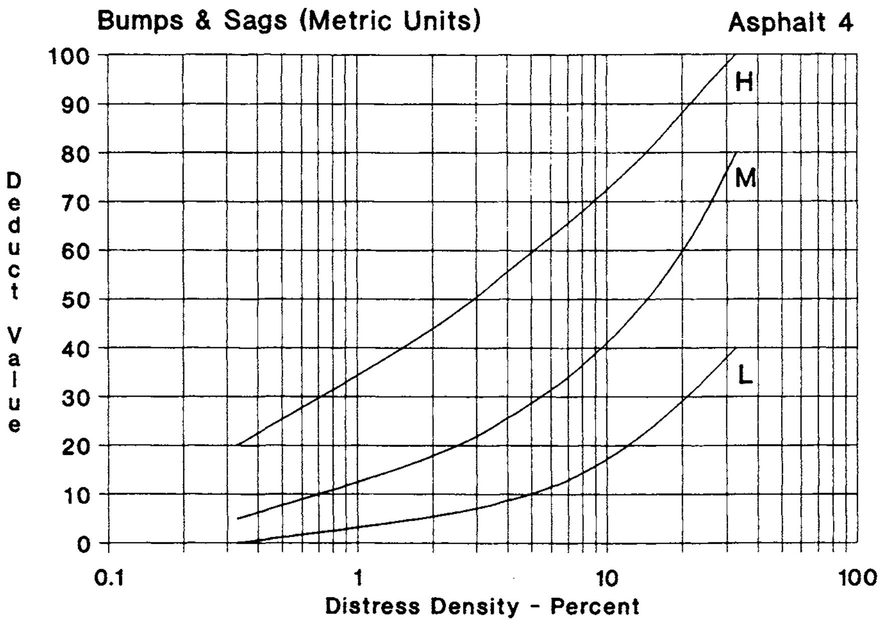


Figure B-24. Bumps and Sags (metric units).

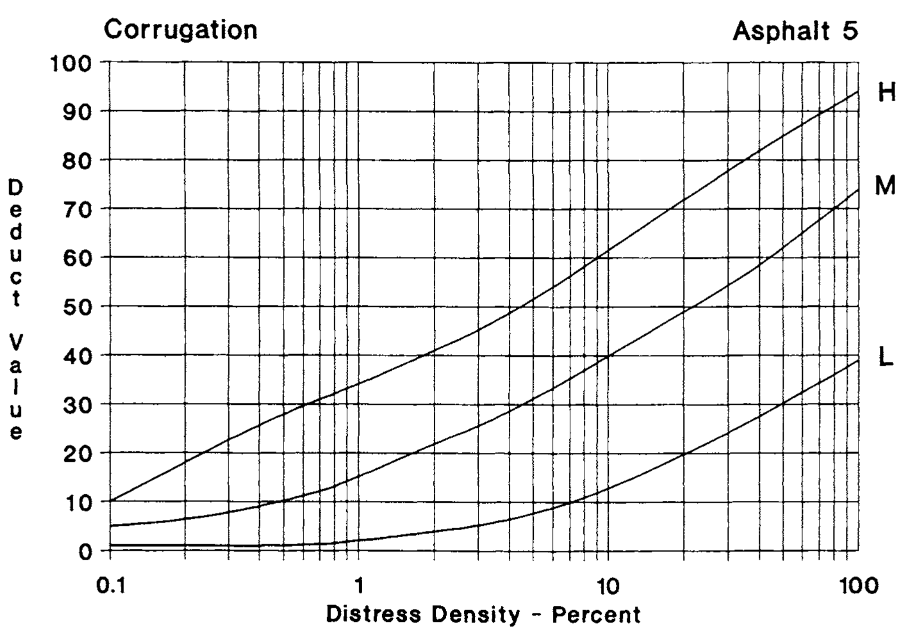


Figure B-25. Corrugation.

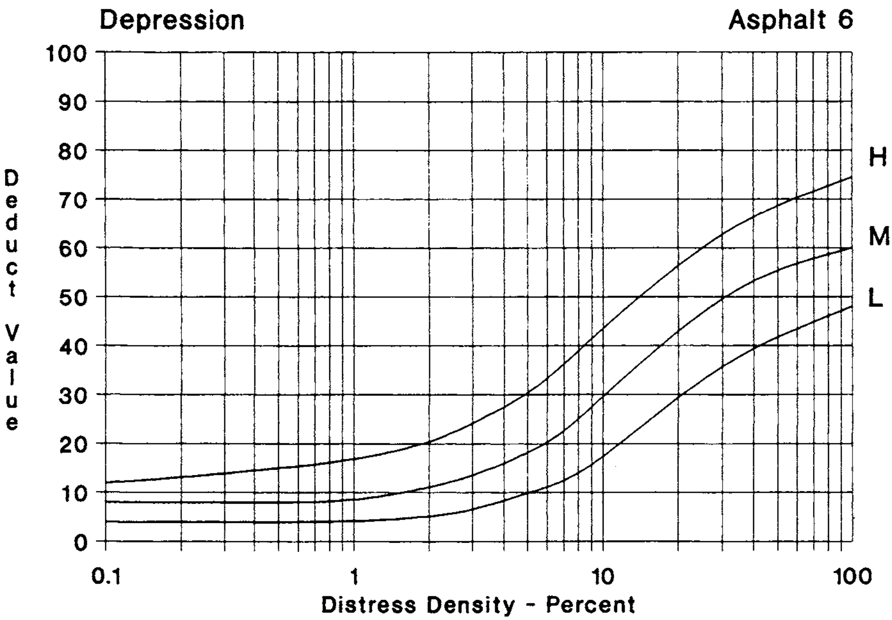


Figure B-26. Depression.

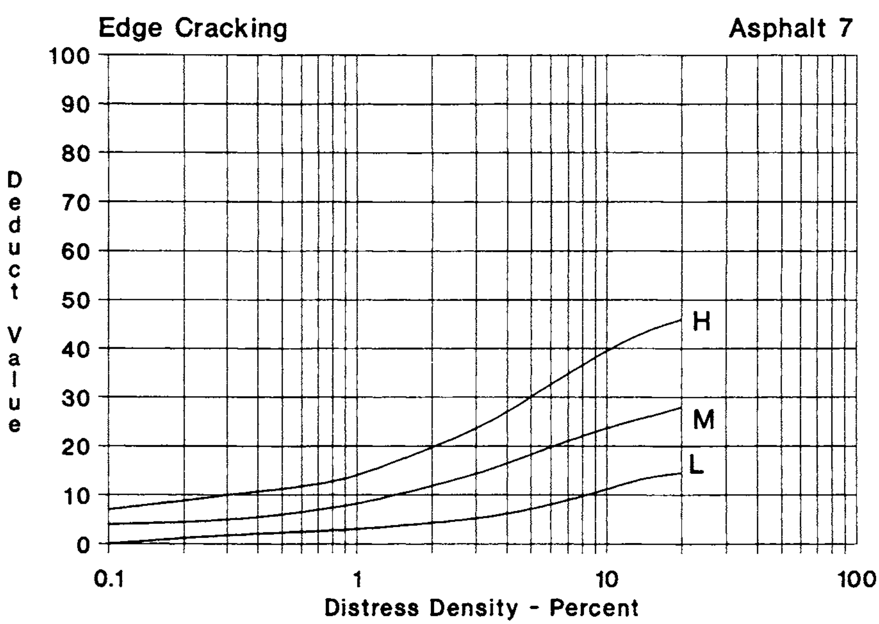


Figure B-27. Edge Cracking.

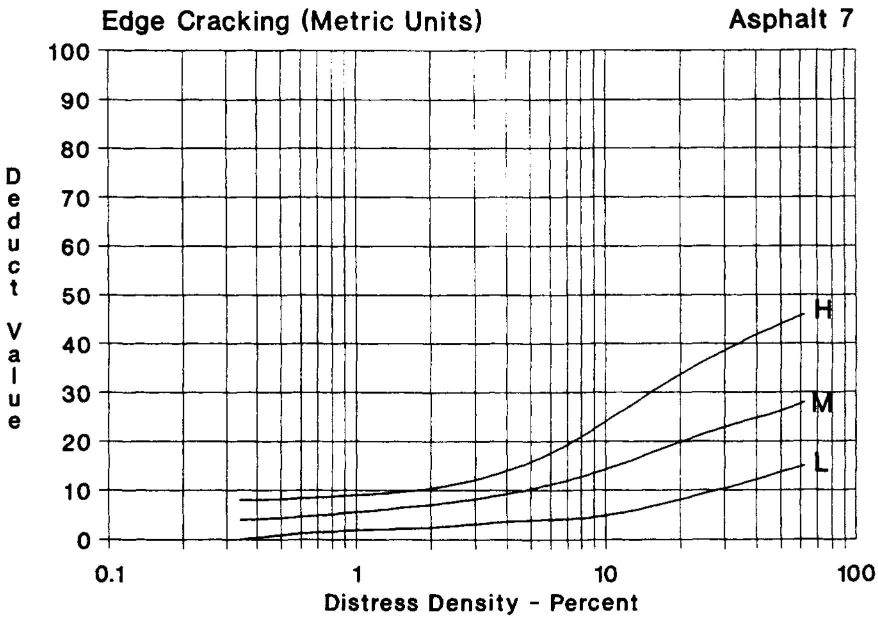


Figure B-28. Edge Cracking (metric units).

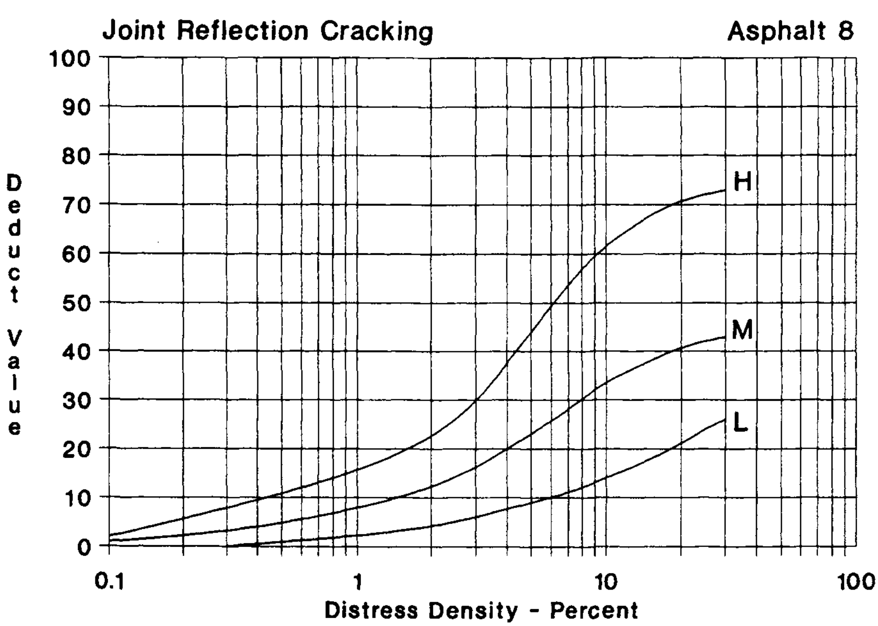


Figure B-29. Joint Reflection Cracking.

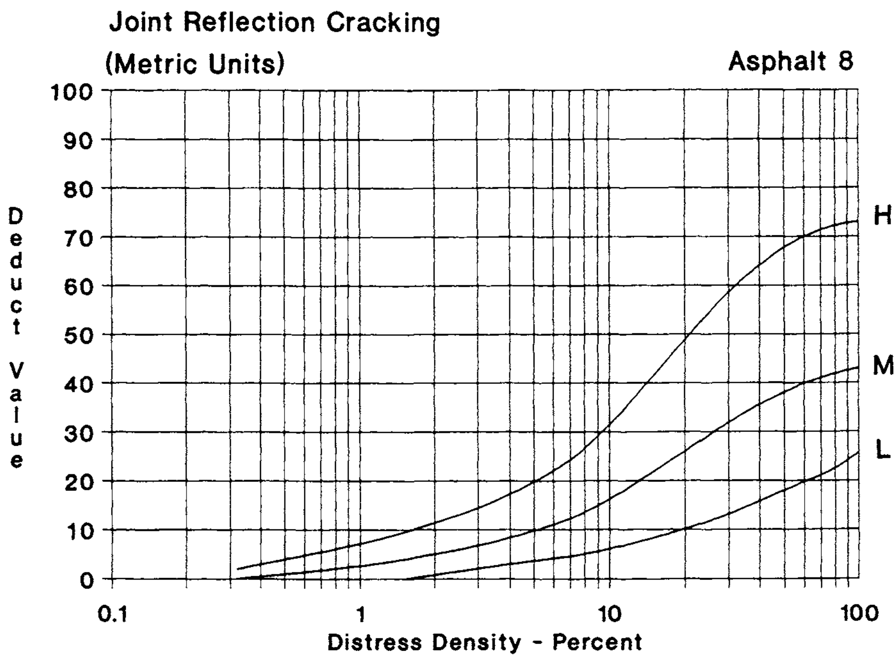


Figure B-30. Joint Reflection Cracking (metric units).

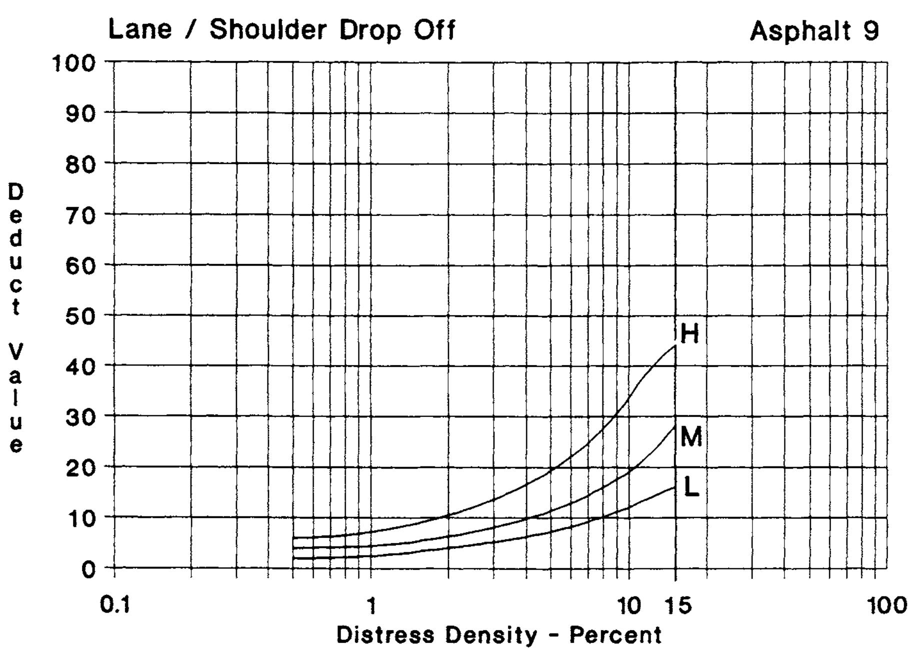


Figure B-31. Lane/Shoulder Drop-off.

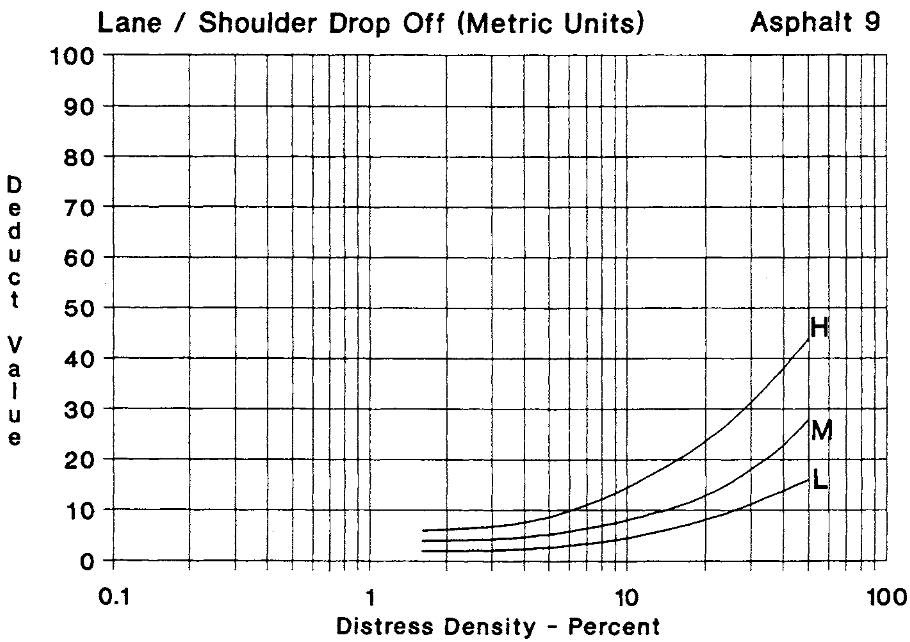


Figure B-32. Lane/Shoulder Drop-off (metric units).

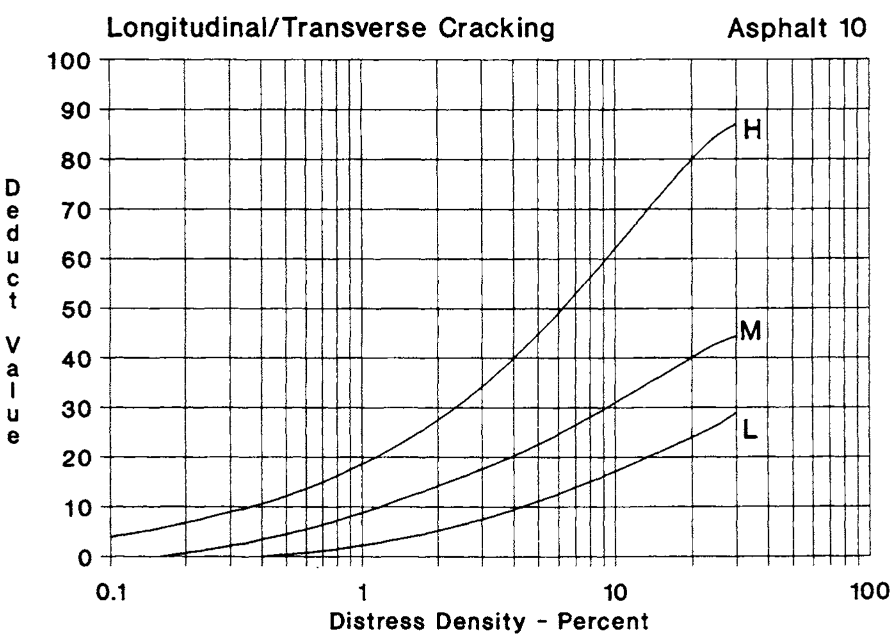


Figure B-33. Longitudinal/Transverse Cracking.

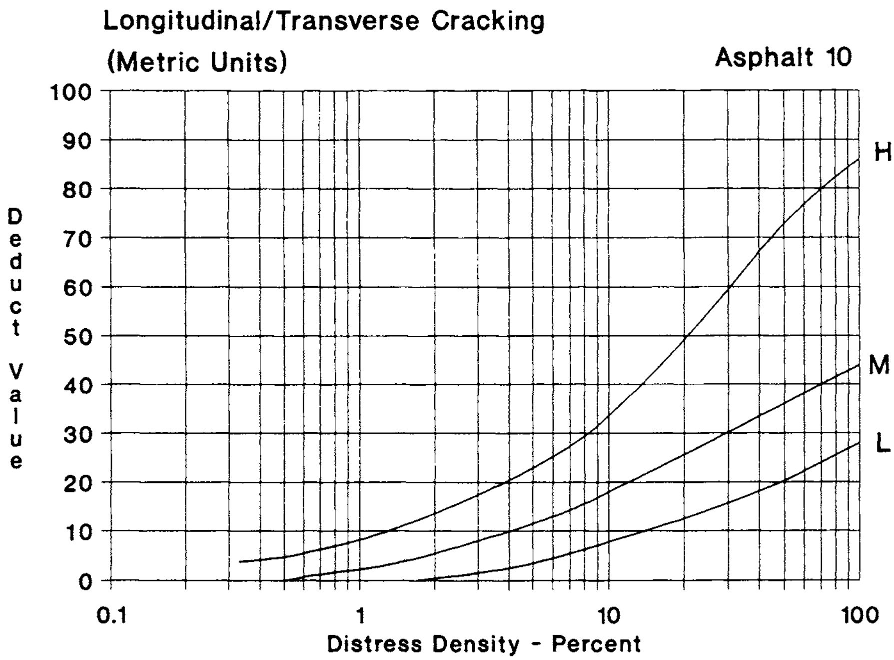


Figure B-34. Longitudinal/Transverse Cracking (metric units).

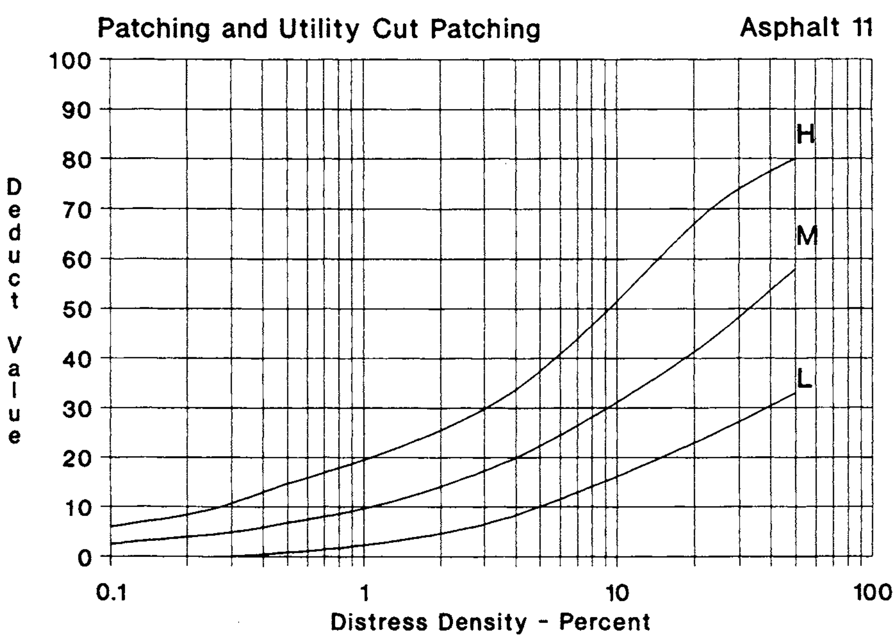


Figure B-35. Patching and Utility Cut Patching.

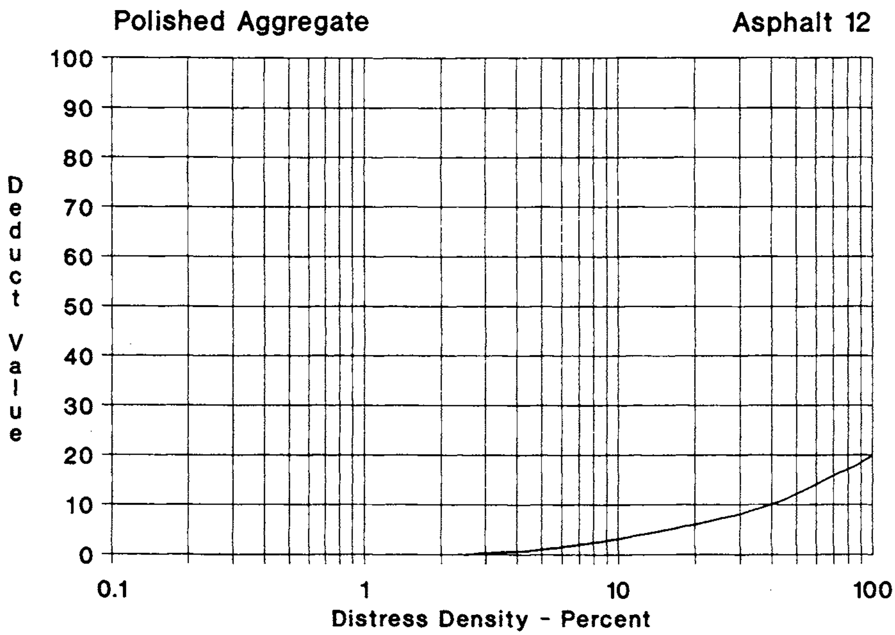


Figure B-36. Polished Aggregate.

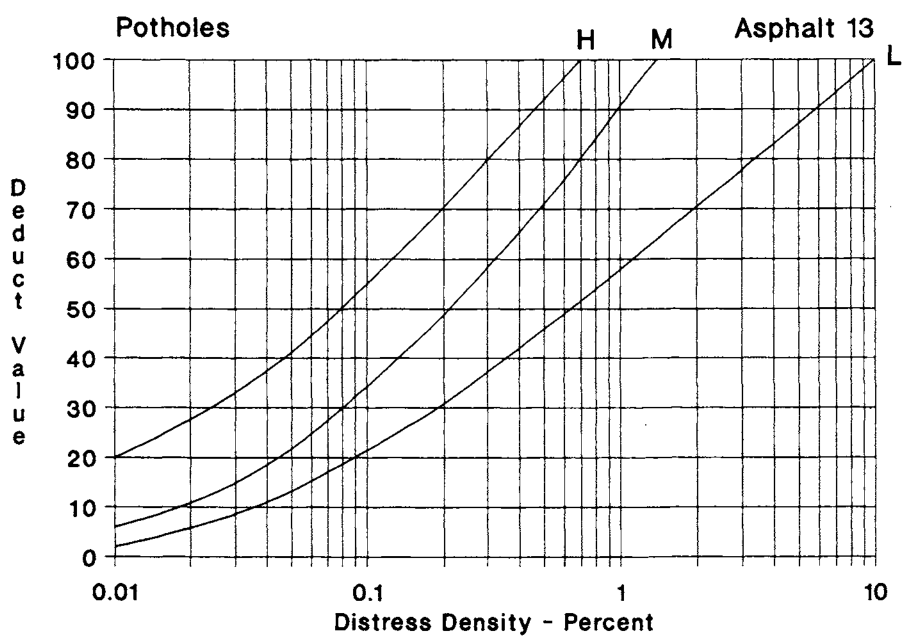


Figure B-37. Potholes.

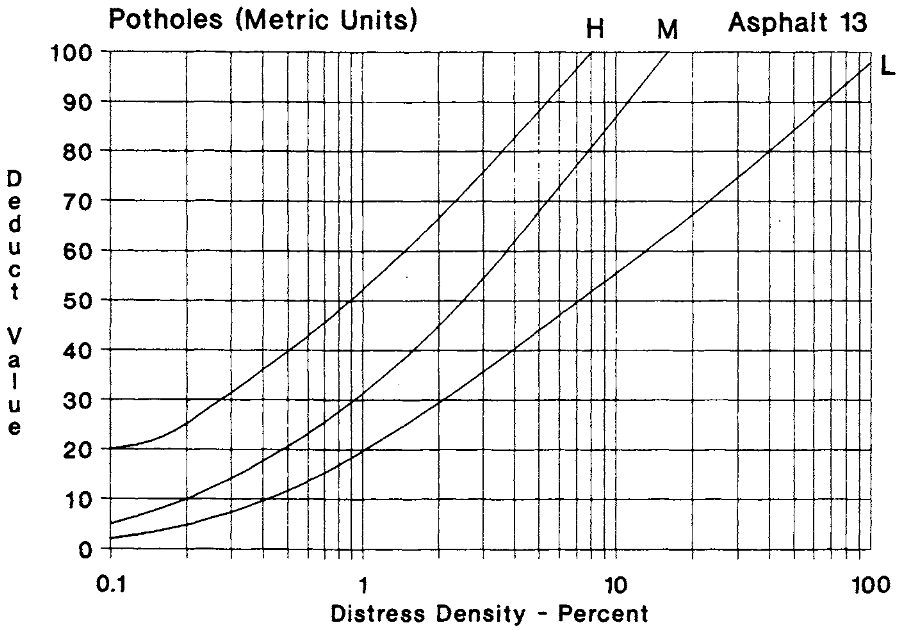


Figure B-38. Potholes (metric units).

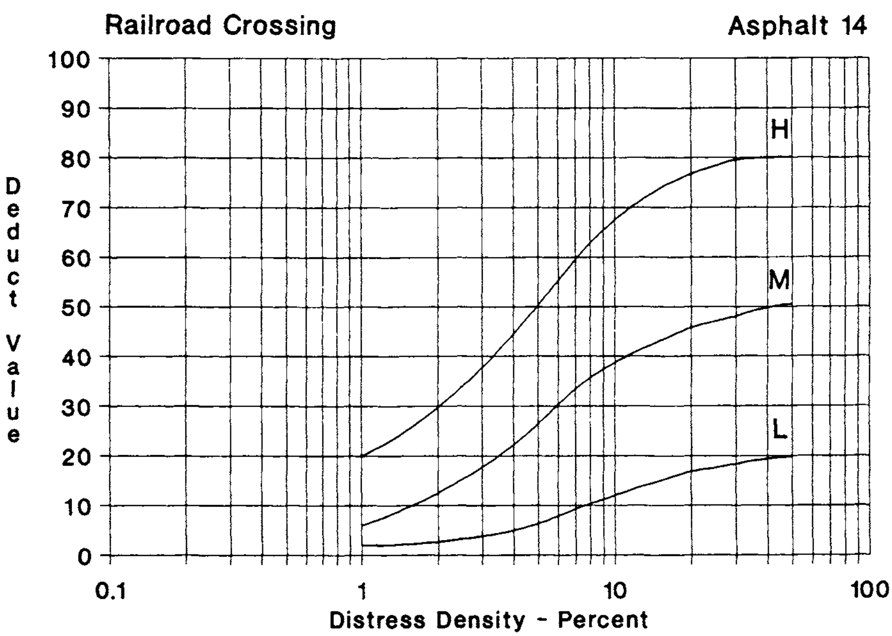


Figure B-39. Railroad Crossing.

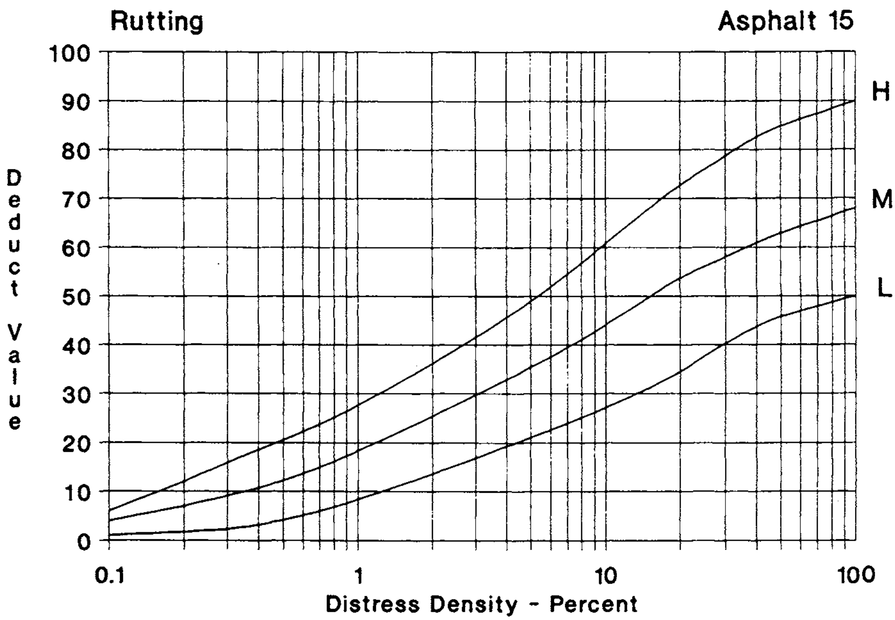


Figure B-40. Rutting.

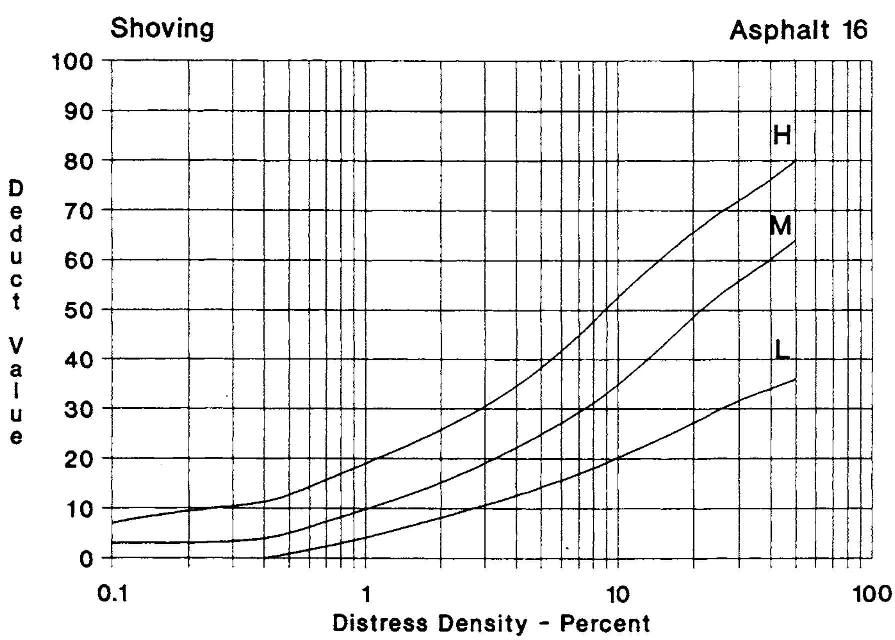


Figure B-41. Shoving.

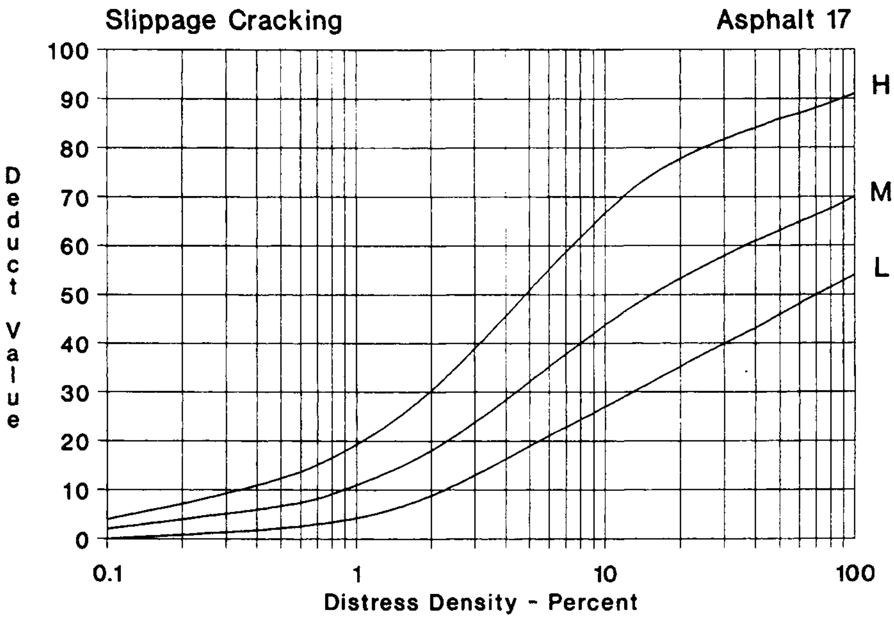


Figure B-42. Slippage Cracking.

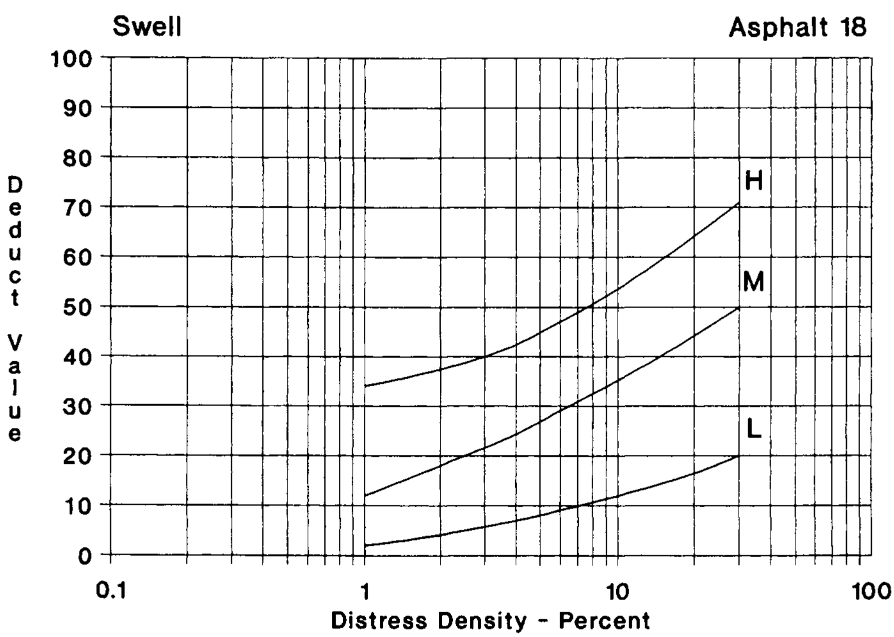


Figure B-43. Swell.

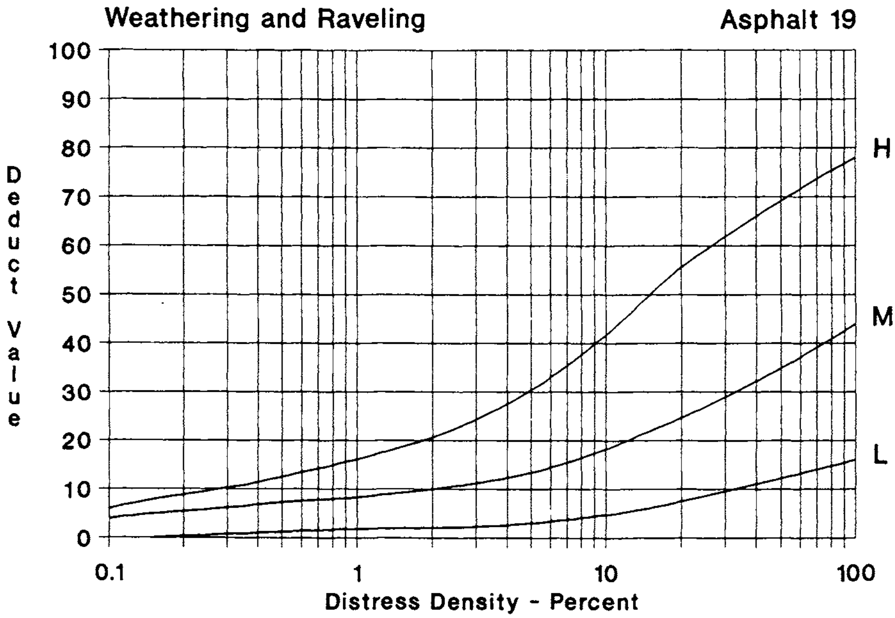


Figure B-44. Weathering and Raveling.

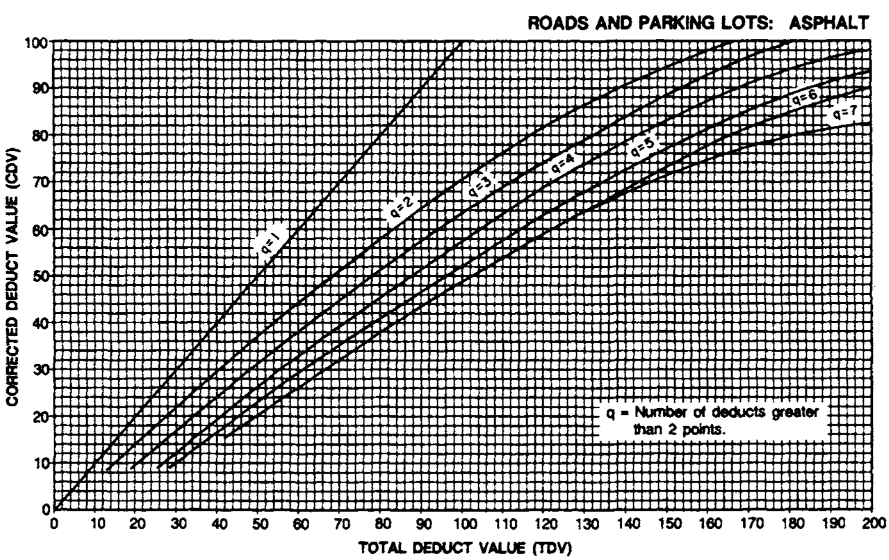


Figure B-45.

APPENDIX C

Portland Cement Concrete Roads: Distress Definitions and Deduct Value Curves

Blowup/Buckling (21)

Description

Blowups or buckles occur in hot weather, usually at a transverse crack or joint that is not wide enough to permit slab expansion. The insufficient width is usually caused by infiltration of incompressible materials into the joint space. When expansion cannot relieve enough pressure, a localized upward movement of the slab edges (buckling) or shattering will occur in the vicinity of the joint. Blowups can also occur at utility cuts and drainage inlets.

Severity Levels (Figure C–1)

L—Buckling or shattering causes low severity ride quality.

M—Buckling or shattering causes medium severity ride quality.

H—Buckling or shattering causes high severity ride quality.

How to Count

At a crack, a blowup is counted as being in one slab. However, if the blowup occurs at a joint and affects two slabs, the distress should be recorded as occurring in two slabs. When a blowup renders the pavement inoperable, it should be repaired immediately.

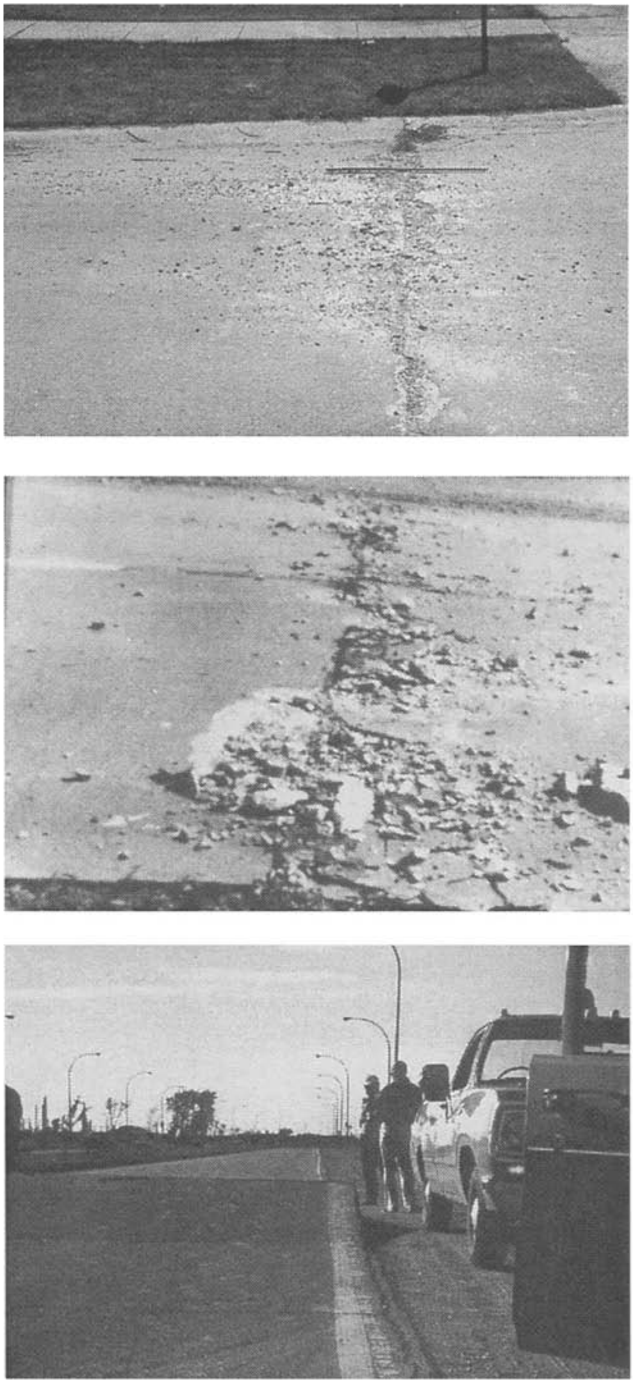
Options for Repair

L[a]—Do nothing; Partial, or full-depth patch.

M[a]—Full-depth patch; Slab replacement.

H[a]—Full-depth patch; Slab replacement.

[a]Must provide expansion joints if patched.



LOW

MEDIUM

HIGH

Figure C-1. B lowup/Buckling.

Corner Break (22)

Description

A corner break is a crack that intersects the joints at a distance less than or equal to one half the slab length on both sides, measured from the corner of the slab. For example, a slab measuring 12 by 20.0 ft (3.7 by 6.1 m) that has a crack 5 ft (1.5 m) on one side and 12 ft (3.7 m) on the other side is not considered a corner break; it is a diagonal crack. However, a crack that intersects 4 ft (0.5 m) on one side and 8 ft (2.4 m) on the other is considered a corner break. A corner break differs from a corner spall in that the crack extends vertically through the entire slab thickness, whereas a corner spall intersects the joint at an angle. Load repetition combined with loss of support and curling stresses usually cause corner breaks.

Severity Levels (Figure C–2)

L—Break is defined by a low-severity crack[a] and the area between the break and the joints is not cracked or may be lightly cracked.

M—Break is defined by a medium-severity crack[a] and/or the area between the break and the joints has a medium crack.

H—Break is defined by a high-severity crack[a] and/or the area between the break and the joints is highly cracked.

How to Count

Distressed slab is recorded as one slab if it:

1. Contains a single corner break.
2. Contains more than one break of a particular severity.
3. Contains two or more breaks of different severities. For two or more breaks, the highest level of severity should be recorded. For example, a slab containing both low and medium severity corner breaks should be counted as one slab with a medium corner break.

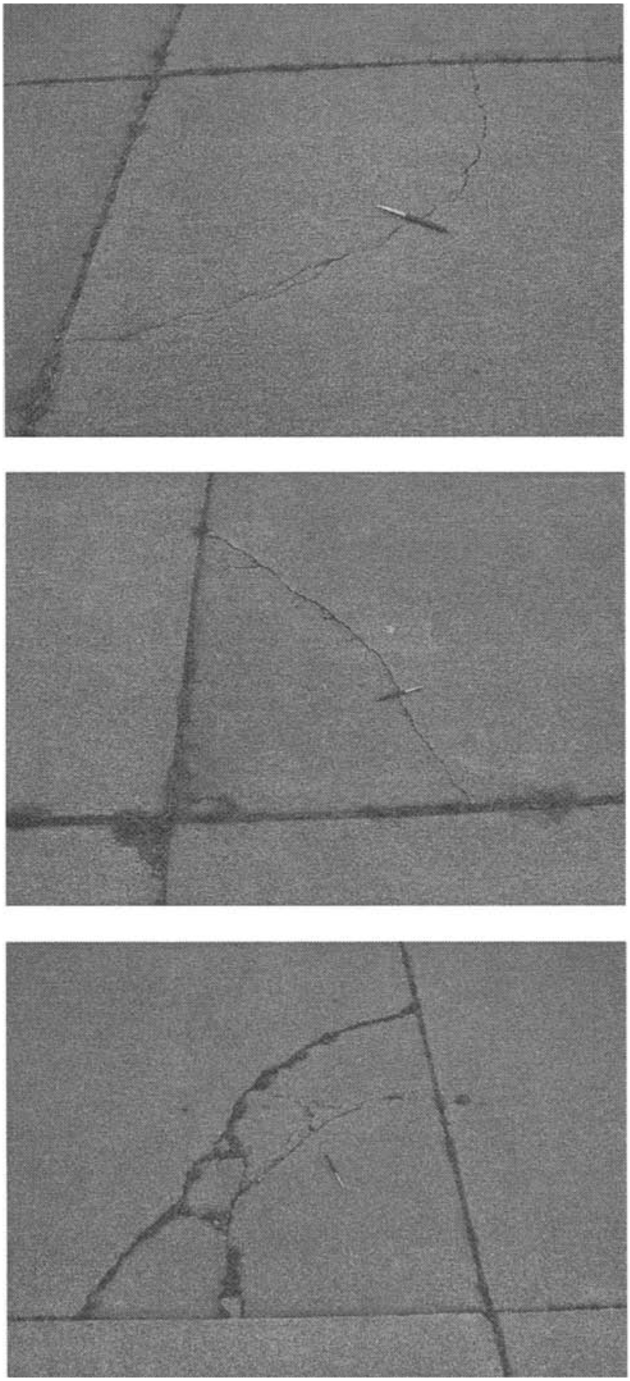
Options for Repair

L[b]—Do nothing; Seal cracks over 1/8 in. (3 mm).

M[b]—Seal cracks; Full-depth patch.

H[b]—Full-depth patch.

[a]See Linear Cracking for a definition of low, medium and high-severity cracks.
[b]Should check for loss of foundation support or voids under corners. If this condition exists, should consider subsealing and installing load transfer devices.



LOW

MEDIUM

HIGH

Figure C-2. Corner Break.

Divided Slab (23)

Description

Slab is divided by cracks into four or more pieces due to overloading and/or inadequate support. If all pieces or cracks are contained within a corner break, the distress is categorized as a severe corner break.

Severity Levels (Figures C–3)

Table C-1 lists severity levels for divided slabs.

How to Count

Table C-1. Levels of Severity for Divided Slabs.

Severity of Majority of Cracks	Number of Pieces in Cracked Slab		
	4 to 5	6 to 8	8 or more
L	L	L	M
M	M	M	H
H	M	H	H

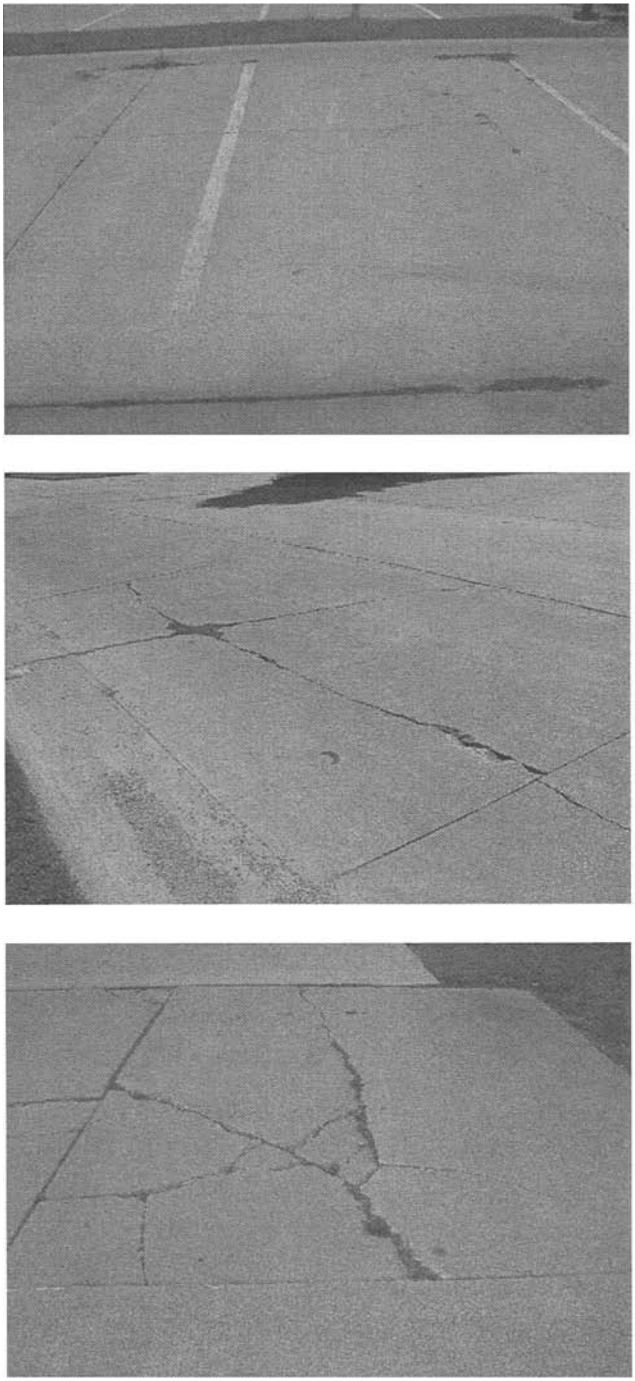
If the divided slab is medium or high-severity, no other distress is counted for that slab.

Options for Repair

L—Do nothing; Seal cracks more than 1/8 in. wide.

M—Replace slab.

H—Replace slab.



LOW

MEDIUM

HIGH

Figure C-3. Divided Slab.

Durability ("D") Cracking (24)

Description

"D" Cracking is caused by freeze-thaw expansion of the large aggregate which, over time, gradually breaks down the concrete. This distress usually appears as a pattern of cracks running parallel and close to a joint or linear crack. Since the concrete becomes saturated near joints and cracks, a dark-colored deposit can usually be found around fine "D" cracks. This type of distress may eventually lead to disintegration of the entire slab.

Severity Levels (Figure C–4)

L—"D" cracks cover less than 15 percent of slab area. Most of the cracks are tight, but a few pieces may be loose and or missing.

M—One of the following conditions exists:

1. "D" cracks cover less than 15 percent of the area and most of the pieces are loose and or missing.
2. "D" cracks cover more than 15 percent of the area. Most of the cracks are tight, but a few pieces may be loose and or missing.

H—"D" cracks cover more than 15 percent of the area and most of the pieces have come out or could be removed easily.

How to Count

When the distress is located and rated at one severity, it is counted as one slab. If more than one severity level exists, the slab is counted as having the higher severity distress. For example, if low and medium "D" cracking are on the same slab, the slab is counted as medium severity cracking only.

Options for Repair

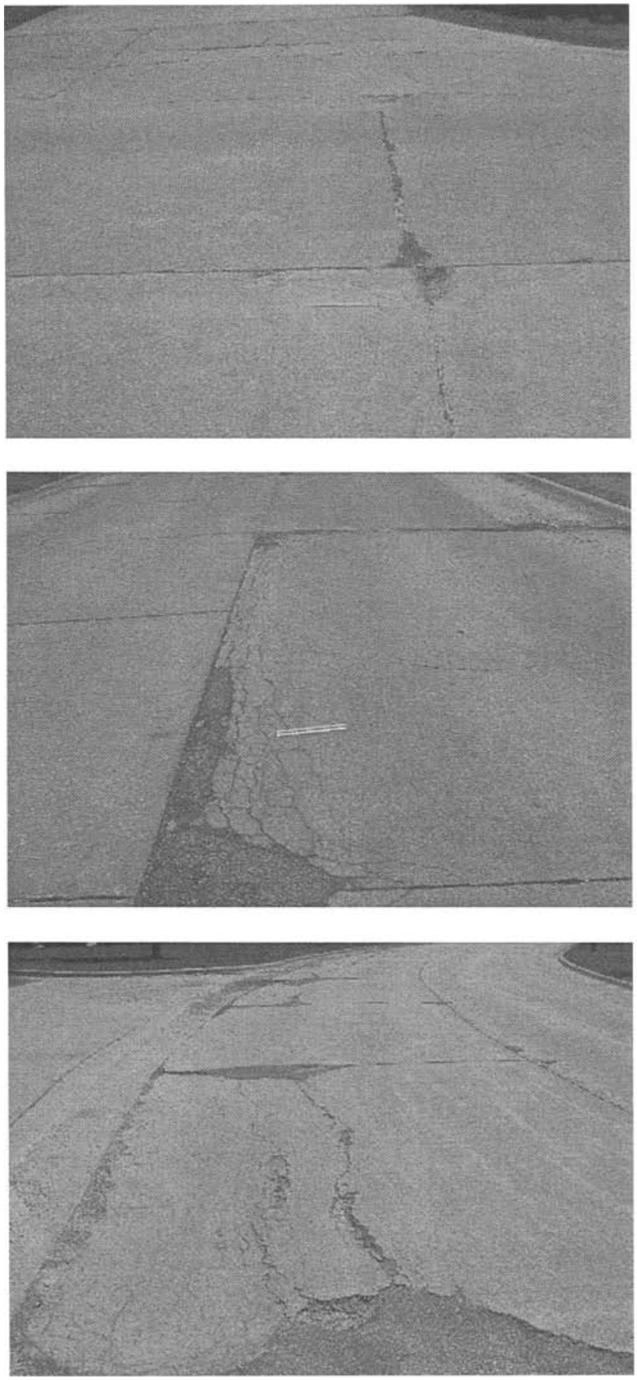
L—Do nothing.

M[a]—Full-depth patch; Reconstruct joints.

H[a]—Full-depth patch; Reconstruct joints; Slab replacement.

---

[a]Complete pavement reconstruction may be considered based on economics.



LOW

MEDIUM

HIGH

Figure C-4. Durability Cracking.

Faulting (25)

Description

Faulting is the difference in elevation across a joint. Some common causes of faulting are:

1. Settlement because of soft foundation.
2. Pumping or eroding of material from under the slab.
3. Curling of the slab edges due to temperature and moisture changes.

Severity Levels (Figure C–5)

Severity levels are defined by the difference in elevation across the joint as indicated in Table C-2.

Table C-2. Levels of Severity for Faulting.

Severity Level	Difference in Elevation
L	1/8 to 3/8 in. (3 to 10 mm)
M	>3/8 to 3/4 in. (10 to 19 mm)
H	>3/4 in. (>19 mm)

How to Count

Faulting across a joint is counted as one slab. Only affected slabs are counted. Faults across a crack are not counted as distress, but are considered when defining crack severity.

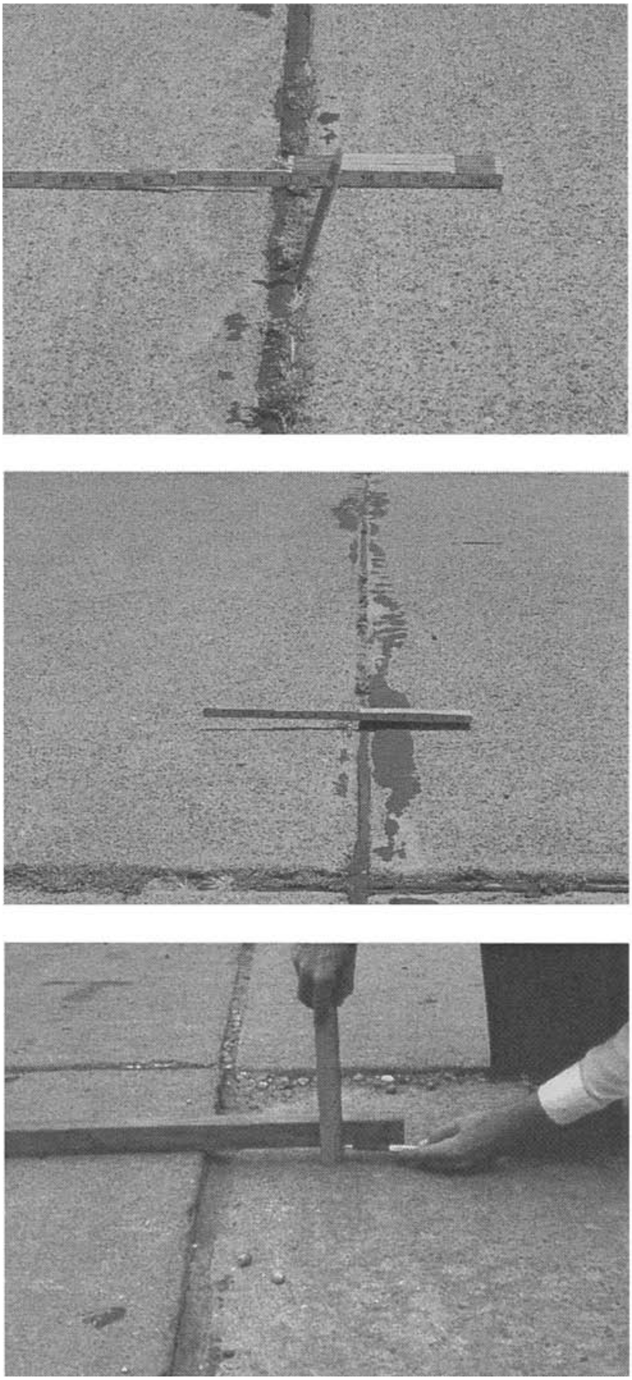
Options for Repair

L[a]—Do nothing; Grind.

M[a]—Grind.

H[a]—Grind.

[a]If faulting is caused by settlement or loss of support, then subsealing and installing load-transfer devices should be considered.



LOW

MEDIUM

HIGH

Figure C-5. Faulting.

Joint Seal Damage (26)

Description

Joint seal damage is any condition that enables soil or rocks to accumulate in the joints or allows significant water infiltration. Accumulation of incompressible materials prevents the slab from expanding and may result in buckling, shattering, or spalling. A pliable joint filler bonded to the edges of the slabs protects the joints from material accumulation and prevents water from seeping down and softening the foundation supporting the slab. Typical types of joint seal damage are:

1. Stripping of joint sealant.
2. Extrusion of joint sealant.
3. Weed growth.
4. Hardening of the filler (oxidation).
5. Loss of bond to the slab edges.
6. Lack or absence of sealant in the joint.

Severity Levels (Figure C–6)

L—Joint sealant is in generally good condition throughout section. Sealant is performing well, with only minor damage (see above).

M—Joint sealant is in generally fair condition over the entire section, with one or more of the above types of damage occurring to a moderate degree. Sealant needs replacement within 2 years.

H—Joint sealant is in generally poor condition over the entire section, with one or more of the above types of damage occurring to a severe degree. Sealant needs immediate replacement.

How to Count

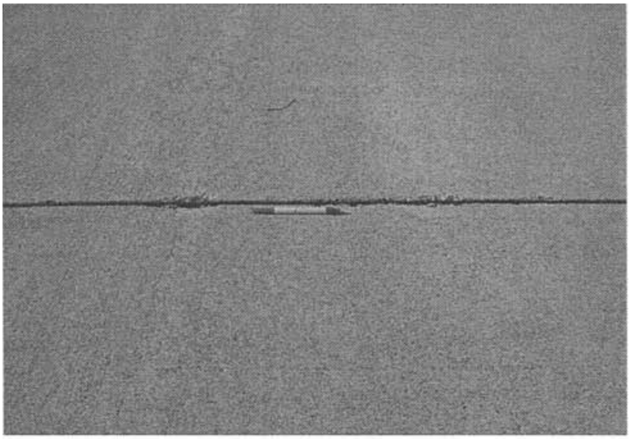
Joint seal damage is not counted on a slab by slab basis, but is rated based on the overall condition of the sealant over the entire area.

Options for Repair

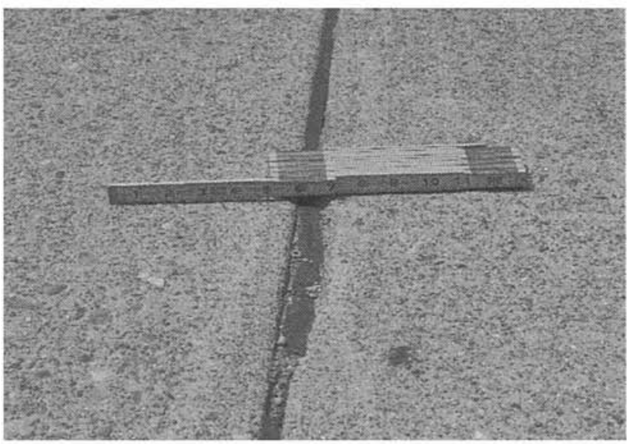
L—Do nothing.

M—Reseal joints.

H—Reseal joints.



LOW



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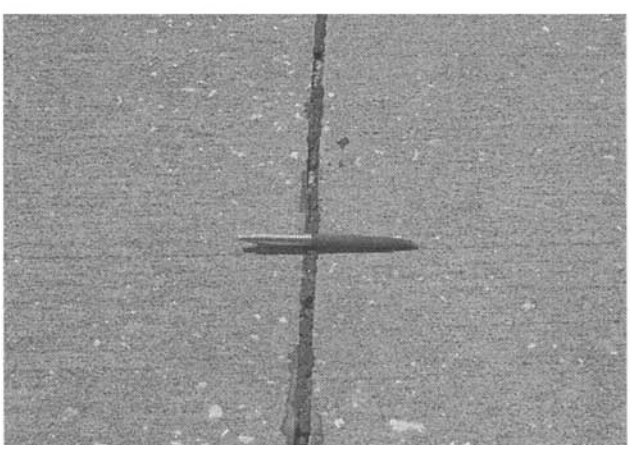


HIGH

Figure C-6. Joint Seal Damage.

Lane/Shoulder Drop-Off (27)

Description

Lane/shoulder drop-off is the difference between the settlement or erosion of the shoulder and the pavement travel-lane edge. The elevation difference can be a safety hazard; it can also cause increased water infiltration.

Severity Levels (Figure C–7)

L—The difference between the pavement edge and shoulder is 1 to 2 in. (25 to 51 mm).

M—The difference in elevation is 2 to 4 in. (51 to 102 mm).

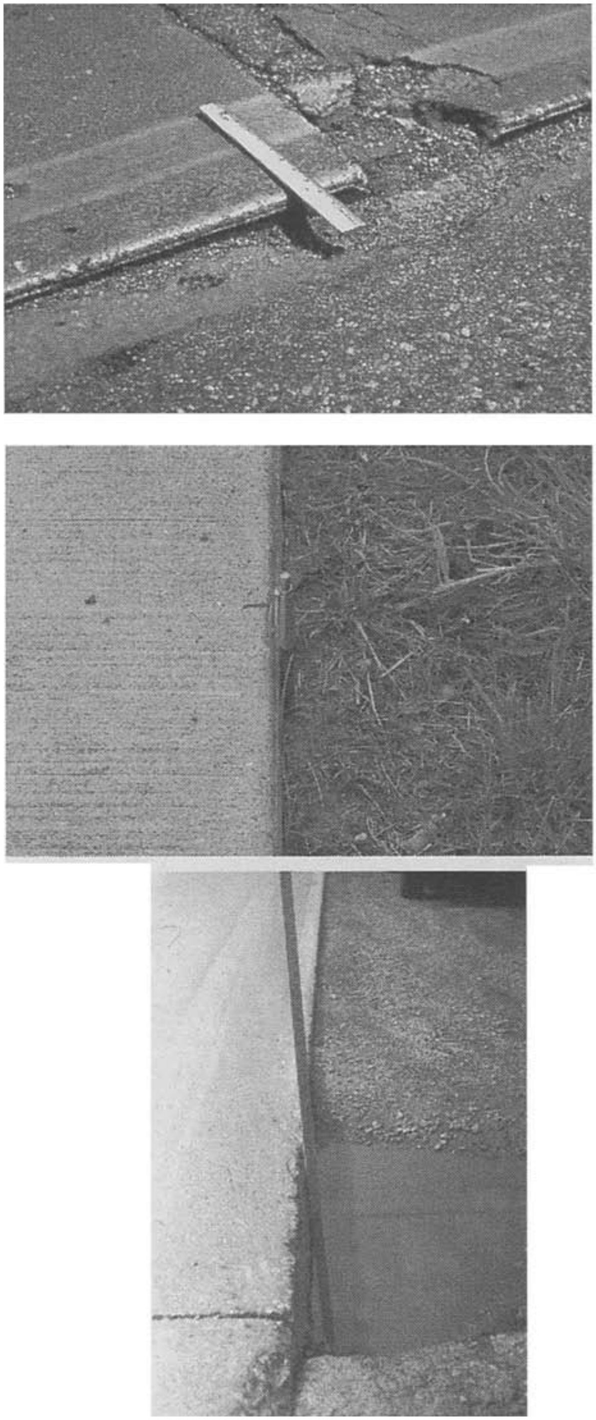
H—The difference in elevation is >4 in. (102 mm).

How to Count

The mean lane/shoulder drop off is computed by averaging the maximum and minimum drop along the slab. Each slab exhibiting distress is measured separately and counted as one slab with the appropriate severity level.

Options for Repair

L, M, H—Regrade and fill shoulders to match lane height.



LOW

MEDIUM

HIGH

Figure C-7. Lane/Shoulder Drop-Off.

Linear Cracking (28) (Longitudinal, Transverse, and Diagonal Cracks)

Description

These cracks, which divide the slab into two or three pieces, are usually caused by a combination of repeated traffic loading, thermal gradient curling, and repeated moisture loading. (Slabs divided into four or more pieces are counted as divided slabs). Low-severity cracks are usually related to warp or friction and are not considered major structural distresses. Medium or high-severity cracks are usually working cracks and are considered major structural distresses.

Hairline cracks that are only a few feet long and do not extend across the entire slab are counted as shrinkage cracks.

Severity Levels (Figure C–8)

Non-reinforced Slabs

L—Non-filled[a] cracks ≤ 1/2 in. (12 mm) or filled cracks of any width with the filler in satisfactory condition. No faulting exists.

M—One of the following conditions exists:

1. Non-filled crack with a width between 1/2 and 2 in. (12 and 51 mm).
2. Non-filled crack of any width up to 2 in. (51 mm) with faulting of 3/8 in. (10 mm).
3. Filled crack of any width with faulting <3/8 in. (10 mm)

H—One of the following conditions exists:

1. Non-filled crack with a width >2 in. (51 mm).
2. Filled or non-filled crack of any width with faulting >3/8 in. (10 mm)

Reinforced Slabs

L—Non-filled cracks 1/8 to 1 in. (3 to 25 mm) wide; filled crack of any width with the filler in satisfactory condition. No faulting exists.

M—One of the following conditions exists:

1. Non-filled cracks with a width between 1 and 3 in. (25 and 76 mm) and no faulting.
2. Non-filled crack of any width up to 3 in. (76 mm) with up to 3/8 in. (10 mm) of faulting.
3. Filled crack of any width with up to 3/8 in. (10 mm) faulting.

H—One of the following conditions exists:

1. Non-filled crack more than 3 in. (76 mm) wide.
2. Filled or non-filled crack of any width with faulting over 3/8 in. (10 mm).

---

[a]Filled cracks for which filler is unsatisfactory are treated as non-filled.

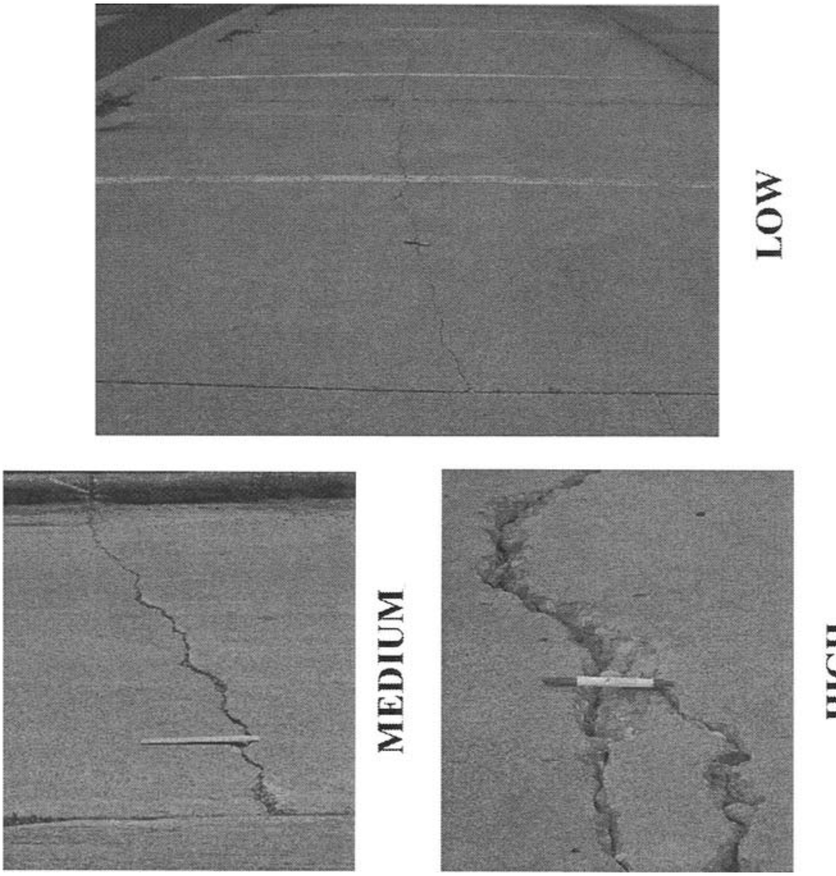


Figure C-8. Linear Cracking.

How to Count

Once the severity has been identified, the distress is recorded as one slab. If two medium severity cracks are within one slab, the slab is counted as having one high severity crack. Slabs divided into four or more pieces are counted as divided slabs. In reinforced slabs, cracks less than 1/8 in. (3 mm) wide are counted as shrinkage cracks.

Slabs longer than 30 ft (9.1 m) are divided into approximately equal length "slabs" having imaginary joints assumed to be in perfect condition.

Options for Repair

L—Do nothing; Seal cracks over 1/8 in.

M—Seal cracks.

H—Seal cracks; Full-depth patch; Slab replacement.

Patching, Large (More Than 5 sq ft [0.45 m^2]) and Utility Cuts (29)

Description

A patch is an area where the original pavement has been removed and replaced by filler material. A utility cut is a patch that has replaced the original pavement to allow the installation or maintenance of underground utilities. The severity levels of a utility cut are assessed according to the same criteria as regular patching.

Severity Levels (Figure C–9)

L—Patch is functioning well, with little or no deterioration.

M—Patch is moderately deteriorated and/or moderate spalling can be seen around the edges. Patch material can be dislodged with considerable effort.

H—Patch is badly deteriorated. The extent of the deterioration warrants replacement .

How to Count

If a single slab has one or more patches with the same severity level, it is counted as one slab containing that distress. If a single slab has more than one severity level, it is counted as one slab with the higher severity level.

Options for Repair

L—Do nothing.

M—Seal cracks; Replace patch.

H—Replace patch.

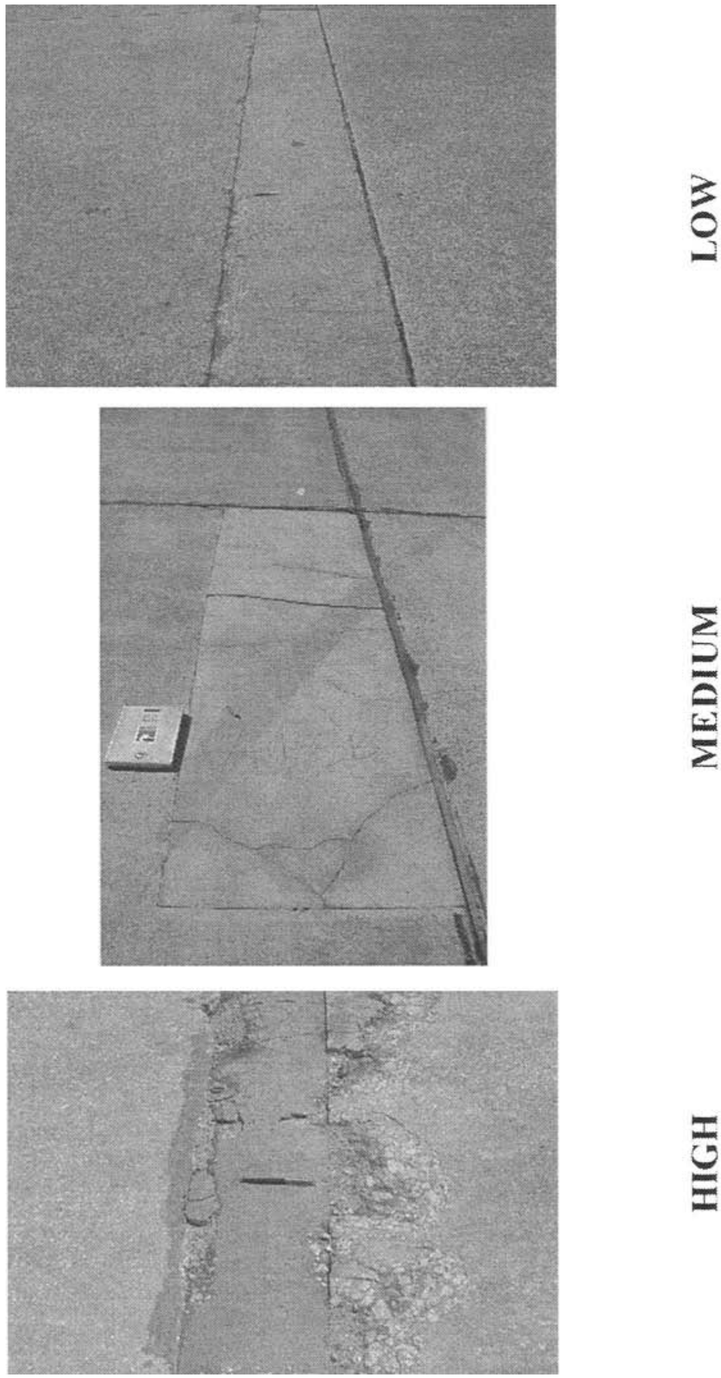


Figure C-9. Patching, Large and Utility Cuts.

Patching, Small (Less than 5 sq ft [0.45 m^2]) (30)

Description

A patch is an area where the original pavement has been removed and replaced by a filler material.

Severity Levels (Figure C–10)

L—Patch is functioning well with little or no deterioration.

M—Patch is moderately deteriorated. Patch material can be dislodged with considerable effort.

H—Patch is badly deteriorated. The extent of deterioration warrants replacement.

How to Count

If a single slab has one or more patches with the same severity level, it is counted as one slab containing that distress. If a single slab has more than one severity level, it is counted as one slab with the higher severity level.

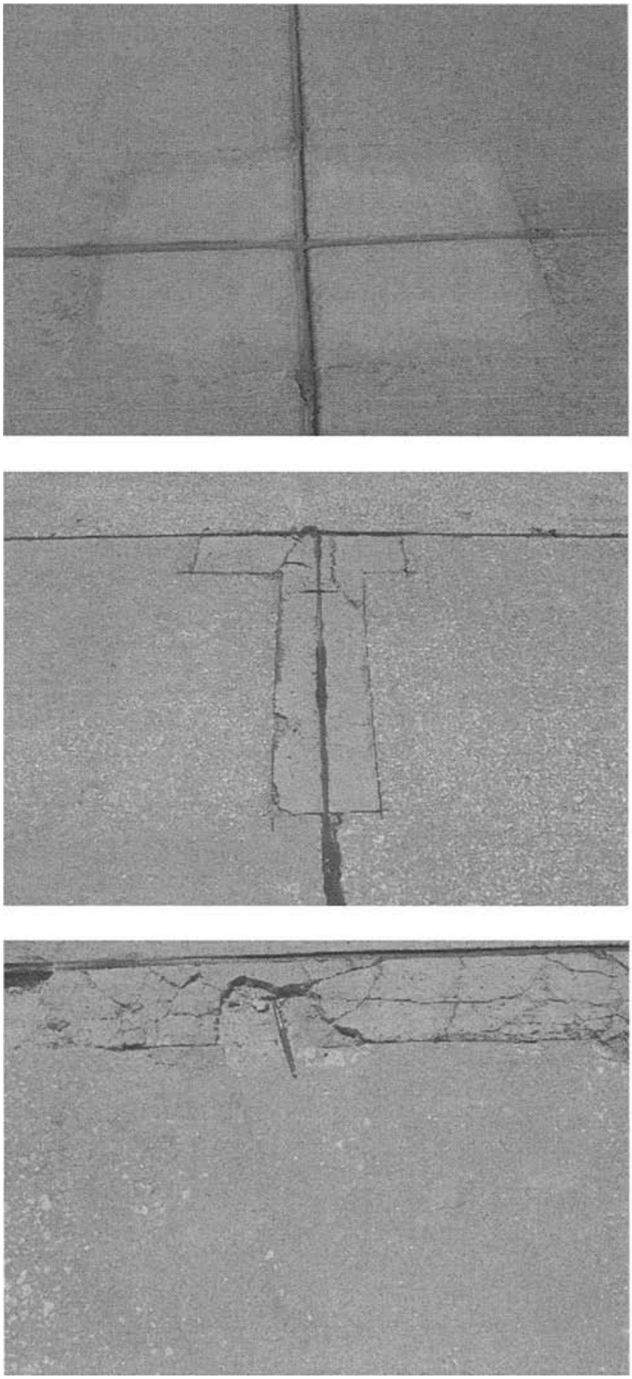
If the cause of the patch is more severe, only the original distress is counted.

Options for Repair

L—Do nothing.

M—Do nothing; Replace patch.

H—Replace patch.



LOW

MEDIUM

HIGH

Figure C-10. Patching, Small.

Polished Aggregate (31)

Description

This distress is caused by repeated traffic applications. When the aggregate in the surface becomes smooth to the touch, adhesion with the vehicle tires is considerably reduced. When the portion of aggregate extending above the surface is small, the pavement texture does not significantly contribute to reducing vehicle speed. Polished aggregate extending above the asphalt is negligible, and the surface aggregate is smooth to the touch. This type of distress is indicated when the number on a skid resistance test is low or has dropped significantly from previous ratings.

Severity Levels (Figure C–11)

No degrees of severity are defined. However, the degree of polishing should be significant before it is included in the condition survey and rated as a defect.

How to Count

A slab with polished aggregate is counted as one slab.

Options for Repair

L, M, H—Grove surface; Overlay.

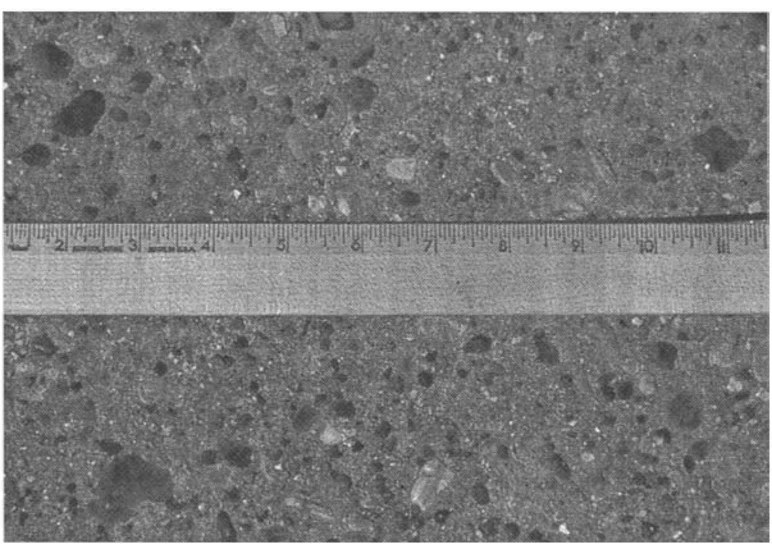


Figure C-11. Polished Aggregate.

Popouts (32)

Description

A popout is a small piece of pavement that breaks loose from the surface due to freeze thaw action, combined with expansive aggregates. Popouts usually range in diameter from approximately 1 to 4 in.(25 to 102 mm) and in depth from 1/2 to 2 in. (13 to 51 mm).

Severity Levels (Figure C–12)

No degrees of severity are defined for popouts. However, popouts must be extensive before they are counted as a distress. Average popout density must exceed approximately three popouts per square yard over the entire slab area.

How to Count

The density of the distress must be measured. If there is any doubt that the average is greater than three popouts per square yard, at least three random 1 sq yd (0.84 m^2) areas should be checked. When the average is greater than this density, the slab should be counted.

Options for Repair

L, M, H—Do nothing.

Figure C-12. Popouts.

Pumping (33)

Description

Pumping is the ejection of material from the slab foundation through joints or cracks. This is caused by deflection of the slab with passing loads. As a load moves across the joint between the slabs, water is first forced under the leading slab, and then forced back under the trailing slab. This action erodes and eventually removes soil particles, resulting in progressive loss of pavement support. Pumping can be identified by surface stains and evidence of base or subgrade material on the pavement close to joints or cracks. Pumping near joints is caused by poor joint sealer and indicates loss of support; repeated loading will eventually produce cracks. Pumping can also occur along the slab edge, causing loss of support.

Severity Levels (Figure C–13)

No degrees of severity are defined. It is enough to indicate that pumping exists.

How to Count

One pumping joint between two slabs is counted as two slabs. However, if the remaining joints around the slab are also pumping, one slab is added per additional pumping joint.

Options for Repair

L, M, H—Underseal; Joint and crack seal; Restore load transfer.

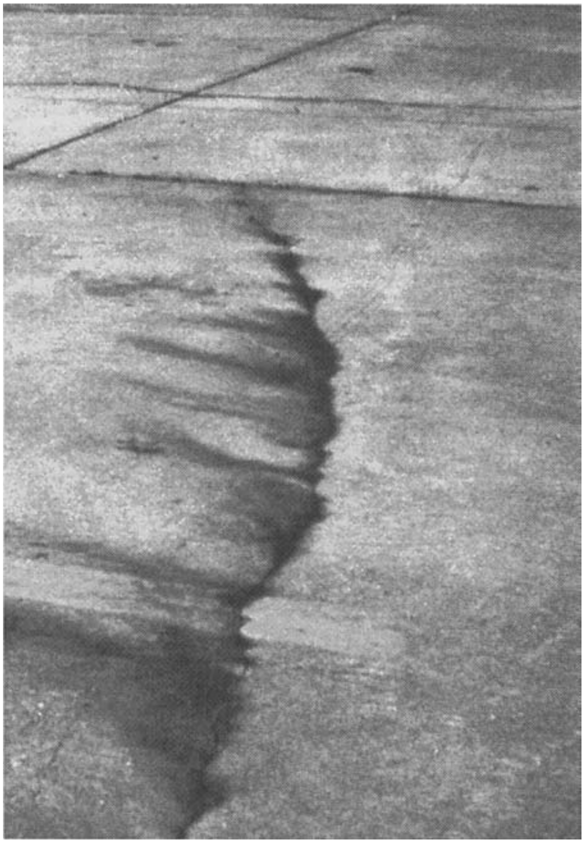


Figure C-13. Pumping.

Punchout (34)

Description

This distress is a localized area of the slab that is broken into pieces. The punchout can take many different shapes and forms, but it is usually defined by a crack and a joint, or two closely spaced cracks (usually 5 ft [1.52 m] wide). This distress is caused by heavy repeated loads, inadequate slab thickness, loss of foundation support, and/or a localized concrete construction deficiency (e.g., honeycombing).

Severity Levels (Figure C–14)

Table C-3 lists the severity levels for punchouts

How to Count

Table C-3. Levels of Severity for Punchouts.

Severity of Majority of Cracks	Number of Pieces		
	2 to 3	4 to 5	>5
L	L	L	M
M	L	M	H
H	M	H	H

If a slab contains one or more punchouts, it is counted as containing a punchout at the severity level of the most severe punchout.

Options for Repair

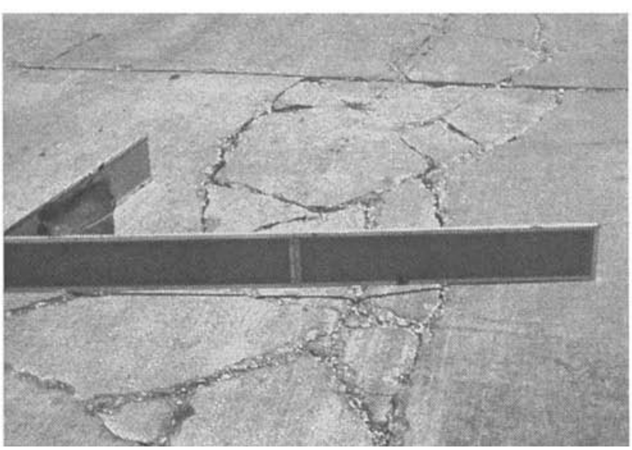
L—Do nothing; Seal cracks.

M—Full-depth patch.

H—Full-depth patch.

LOW

MEDIUM



HIGH

Figure C-14. Punchout.

Railroad Crossing (35)

Description

Railroad crossing distress is characterized by depressions or bumps around the tracks.

Severity Levels (Figure C–15)

L—Railroad crossing causes low severity ride quality.

M—Railroad crossing causes medium severity ride quality.

H—Railroad crossing causes high severity ride quality.

How to Count

The number of slabs crossed by the railroad tracks is counted. Any large bump created by the tracks should be counted as part of the crossing.

Options for Repair

L—Do nothing.

M—Partial-depth patch approach; Reconstruct crossing.

H—Partial-depth patch approach; Reconstruct crossing.

LOW

MEDIUM

HIGH

Figure C-15. Railroad Crossing.

Scaling, Map Cracking, and Crazing (36)

Description

Map cracking or crazing refers to a network of shallow, fine, or hairline cracks that extend only through the upper surface of the concrete. The cracks tend to intersect at angles of 120 degrees. Map cracking or crazing is usually caused by concrete over-finishing, and may lead to surface scaling, which is the breakdown of the slab surface to a depth of approximately 1/4 to 1/2 in. (6 to 13 mm). Scaling may also be caused by deicing salts, improper construction, freeze thaw cycles, and poor aggregate. If scaling is caused by "D" cracking, it should be counted under that distress only.

Severity Levels (Figure C–16)

L—Crazing or map cracking exists over most of the slab area; the surface is in good condition, with only minor scaling present.

M—Slab is scaled, but less than 15 percent of the slab is affected.

H—Slab is scaled over more than 15 percent of its area.

How to Count

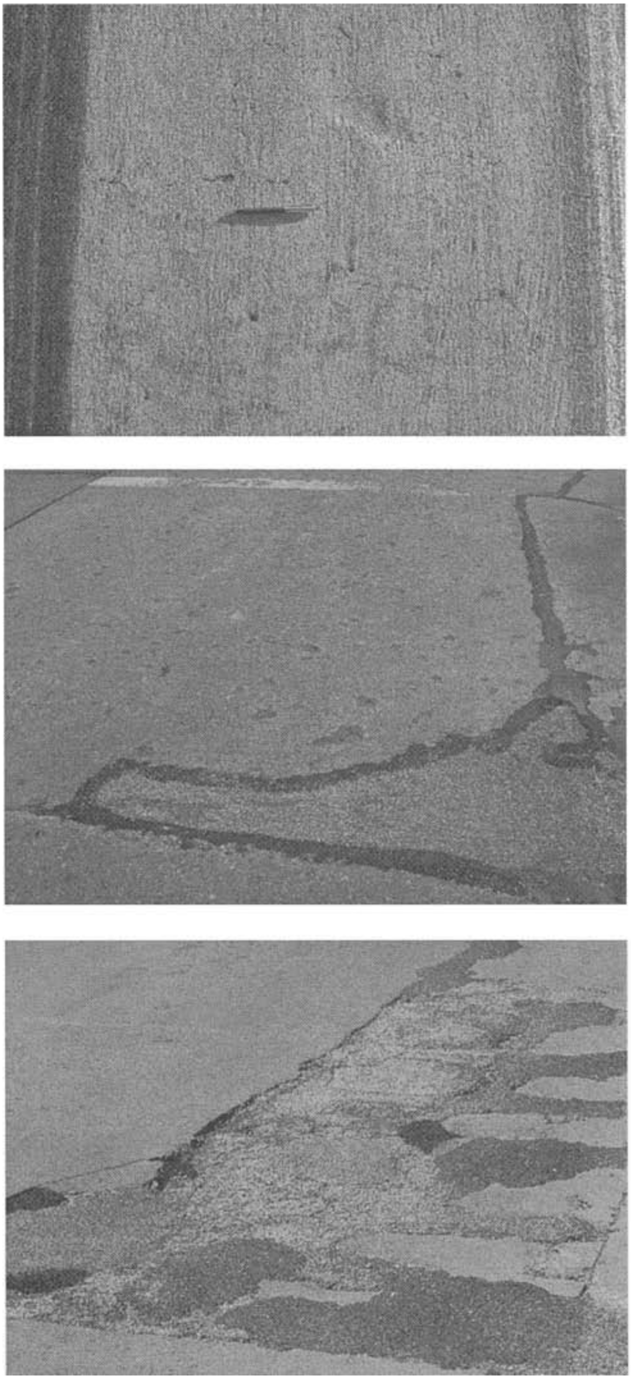
A scaled slab is counted as one slab. Low-severity crazing should only be counted if the potential for scaling appears to be imminent, or a few small pieces come out.

Options for Repair

L—Do nothing.

M—Do nothing; Slab replacement.

H—Partial or full-depth patch; Slab replacement; Overlay.



LOW

MEDIUM

HIGH

Figure C-16. Scaling, Map Cracking, and Crazing.

Shrinkage Cracks (37)

Description

Shrinkage cracks are hairline cracks that are usually less than 2m long and do not extend across the entire slab. They are formed during the setting and curing of the concrete and usually do not extend through the depth of the slab.

Severity Levels (Figure C–17)

No degrees of severity are defined. It is enough to indicate that shrinkage cracks are present.

How to Count

If any shrinkage cracks exist on a particular slab, the slab is counted as one slab with shrinkage cracks.

Options for Repair

L, M, H—Do nothing.

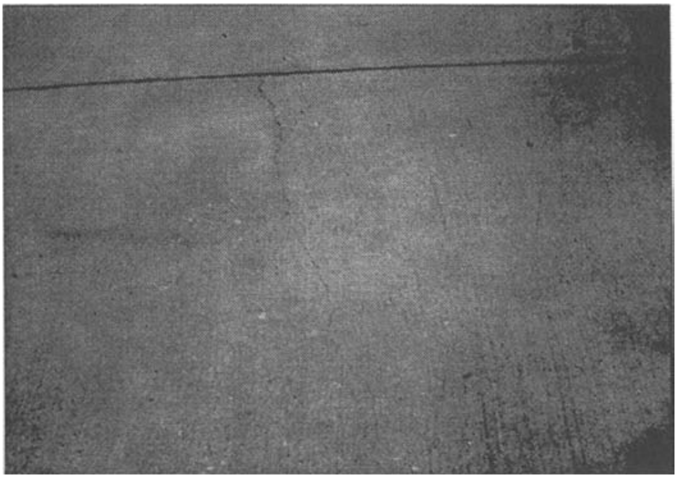


Figure C-17. Shrinkage Cracks.

Spalling, Corner (38)

Description

Corner spalling is the breakdown of the slab within approximately 2 ft (0.6 m) of th corner. A corner spall differs from a corner break in that the spall usually angles dowr ward to intersect the joint, whereas a break extends vertically through the slab corne Spalls less than 5 in. (127 mm) from the crack to the corner on both sides should not b counted.

Severity Levels (Figure C–18)

Table C-4 lists the levels of severity for corner spalling. Corner spalling with an area (less than 10 sq. in. (6452 mm^2) from the crack to the corner on both sides should not b counted.

Table C-4. Levels of Severity for Corner Spalling.

	Dimensions of Side Spall	
Depth of Spall	5 x 5 in. to 12 x 12 in. (125 x 127 mm) to (305 x 305 mm)	>12 x 12 in. (305 x 305 mm)
<1 in. (25 mm)	L	L
>1 to 2 in. (>25 to 51 mm)	L	M
>2 in. (51 mm)	M	H

How to Count

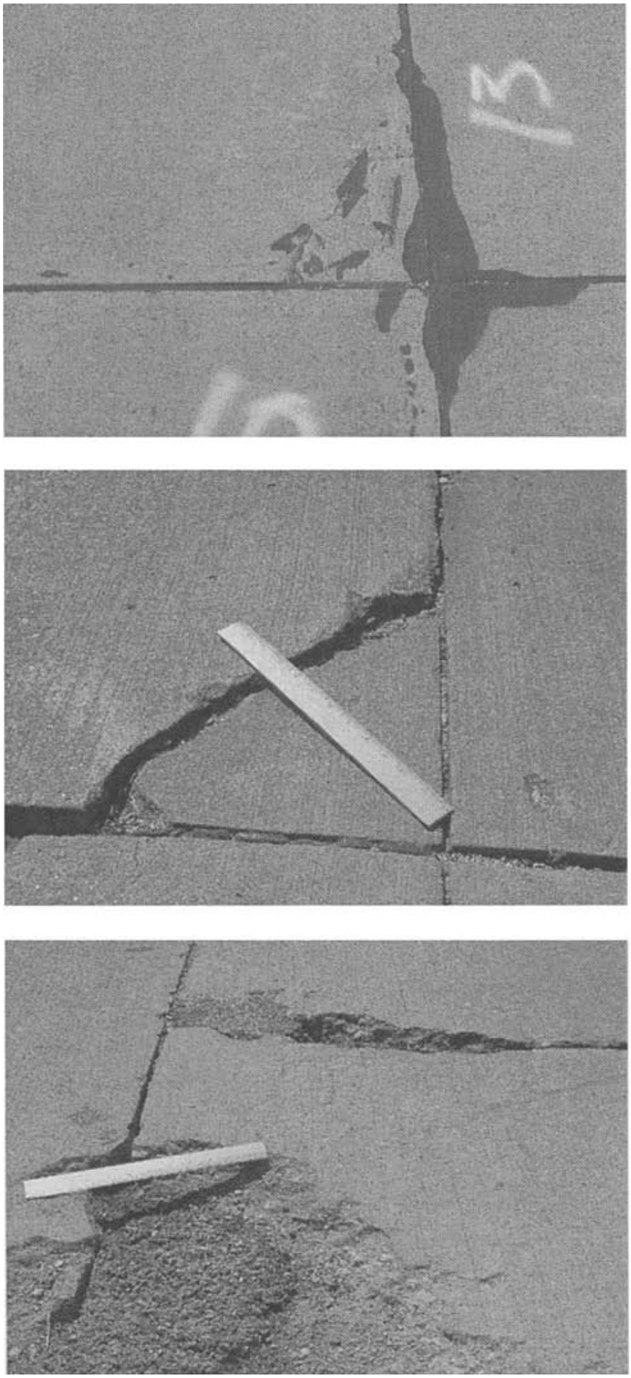
If one or more corner spalls with the same severity level are in a slab, the slab is counte as one slab with corner spalling. If more than one severity level occurs, it is counted a one slab with the higher severity level.

Options for Repair

L—Do nothing.

M—Partial-depth patch.

H—Partial-depth patch.



LOW

MEDIUM

HIGH

Figure C-18. Spalling, Corner.

Spalling, Joint (39)

Description

Joint spalling is the breakdown of the slab edges within 2 ft (0.6 m) of the joint. A joint spall usually does not extend vertically through the slab, but intersects the joint at an angle. Spalling results from:

1. Excessive stresses at the joint caused by traffic loading or by infiltration of incompressible materials.
2. Weak concrete at the joint caused by overworking.
3. Water accumulation in the joint and freeze thaw action.

Severity Levels (Figure C–19)

Table C-5 shows the severity levels of joint spalling. A frayed joint where the concrete has been worn away along the entire joint is rated as low-severity.

How to Count

Table C-5. Levels of Severity for Joint Spalling.

		Length of Spall	
Spall Pieces	Width of Spall	<2 ft (0.6 mm)	>2 ft (0.6 m)
Tight--cannot be easily removed (may be a few pieces missing)	< 4 in. (102 mm)	L	L
	> 4 in.	L	L
Loose--can be removed and some pieces are missing; if most or all pieces are missing, spall is shallow, less than 1 in. (25 mm)	< 4 in.	L	M
	> 4 in.	L	M
Missing--most or all pieces have been removed	< 4 in.	L	M
	> 4 in.	M	H

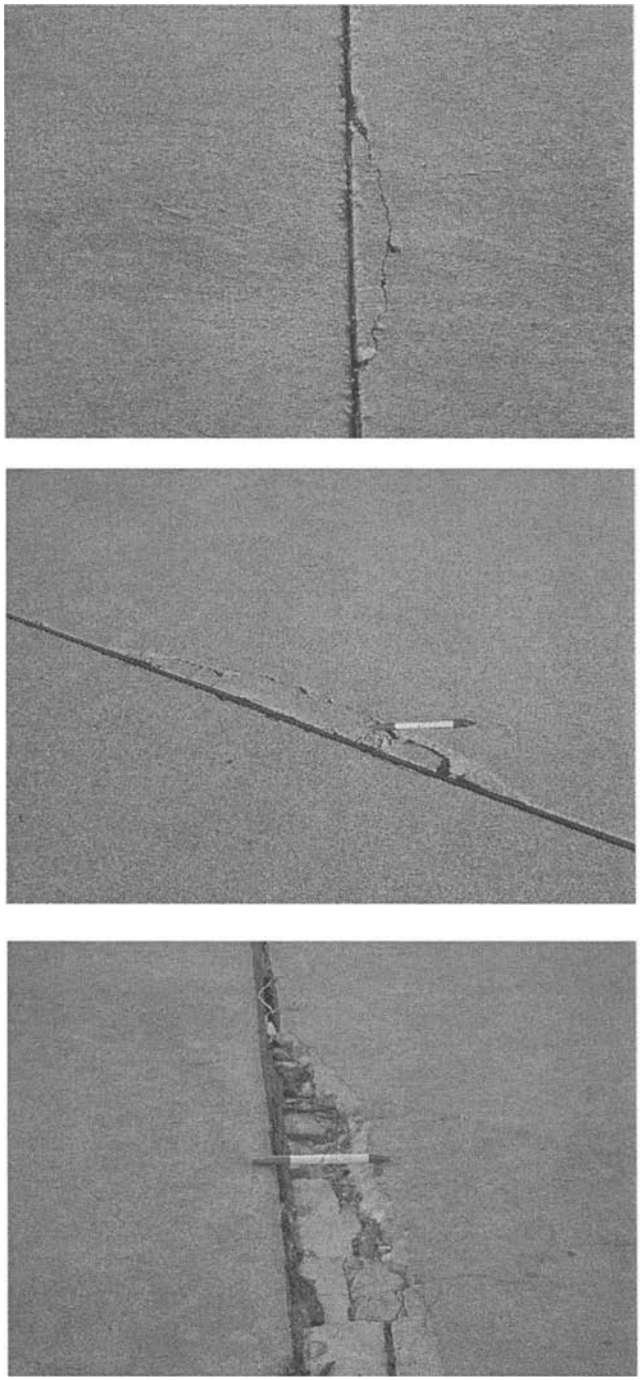
If spall is along the edge of one slab, it is counted as one slab with joint spalling. If spalling is on more than one edge of the same slab, the edge having the highest severity is counted and recorded as one slab. Joint spalling can also occur along the edges of two adjacent slabs. If this is the case, each slab is counted as having joint spalling.

Options for Repair

L—Do nothing.

M—Partial-depth patch.

H—Partial-depth patch; Reconstruct joint.



LOW

MEDIUM

HIGH

Figure C-19. Spalling, Joint.

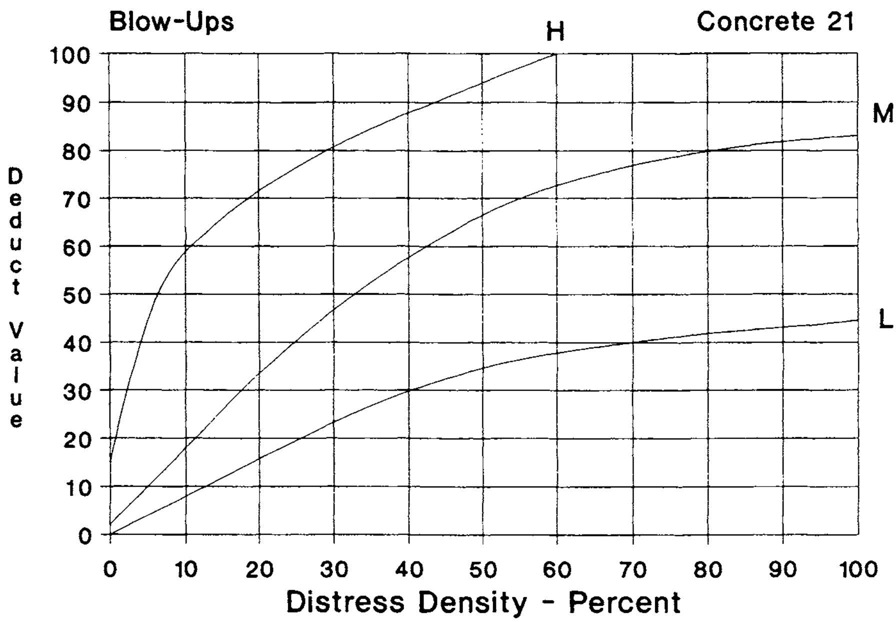


Figure C-20. Blowups.

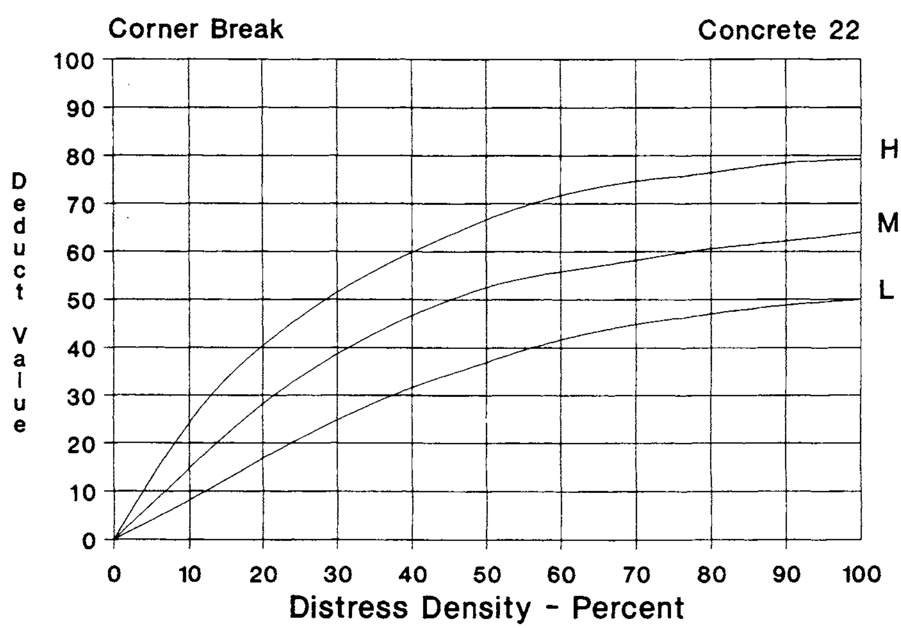


Figure C-21. Corner Break.

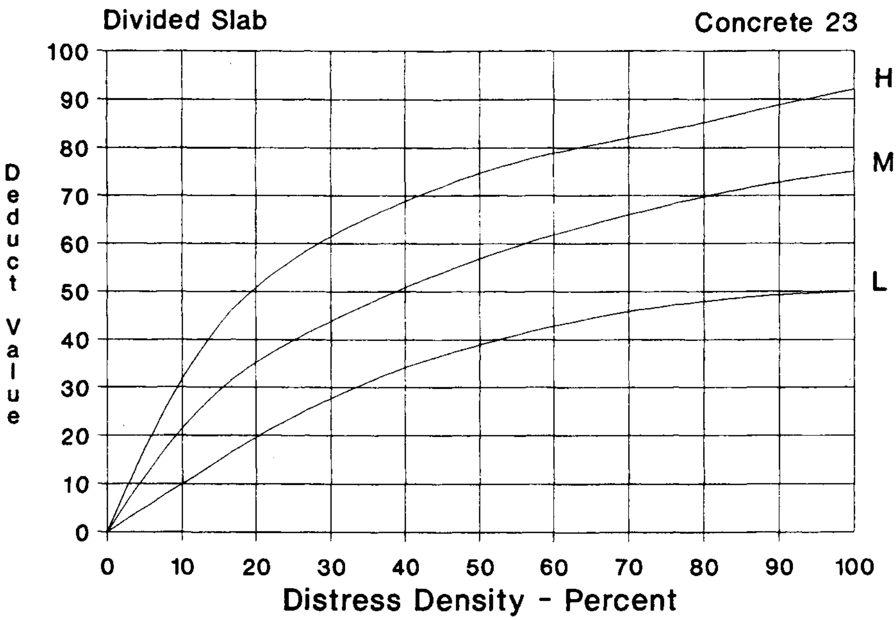


Figure C-22. Divided Slab.

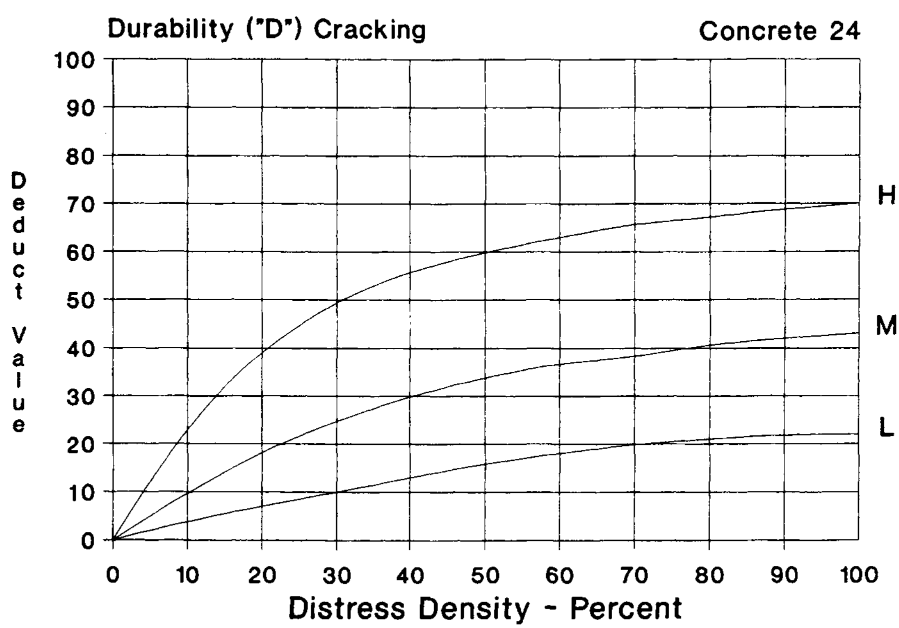


Figure C-23. Durability ("D") Cracking.

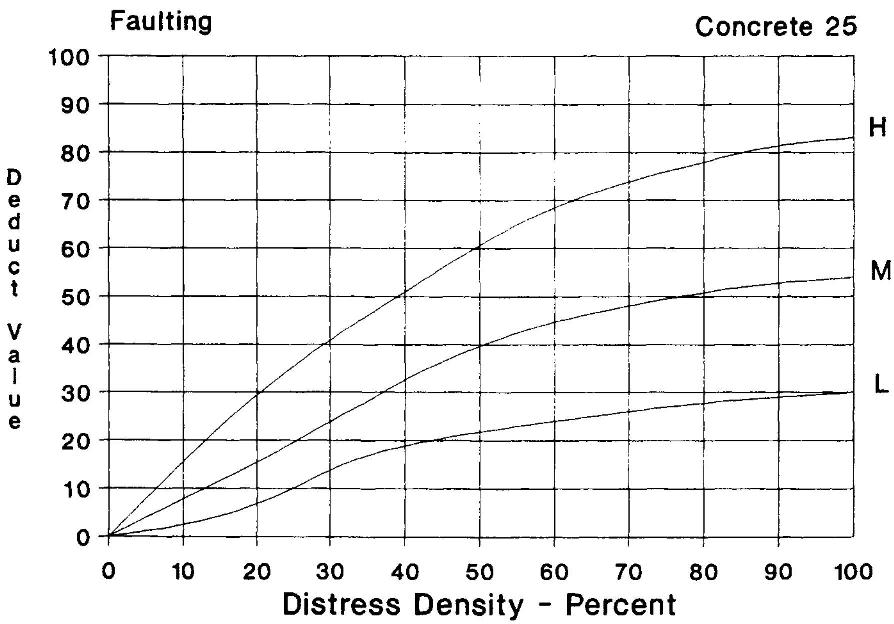


Figure C-24. Faulting.

JOINT SEAL DAMAGE **CONCRETE 26**

Joint seal damage is not rate by density. The severity of the distress is determined by the sealant's overall condition for a particular sample unit.

The deduct values for the three levels of severity are:

1. **High Severity - 8 Points**
2. **Medium Severity - 4 Points**
3. **Low Severity - 2 Points**

Figure C-25. Rigid Pavement Deduct Values. Distress 26. Joint Seal Damage.

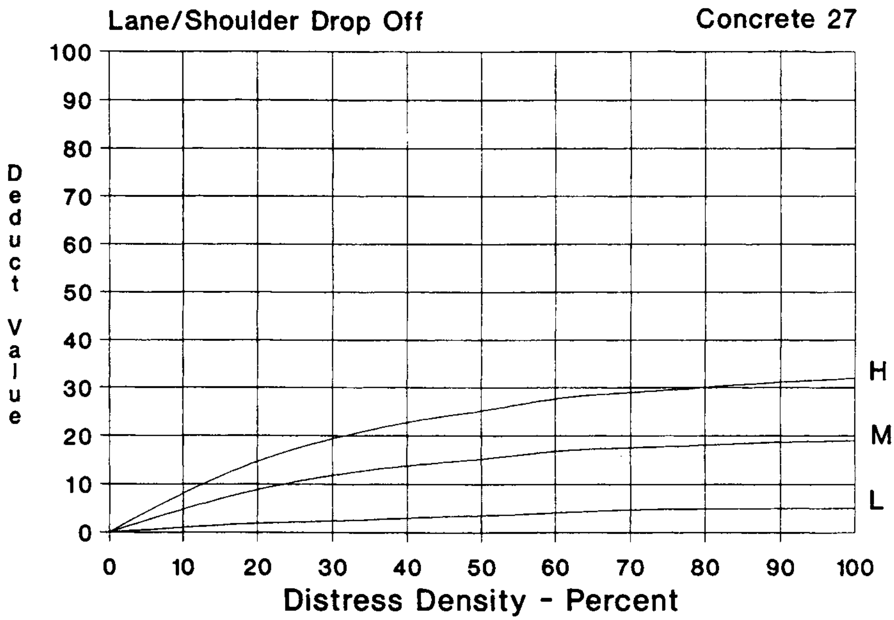


Figure C-26. Lane/Shoulder Drop-off.

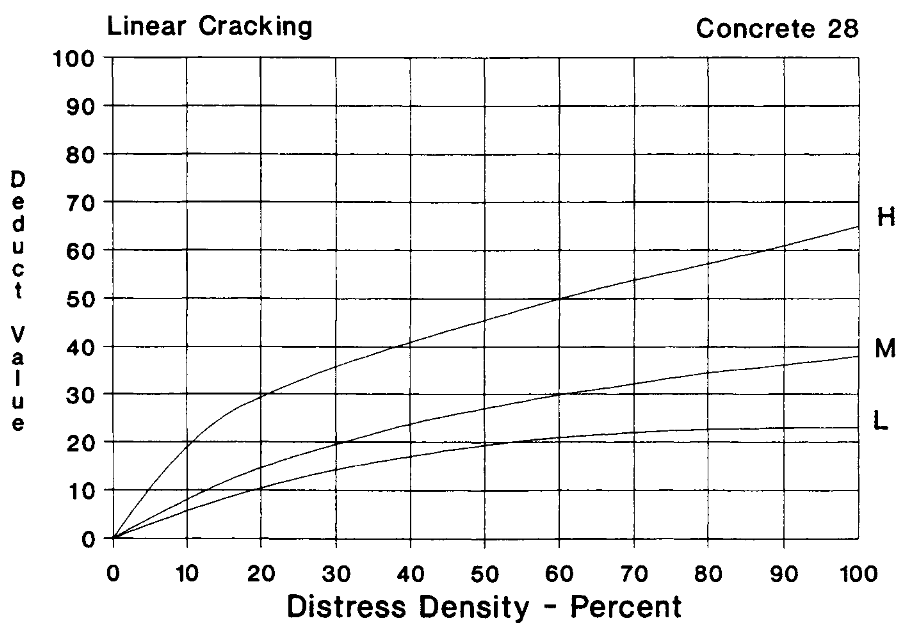


Figure C-27. Linear Cracking.

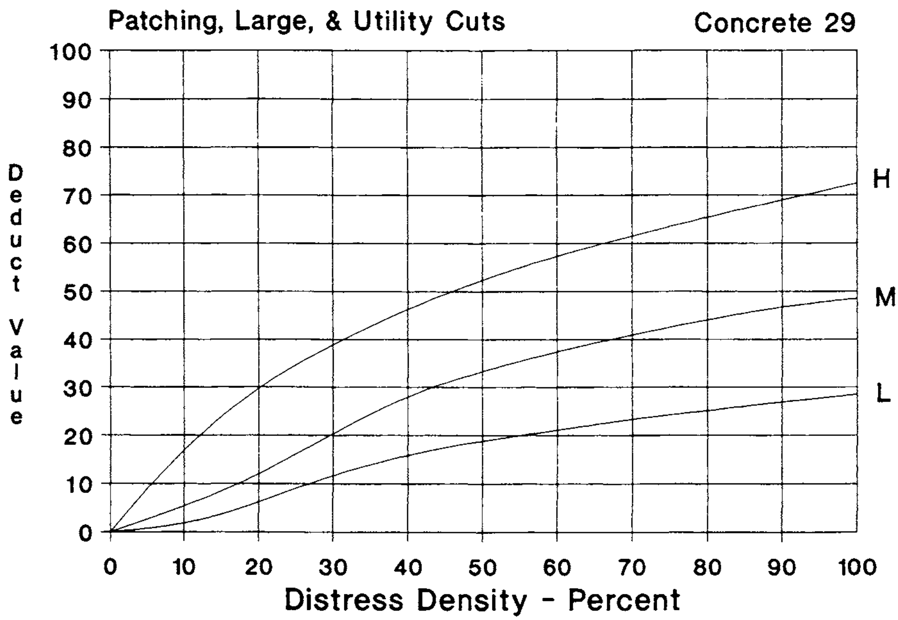


Figure C-28. Patching, Large, and Utility Cuts.

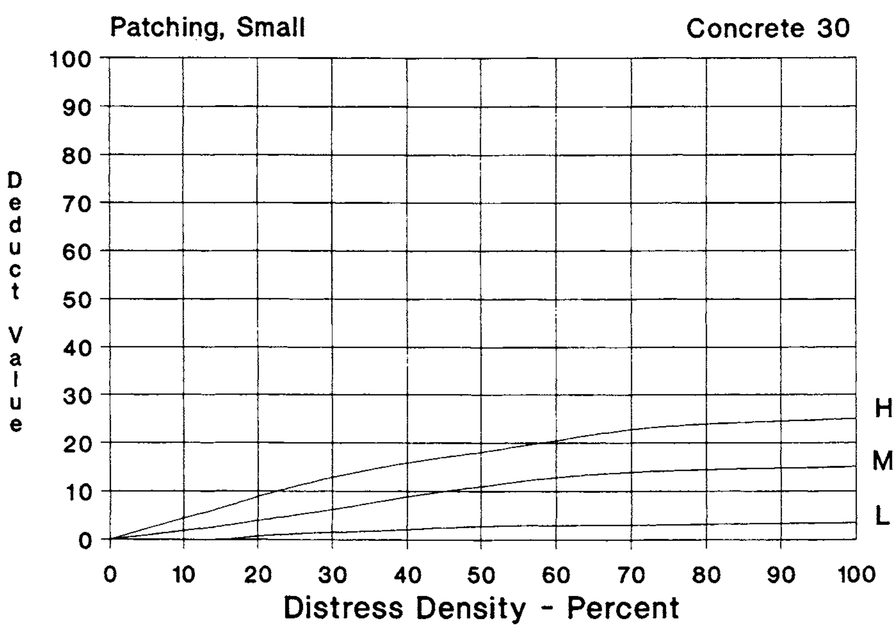


Figure C-29. Patching, Small.

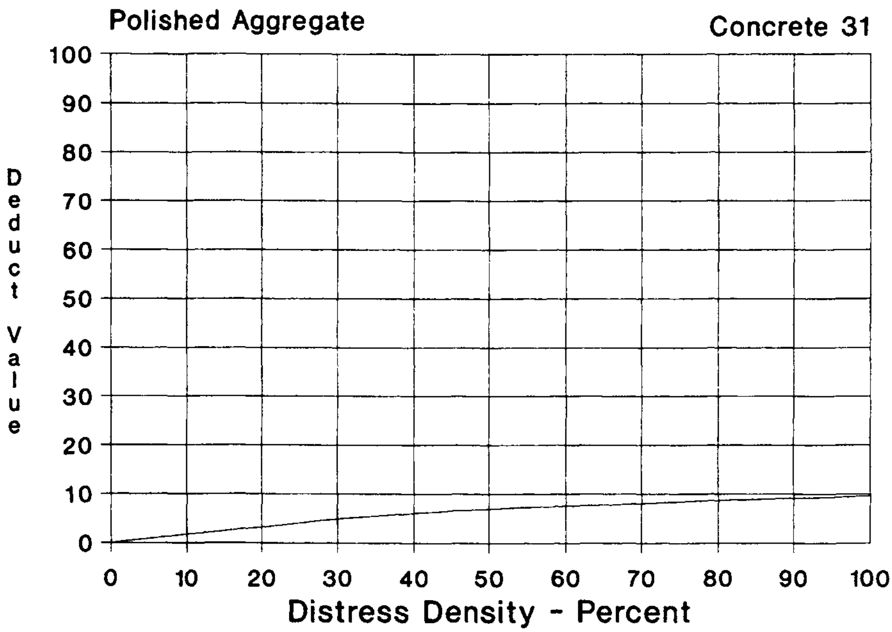


Figure C-30. Polished Aggregate.

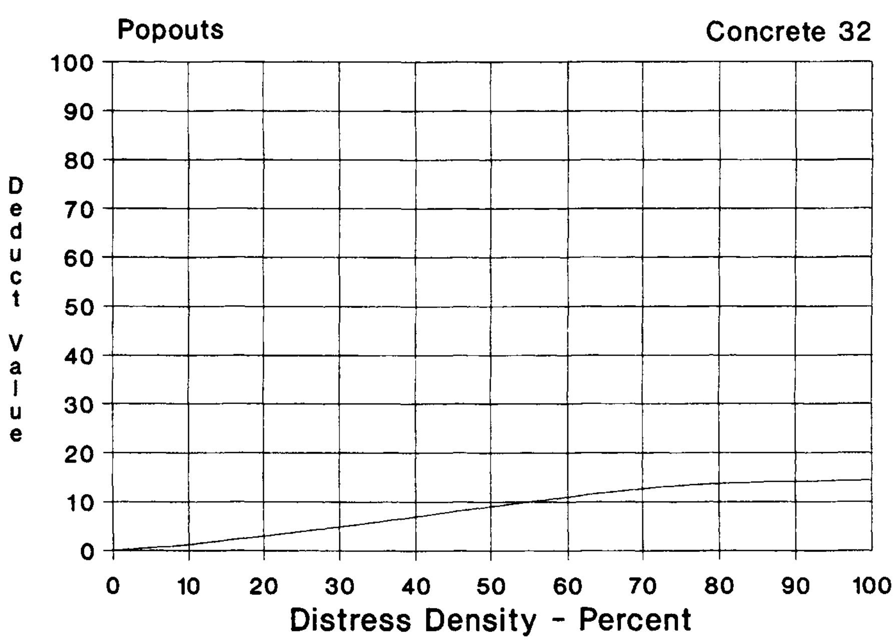


Figure C-31. Popouts.

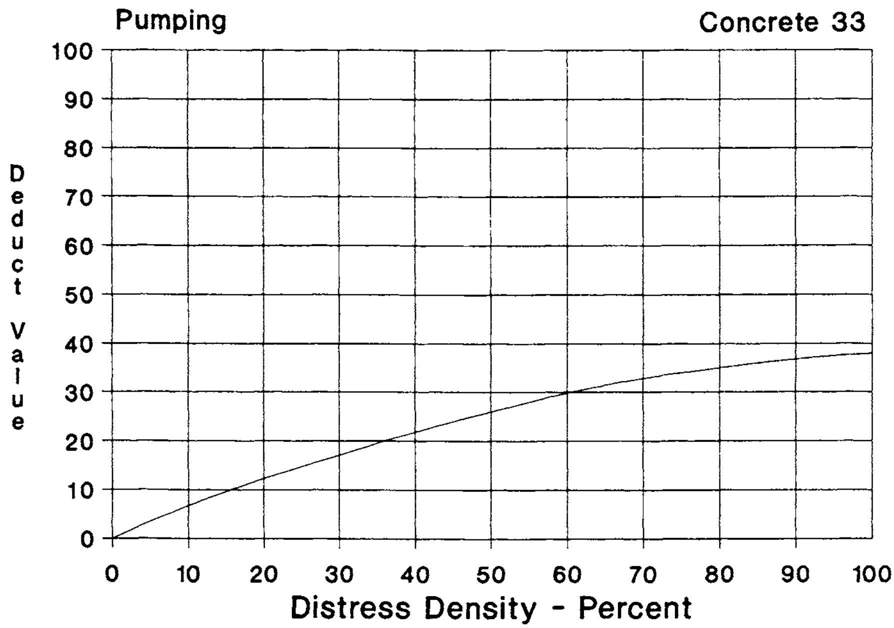


Figure C-32. Pumping.

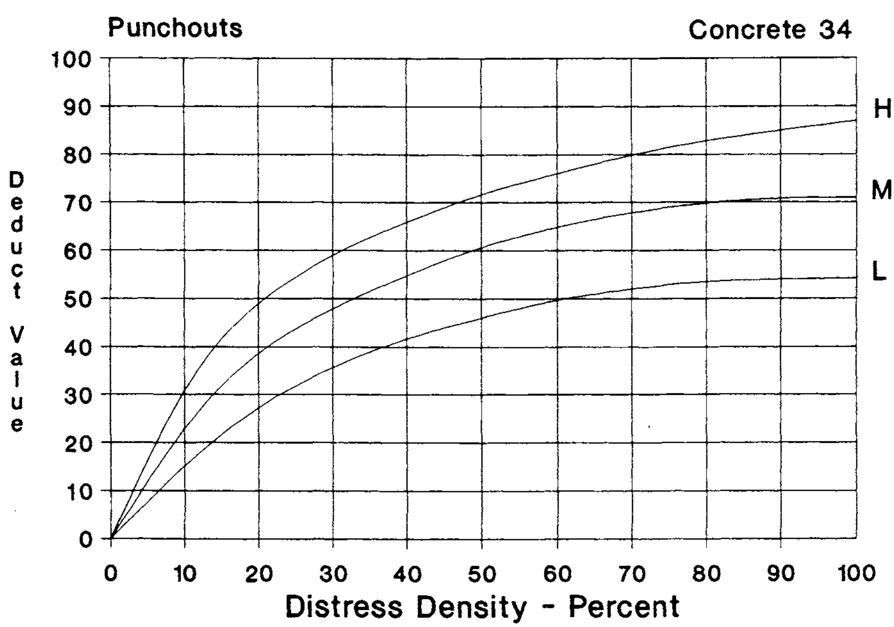


Figure C-33. Punchouts.

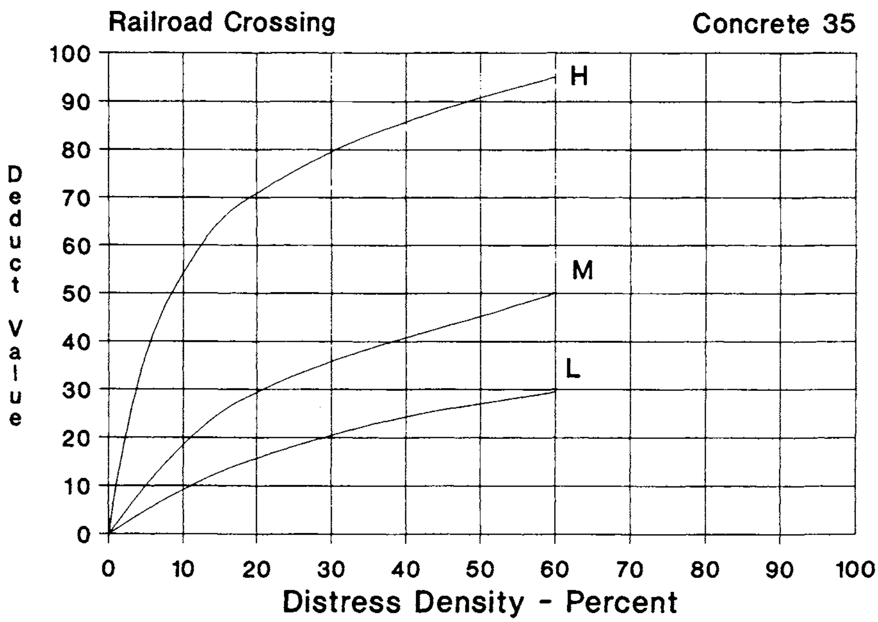


Figure C-34. Railroad Crossing.

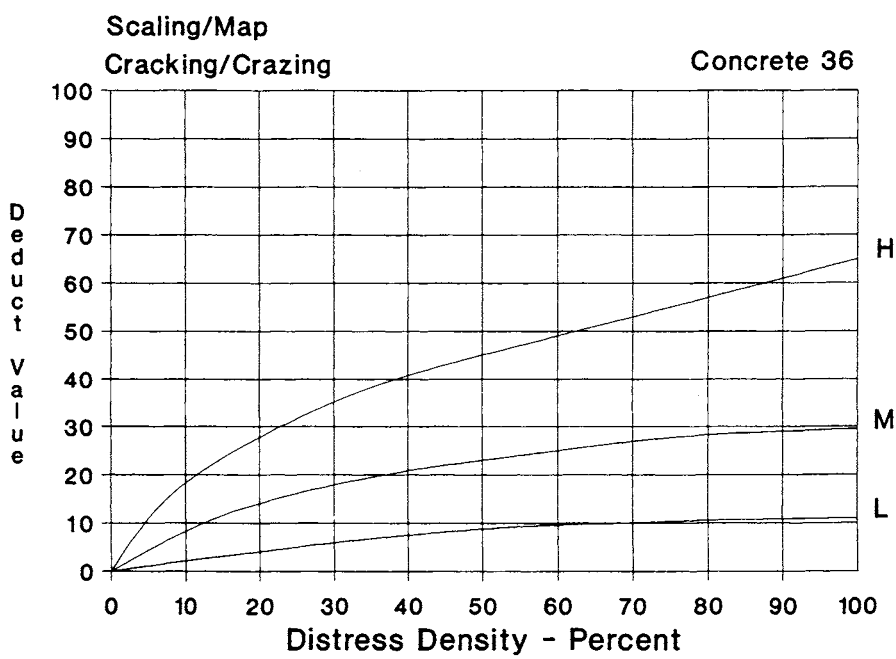


Figure C-35. Scaling/Map Cracking/Crazing.

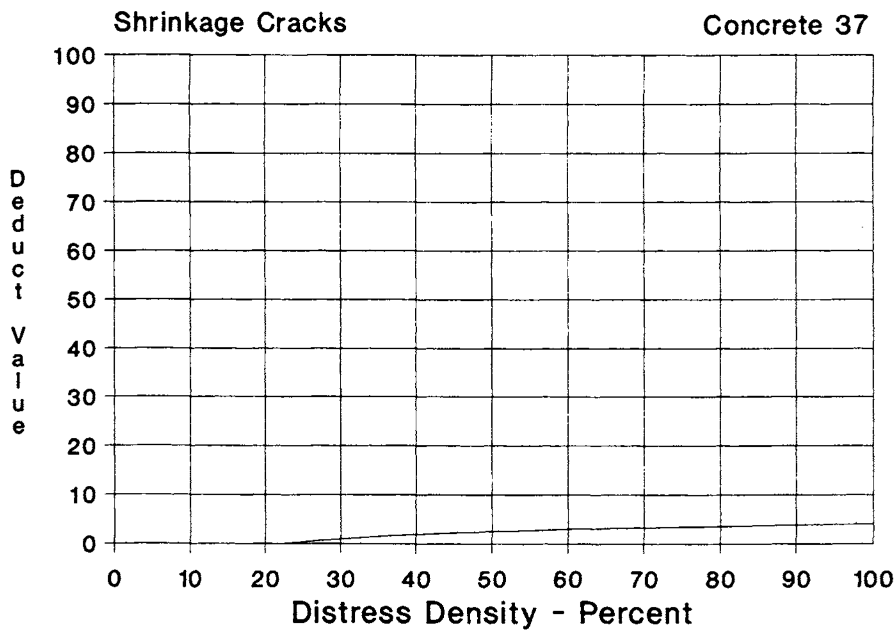


Figure C-36. Shrinkage Cracks.

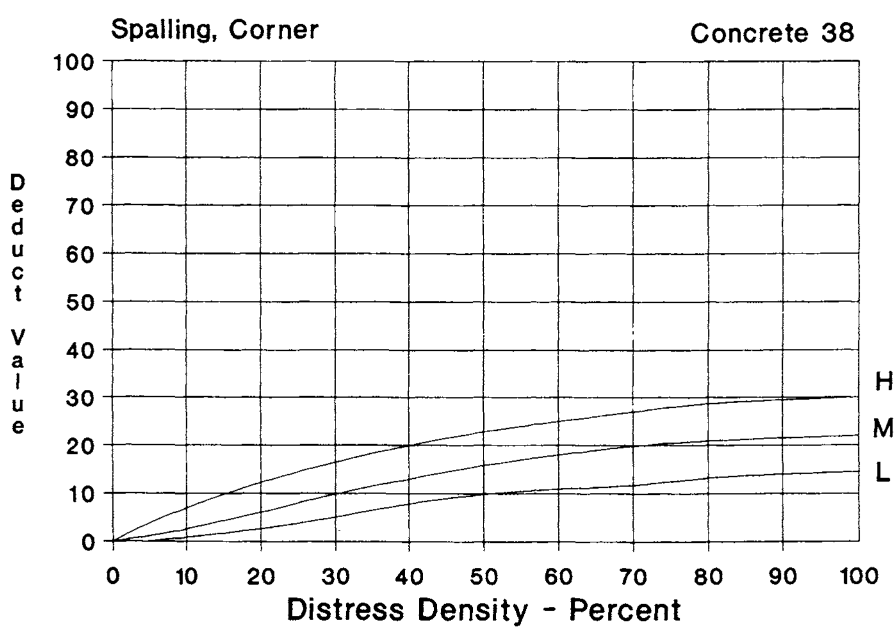


Figure C-37. Spalling, Corner.

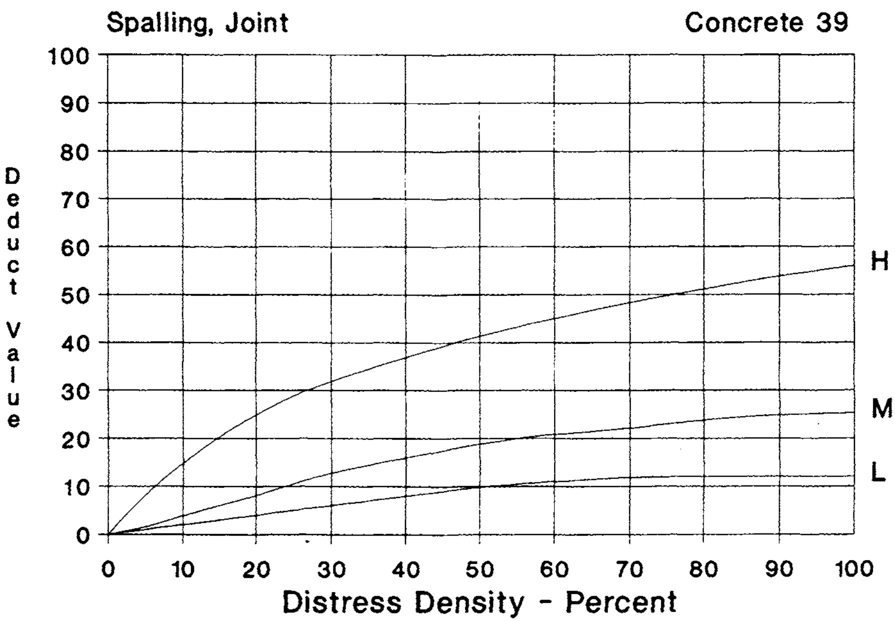


Figure C-38. Spalling, Joint.

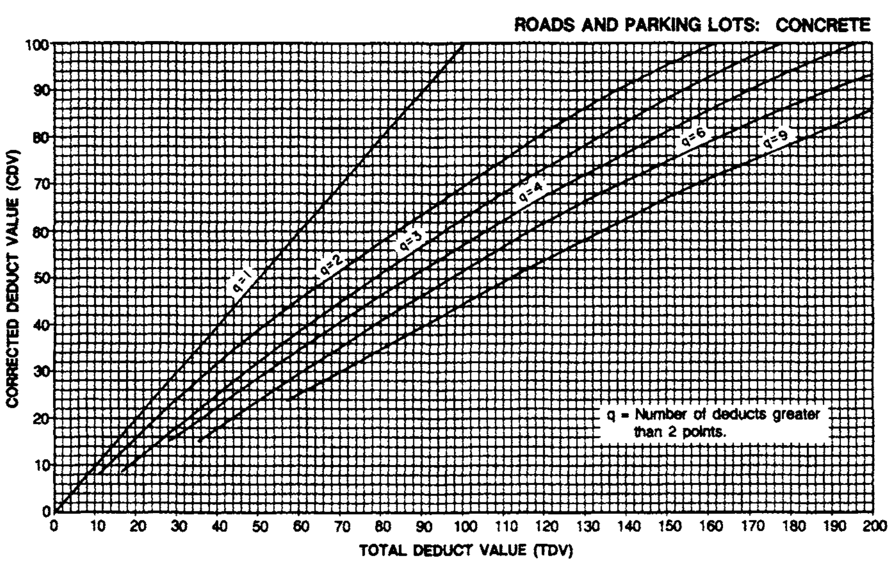


Figure C-39. Corrected Deduct Values for Jointed Concrete Pavement.

APPENDIX D

Asphalt Concrete Airfields: Distress Definitions and Deduct Value Curves

Alligator or Fatigue Cracking (41)

Description

Alligator or fatigue cracking is a series of interconnecting cracks caused by fatigue failure of the asphalt surface under repeated traffic loading. The cracking initiates at the bottom of the asphalt surface (or stabilized base) where tensile stress and strain is highest under a wheel load. The cracks propagate to the surface initially as a series of parallel cracks. After repeated traffic loading, the cracks connect and form many-sided, sharp-angled pieces that develop a pattern resembling chicken wire or the skin of an alligator. The pieces are less than 2 ft. (0.6 m) on the longest side.

Alligator cracking occurs only in areas that are subjected to repeated traffic loadings, such as wheel paths. Therefore, it would not occur over an entire area unless the entire area was subjected to traffic loading. (Pattern-type cracking, which occurs over an entire area that is not subject to loading, is rated as block cracking, which is not a load-associated distress.)

Alligator cracking is considered a major structural distress.

Severity Levels (Figure D–1)

L—Fine, longitudinal hairline cracks running parallel to each other with no or only a few interconnecting cracks. The cracks are not spalled.

M—Further development of light alligator cracking into a pattern or network of cracks that may be lightly spalled.

H—Network or pattern cracking progressed so that pieces are well-defined and spalled at the edges; some of the pieces rock under traffic.

How to Measure

Alligator cracking is measured in square feet (square meters) of surface area. The major difficulty in measuring this type of distress is that many times two or three levels of severity exist within one distressed area. If these portions can be easily distinguished from each other, they should be measured and recorded separately. However, if the different levels of severity cannot be easily divided, the entire area should be rated at the highest severity level present. If alligator cracking and rutting occur in the same area, each is recorded separately at its respective severity level.

Options for Repair

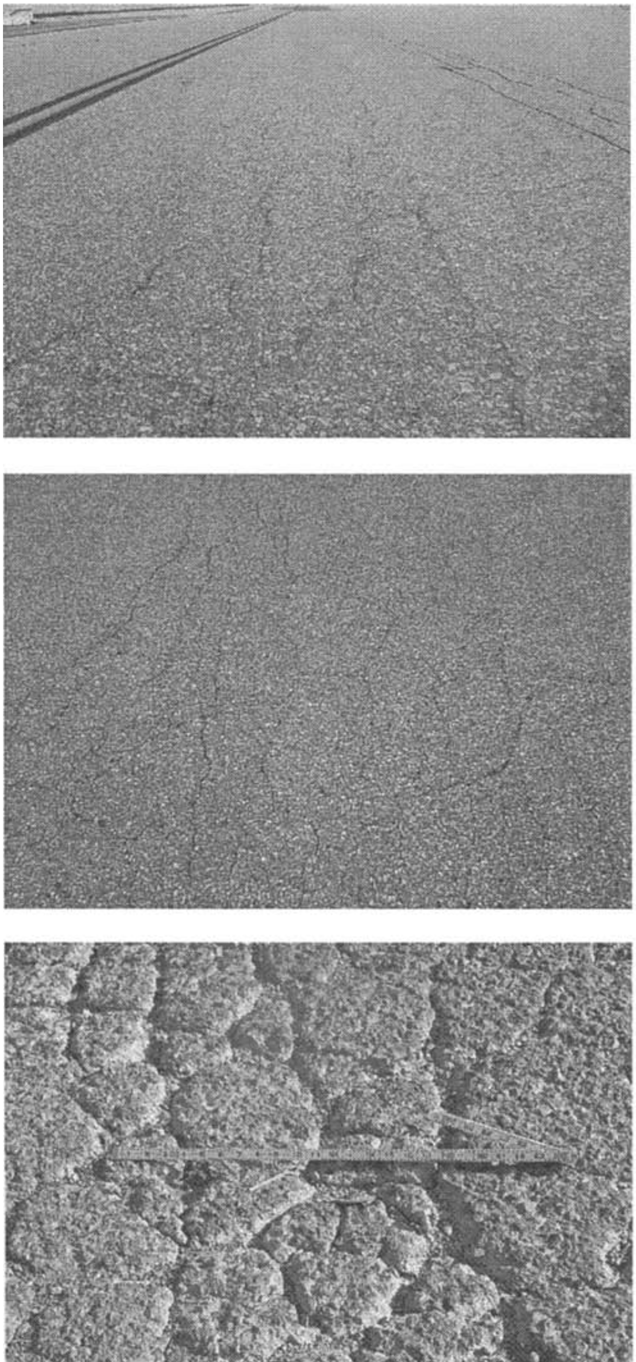
L—Do nothing; Surface seal[a]; Overlay.

M—Partial or full-depth patch; Overlay; Reconstruct.

H—Partial or full-depth patch; Overlay; Reconstruct.

---

[a]Improperly applied rejuvenators or surface seals may cause skid problems on high-speed surfaces.



LOW

MEDIUM

HIGH

Figure D-1. Alligator Cracking.

Bleeding (42)

Description

Bleeding is a film of bituminous material on the pavement surface which creates a shin glass-like, reflecting surface that usually becomes quite sticky. Bleeding is caused t excessive amounts of asphalt cement or tars in the mix and/ or low air-void content. occurs when asphalt fills the voids of the mix during hot weather and then expands on the surface of the pavement. Since the bleeding process is not reversible during co weather, asphalt or tar will accumulate on the surface.

Severity Levels (Figure D–2)

No degrees of severity are defined. Bleeding should be noted when it is extensi enough to cause a reduction in skid resistance.

How to Measure

Bleeding is measured in square feet (square meters) of surface area. If bleeding counted, polished aggregate is not counted in the same area.

Options for Repair

Do nothing; Apply heat, roll sand, and sweep loose material.

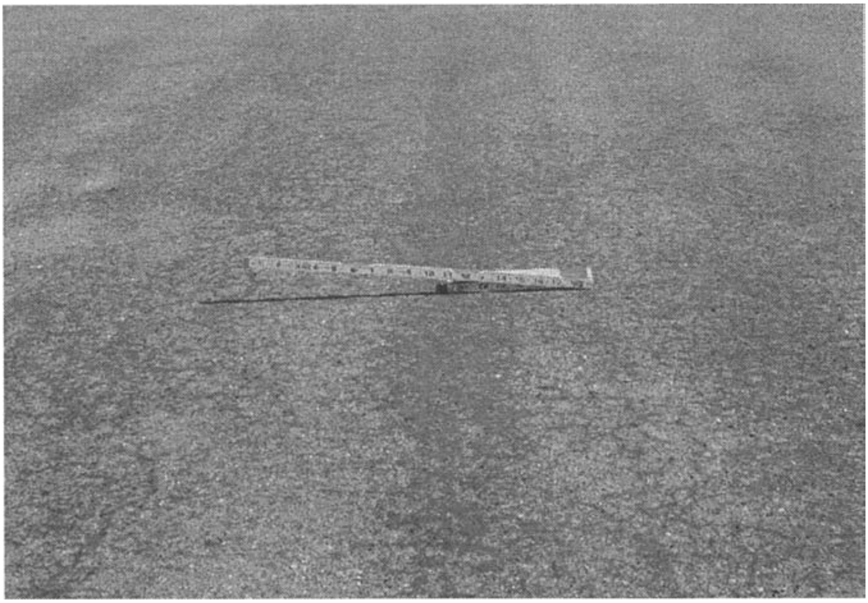


Figure D-2. Bleeding.

Block Cracking (43)

Description

Block cracks are interconnected cracks that divide the pavement into approximately rectangular pieces. The blocks may range in size from approximately 1 by 1 ft (0.3 by 0.3 m) to 10 by 10 ft (3 by 3 m). Block cracking is caused mainly by shrinkage of the asphalt concrete (AC) and daily temperature cycling (which results in daily stress/ strain cycling). It is not load-associated. The occurrence of block cracking usually indicates that the asphalt has hardened significantly. Block cracking normally occurs over a large proportion of pavement area but sometimes will occur in non-traffic areas. This type of distress differs from alligator cracking in that alligator cracks form smaller, many-sided pieces with sharp angles. Also, unlike block cracks, alligator cracks are caused by repeated traffic loadings and, therefore, are located only in traffic areas (i.e., wheel paths).

Severity Levels (Figure D–3)

L—Blocks are defined by cracks that are non-spalled (sides of the crack are vertical) or only lightly spalled, causing no FOD potential. Non-filled cracks have 1/4 in. (6.4 mm) or less mean width, and filled cracks have filler in satisfactory condition.

M—Blocks are defined by either:

1. Filled or non-filled cracks that are moderately spalled (some FOD potential)
2. Non-filled cracks that are not spalled or have only minor spalling (some FOD potential), but have a mean width greater than approximately 1/4 in. (6.4 mm).
3. Filled cracks that are not spalled or have only minor spalling (some FOD potential), but have filler in unsatisfactory condition.

H—Blocks are well-defined by cracks that are severely spalled, causing a definite FOD potential.

How to Measure

Block cracking is measured in square feet (square meters) of surface area. It usually occurs at one severity level in a given pavement section; however, any areas of the pavement section having distinctly different levels of severity should be measured and recorded separately. For asphalt pavements, not including AC over PCC, if block cracking is recorded, no longitudinal and transverse cracking should be recorded in the same area. For asphalt overlay over concrete, block cracking, joint reflection cracking, and longitudinal and transverse cracking reflected from old concrete should all be recorded separately.

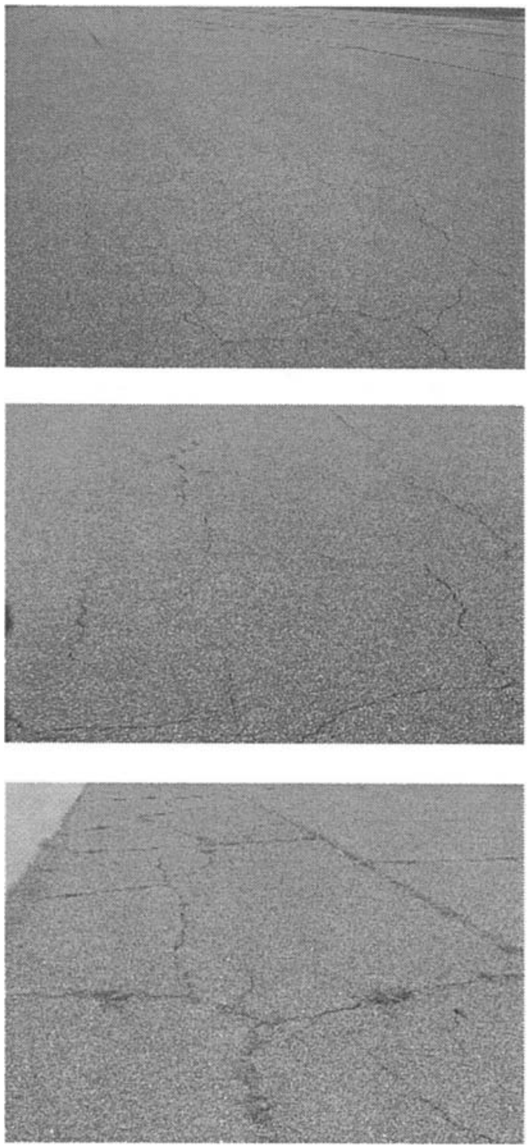


Figure D-3. Block Cracking.

Options for Repair

L—Do nothing; Apply rejuvenator[a].

M—Seal cracks; Apply rejuvenator[a]; Recycle surface; Heater scarify and overlay.

H—Seal cracks; Recycle surface; Heater scarify and overlay.

---

[a]Improperly applied rejuvenators or surface seals may cause skid problems on high-speed surfaces.

Corrugation (44)

Description

Corrugation is a series of closely spaced ridges and valleys (ripples) occurring at fairly regular intervals (usually less than 10 ft) (3 m) along the pavement. The ridges are perpendicular to the traffic direction. Traffic action combined with an unstable pavement surface or base usually causes this type of distress.

Severity Levels (Figure D–4)

L—Corrugations are minor and do not significantly affect ride quality (see measurement criteria below).

M—Corrugations are noticeable and significantly affect ride quality (see measurement criteria below).

H—Corrugations are easily noticed and severely affect ride quality (see measurement criteria below).

How to Measure

Corrugation is measured in square feet (square meters) of surface area. The mean elevation difference between the ridges and valleys of the corrugations indicates the level of severity. To determine the mean elevation difference, a 10-foot straightedge should be placed perpendicular to the corrugations so that the depth of the valleys can be measured in inches (mm). The mean depth is calculated from five such measurements.

Severity	Runways and High-Speed Taxiways	Taxiways and Aprons
L	<1/4 in. (<6.4 mm)	<1/2 in. (<12.7 mm)
M	1/4 to 1/2 in (6.4 to 12.7 mm)	1/2 to 1 in. (12.7 to 25.4 mm)
H	>1/2 in. (>12.7 mm)	>1 in. (>25.4 mm)

Options for Repair

L—Do nothing.

M—Reconstruct.

H—Reconstruct.

LOW

MEDIUM

HIGH

Figure D-4. Corrugation.

Depression (45)

Description

Depressions are localized pavement surface areas having elevations slightly lower than those of the surrounding pavement. In many instances, light depressions are not noticeable until after a rain, when ponding water creates "birdbath" areas; but the depressions can also be located without rain because of stains created by ponding water. Depressions can be caused by settlement of the foundation soil or can be "built up" during construction. Depressions cause roughness and, when filled with water of sufficient depth, can cause hydroplaning of aircraft.

Severity Levels (Figure D–5)

L—Depression can be observed or located by stained areas, only slightly affects pavement riding quality, and may cause hydroplaning potential on runways (see measurement criteria below).

M—The depression can be observed, moderately affects pavement riding quality, and causes hydroplaning potential on runways (see measurement criteria below).

H—The depression can be readily observed, severely affects pavement riding quality, and causes definite hydroplaning potential (see measurement criteria below).

How to Measure

Depressions are measured in square feet (square meters) of surface area. The maximum depth of the depression determines the level of severity. This depth can be measured by placing a 10-ft (3 m) straightedge across the depressed area and measuring the maximum depth in inches (mm). Depressions larger than 10 feet (3 m) across must be measured by either visual estimation or direct measurement when filled with water.

Options for Repair

	Maximum Depth of Depression	
Severity	Runways and High Speed Taxiways	Taxiways and Aprons
L	1/8 to 1/2 in. (3.2 to 12.7 mm)	1/2 to 1 in. (12.7 to 25.4 mm)
M	1/2 to 1 in. (12.7 to 25.4 mm)	>1 to 2 in. (25.4 to 50.8 mm)
H	>1 in. (>25.4 mm)	>2 in. (>50.8 mm)

L—Do nothing.

M—Shallow[a], partial or full-depth patch.

H—Shallow[a], partial or full-depth patch.

[a]Shallow patching should not be used on runways where FOD is of concern.

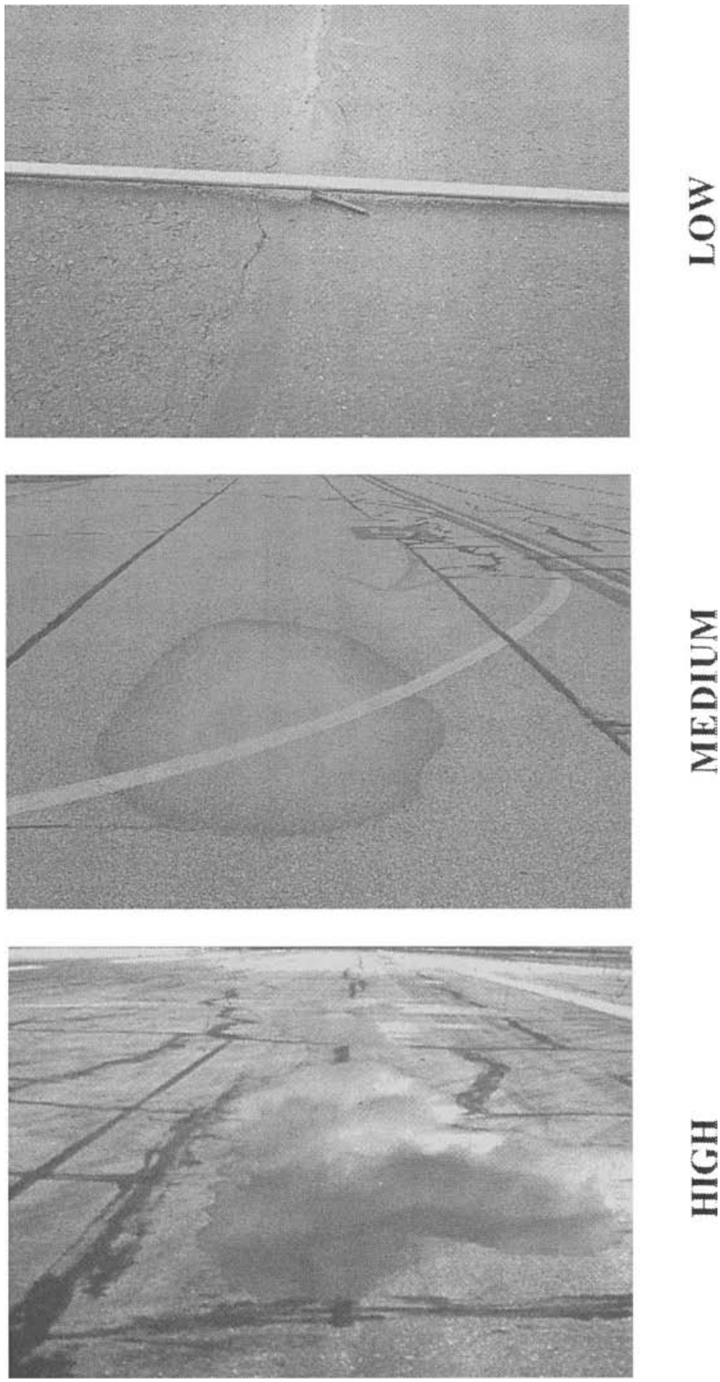


Figure D-5. Depression.

Jet Blast Erosion (46)

Description

Jet blast erosion causes darkened areas on the pavement surface when bituminous binder has been burned or carbonized; localized burned areas may vary in depth up to approximately 1/2 in.(12.7 mm).

Severity Levels (Figure D–6)

No degrees of severity are defined. It is sufficient to indicate that jet blast erosion exists.

How to Measure

Jet blast erosion is measured in square feet (square meters) of surface area.

Options for Repair

Do nothing; Partial-depth patch; Apply rejuvenator[a].

---

[a]Improperly applied rejuvenators or surface seals may cause skid problems on high-speed surfaces.

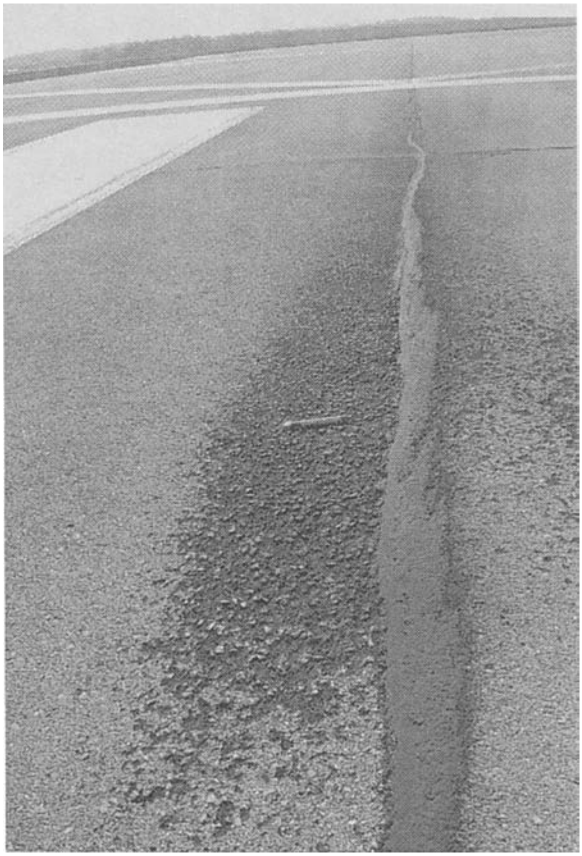


Figure D-6. Jet Blast Erosion.

Joint Reflection Cracking from PCC (47) (Longitudinal and Transverse)

Description

This distress occurs only on pavements having an asphalt or tar surface over a Portland cement concrete (PCC) slab. This category does not include reflection cracking from any other type of base (i.e., cement stabilized, lime stabilized); such cracks are listed as longitudinal and transverse cracks. Joint-reflection cracking is caused mainly by movement of the PCC slab beneath the asphalt concrete (AC) surface because of thermal and moisture changes; it is not load related. However, traffic loading may cause a breakdown of the AC near the crack, resulting in spalling and FOD potential. If the pavement is fragmented along a crack, the crack is said to be spalled. A knowledge of slab dimensions beneath the AC surface will help to identify these cracks.

Severity Levels (Figure D–7)

L—Cracks have only light spalling (little or no FOD potential) or no spalling and can be filled or non-filled. If non-filled, the cracks have a mean width of 1/4 in. (6.4 mm) or less. Filled cracks are of any width, but their filler material is in satisfactory condition.

M—One of the following conditions exists:

1. Cracks are moderately spalled (some FOD potential) and can be either filled or non-filled of any width.
2. Filled cracks are not spalled or are only lightly spalled, but the filler is in unsatisfactory condition.
3. Non-filled cracks are not spalled or are only lightly spalled, but the mean crack width is greater than 1/4 in. (6.4 mm).
4. Light random cracking exists near cracks or at the corner of intersecting cracks.

H—Cracks are severely spalled (definite FOD potential) and can be either filled or non-filled of any width.

How to Measure

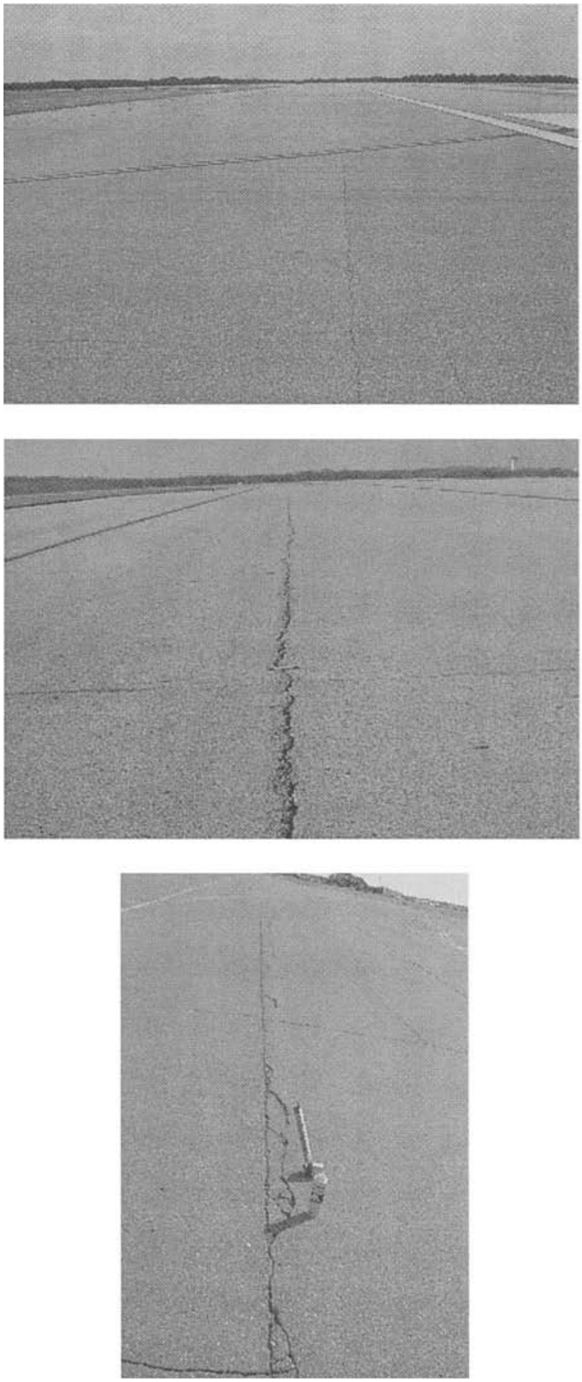
Joint-reflection cracking is measured in linear feet (linear meters). The length and severity level of each crack should be identified and recorded. If the crack does not have the same severity level along its entire length, each portion should be recorded separately. For example, a crack that is 50 ft (15 m) long may have 10 ft (3 m) of high severity, 20 ft (6 m) of medium severity, and 20 ft (6 m) of light severity; all would be recorded separately.

Options for Repair

L—Do nothing; Seal cracks greater than 1/8 in. (3.2 mm).

M—Seal cracks; Partial-depth patch.

H—Seal cracks; Partial-depth patch; Reconstruct joint.



LOW

MEDIUM

HIGH

Figure D-7. Reflection Cracking.

Longitudinal and Transverse Cracking (48) (Non-PCC Joint Reflective)

Description

Longitudinal cracks are parallel to the pavement's centerline or laydown direction. They may be caused by (1) a poorly constructed paving lane joint, (2) shrinkage of the AC surface due to low temperatures or hardening of the asphalt, or (3) a reflective crack caused by cracks beneath the surface course, including cracks in PCC slabs (but not at PCC joints). Transverse cracks extend across the pavement at approximately right angles to the pavement centerline or direction of laydown. They may be caused by items 2 or 3 above. These types of cracks are not usually load-associated. If the pavement is fragmented along a crack, the crack is said to be spalled.

Severity Levels (Figure D–8a)

L—Cracks have either minor spalling (little or no FOD potential) or no spalling. The cracks can be filled or non-filled. Non-filled cracks have a mean width of 1/4 in. (6.4 mm) or less; filled cracks are of any width, but their filler material is in satisfactory condition.

M—One of the following conditions exists:

1. Cracks are moderately spalled (some FOD potential) and can be either filled or non-filled of any width.
2. Filled cracks are not spalled or are only lightly spalled, but the filler is in unsatisfactory condition.
3. Non-filled cracks are not spalled or are only lightly spalled, but mean crack width is greater than 1/4 in. (6.4 mm).
4. Lightly random cracking exists near the crack or at the corners of intersecting cracks.

H—Cracks are severely spalled, causing definite FOD potential. They can be either filled or non-filled of any width.

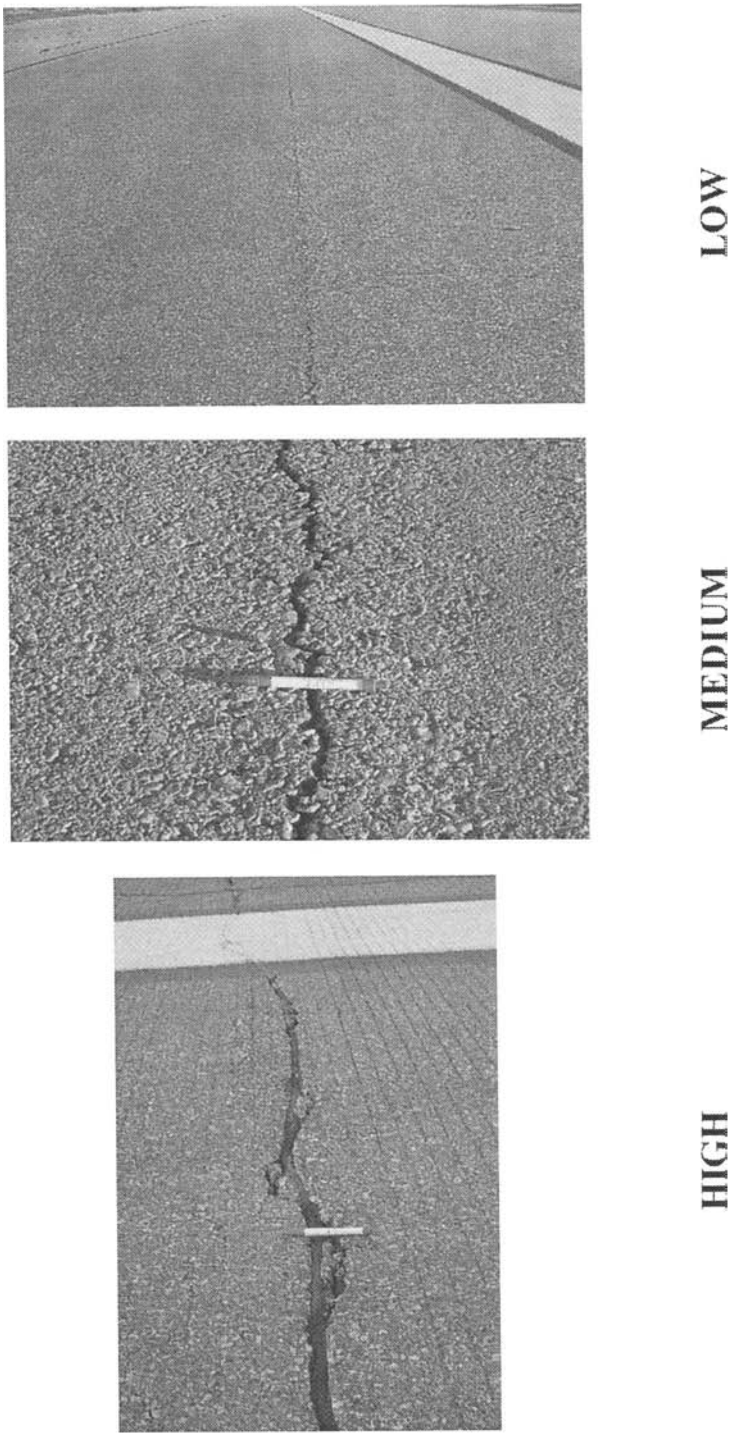


Figure D-8a. Longitudinal and Transverse Cracking.

Porous Friction Course Severity Levels (Figure D–8b)

Note: these severity levels are in addition to the existing definitions.

L—Average raveled area around the crack is less than 1/4 in. (6.4 mm) wide.

M—Average raveled area around the crack is 1/4 to 1 in. (6.4 to 25.4 mm) wide.

H—Average raveled area around the crack is greater than 1 in. (25.4 mm) wide.

How to Measure

Longitudinal and transverse cracks are measured in linear feet (linear meters). The length and severity of each crack should be identified and recorded. If the crack does not have the same severity level along its entire length, each portion of the crack having a different severity level should be recorded separately. For an example, see Joint-Reflection Cracking.

Options for Repair

L—Do nothing; Seal cracks greater than 1/8 in. (3.2 mm); Apply rejuvenator[a]; Surface seal[a].

M—Seal cracks.

H—Seal cracks; Partial depth patch.

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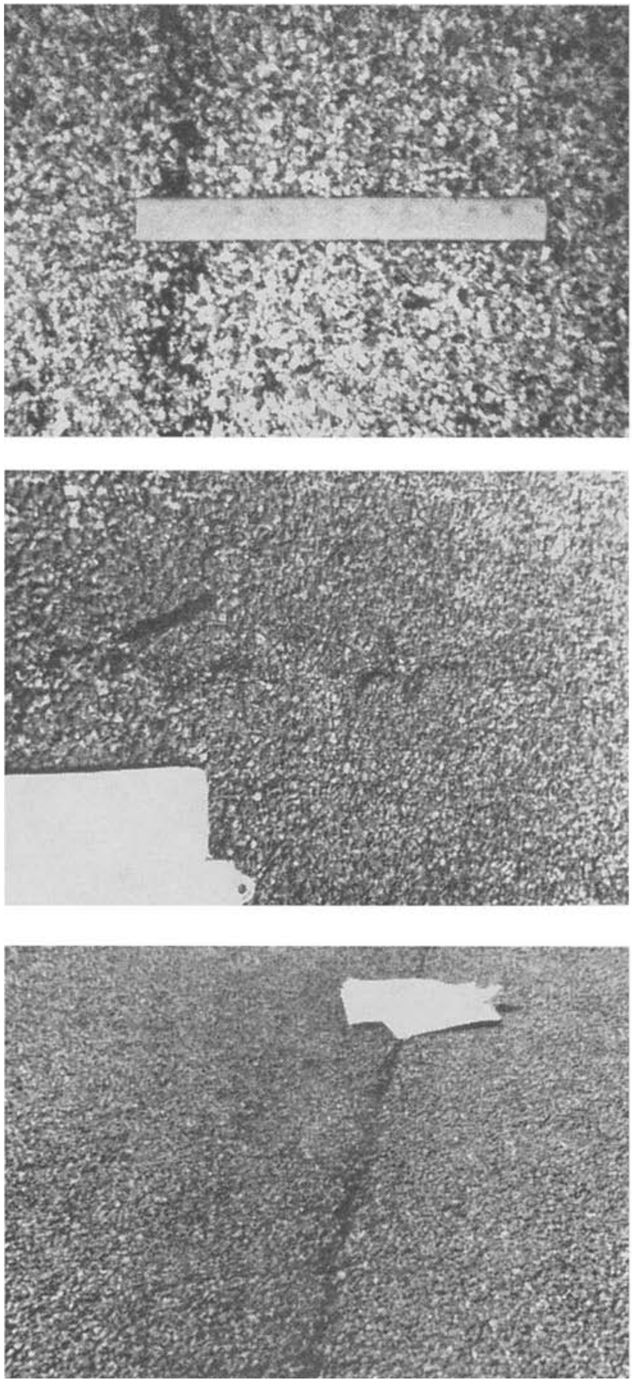
[a]Improperly applied rejuvenators or surface seals may cause skid problems on high-speed surfaces.



LOW

MEDIUM

HIGH

Figure D-8b. Crack in Porous Friction Course.

Oil Spillage (49)

Description

Oil spillage is the deterioration or softening of the pavement surface caused by the spilling of oil, fuel, or other solvents

Severity Levels (Figure D–9)

No degrees of severity are defined. It is sufficient to indicate that oil spillage exists.

How to Measure

Oil spillage is measured in square feet (square meters) of surface area.

Options for Repair

Do nothing; Partial or full-depth patch.

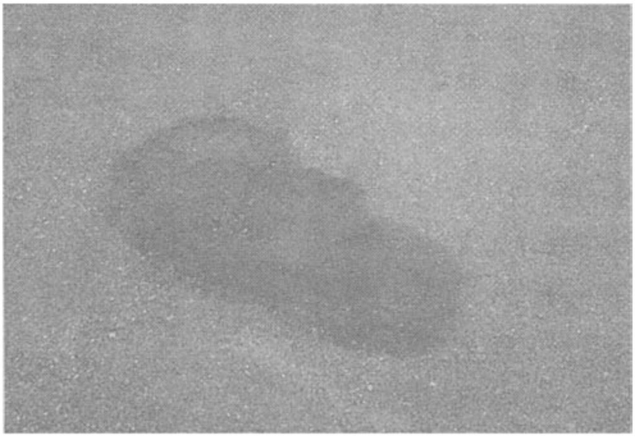


Figure D-9. Oil Spillage.

Patching and Utility Cut Patch (50)

Description

A patch is considered a defect, regardless of how well it is performing.

Severity Levels (Figure D–10)

L—Patch is in good condition and is performing satisfactorily.

M—Patch is somewhat deteriorated and affects riding quality to some extent.

H—Patch is badly deteriorated and affects riding quality significantly or has high FOD potential. Patch needs replacement.

Porous Friction Courses

The use of dense-graded AC patches in porous friction surfaces causes a water damming effect at the patch that contributes to differential skid resistance of the surface. Low-severity, dense-graded patches should be rated as medium severity because of the differential friction problem. Medium and high-severity patches are rated the same as above.

How to Measure

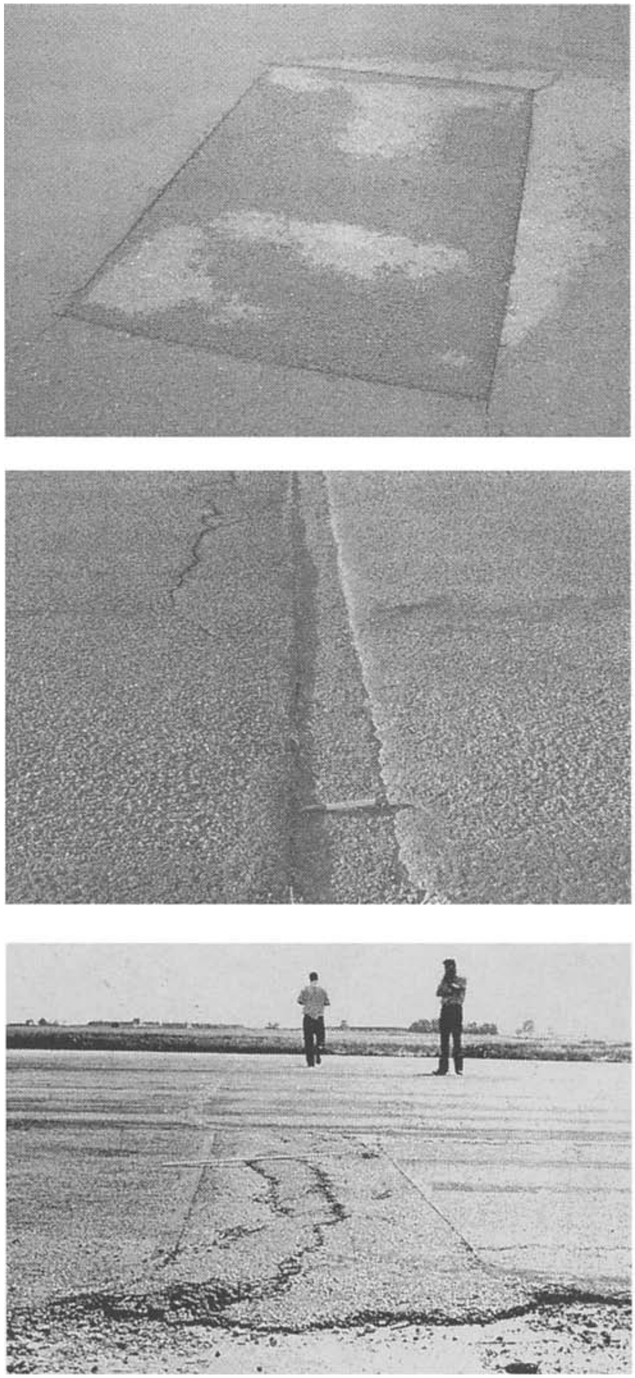
Patching is measured in square feet (square meters) of surface area. However, if a single patch has areas of differing severity levels, these areas should be measured and recorded separately. For example, a 25 square foot (7.4 square meter) patch may have 10 square feet (3.5 square meters) of medium severity and 15 square feet (4.5 square meters) of light severity. These areas would be recorded separately. Any distress found in a patched area will not be recorded; however, its effects on the patch will be considered when determining the patch's severity level.

Options for Repair

L—Do nothing.

M—Seal cracks; Repair distress in patch; Replace patch.

H—Replace patch.



LOW

MEDIUM

HIGH

Figure D-10. Patching and Utility Cut Patch.

Polished Aggregate (51)

Description

Aggregate polishing is caused by repeated traffic applications. Polished aggregate is present when close examination of a pavement reveals that the portion of aggregate extending above the asphalt is either very small or there are no rough or angular aggregate particles to provide good skid resistance. Existence of this type of distress is also indicated when the number on a skid resistance rating test is low or has dropped significantly from previous ratings.

Severity Levels (Figure D–11)

No degrees of severity are defined. However, the degree of polishing should be significant before it is included in the condition survey and rated as a defect.

How to Measure

Polished aggregate is measured in square feet (square meters) of surface area. If bleeding is counted, polished aggregate is not counted in the same area.

Options for Repair

Do nothing; Overlay; Surface friction course.

Figure D-11. Polished Aggregate.

Raveling and Weathering (52)

Description

Raveling and weathering are the wearing away of the pavement surface caused by the dislodging of aggregate particles and loss of asphalt or tar binder. They may indicate that the asphalt binder has hardened significantly.

Dense Mix Severity Levels. (Figure D–12a)

As used herein, coarse aggregate refers to aggregate with a smallest dimension greater than or equal to 3/8 inch (10mm). If in doubt, three representative square yards should be inspected and the number of missing pieces of aggregate counted.

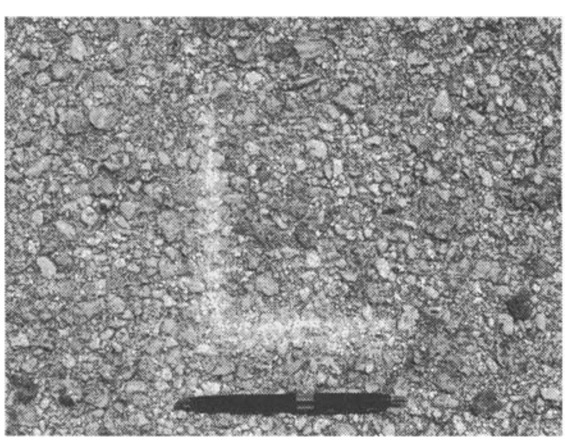
L—(1) The surface is in generally good condition, but fine aggregate and binder have worn away exposing the coarse aggregate. The coarse aggregate, however, is still firmly embedded in the mix. (2) In a square yard representative sample, the number of coarse aggregate pieces missing is between 5 and 20. (3) In a square yard representative sample, brushing one's foot across the surface does not dislodge more than 20 coarse aggregate pieces.

M—(1) In a square yard representative sample, the number of coarse aggregate pieces missing is between 21 and 40. (2) In a square yard representative sample, brushing one's foot across the surface dislodges between 21 and 40 coarse aggregate pieces.

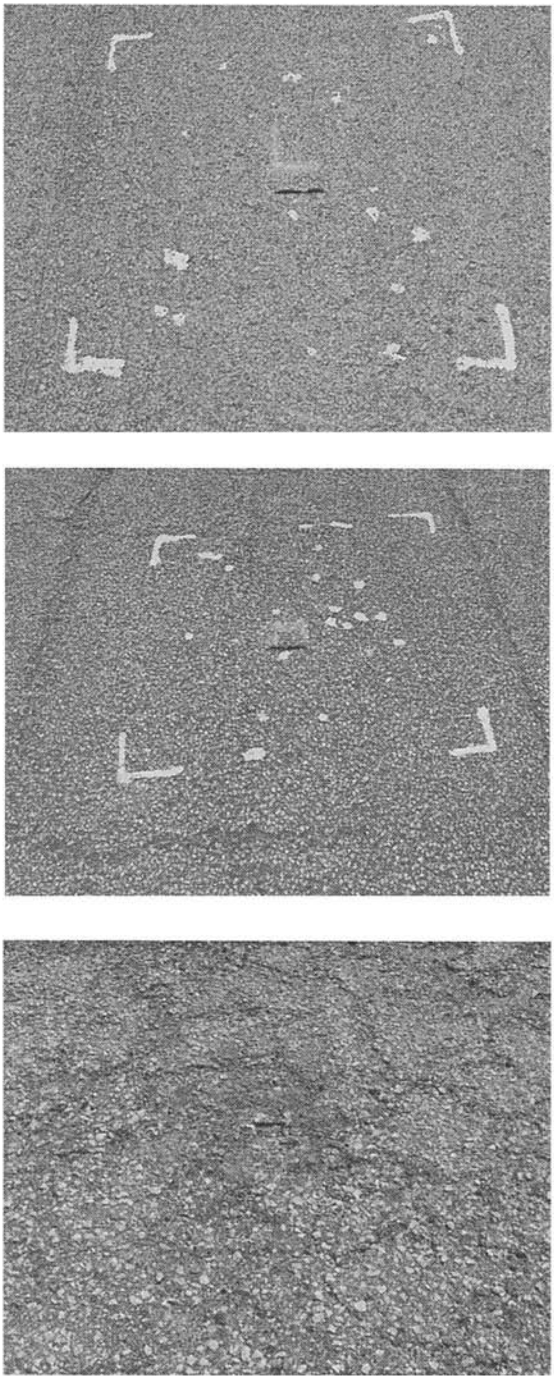
H—(1) In a square yard representative sample, the number of coarse aggregate pieces missing is over 40. (2) In a square yard representative sample, brushing one's foot across the surface dislodges more than 40 coarse aggregate pieces.

How to Measure

Raveling and weathering are measured in square meter (square feet) or surface area. Mechanical damage caused by hook drags, tire rims, or snowplows is counted as areas of high-severity raveling and weathering.



LOW (1)



LOW (2)

MEDIUM

HIGH

Figure D-12a. Raveling and Weathering.

Raveling and Weathering (52) Continued

Surface Treatment /Tar over Dense Mix Severity Levels (Figure D–12b)

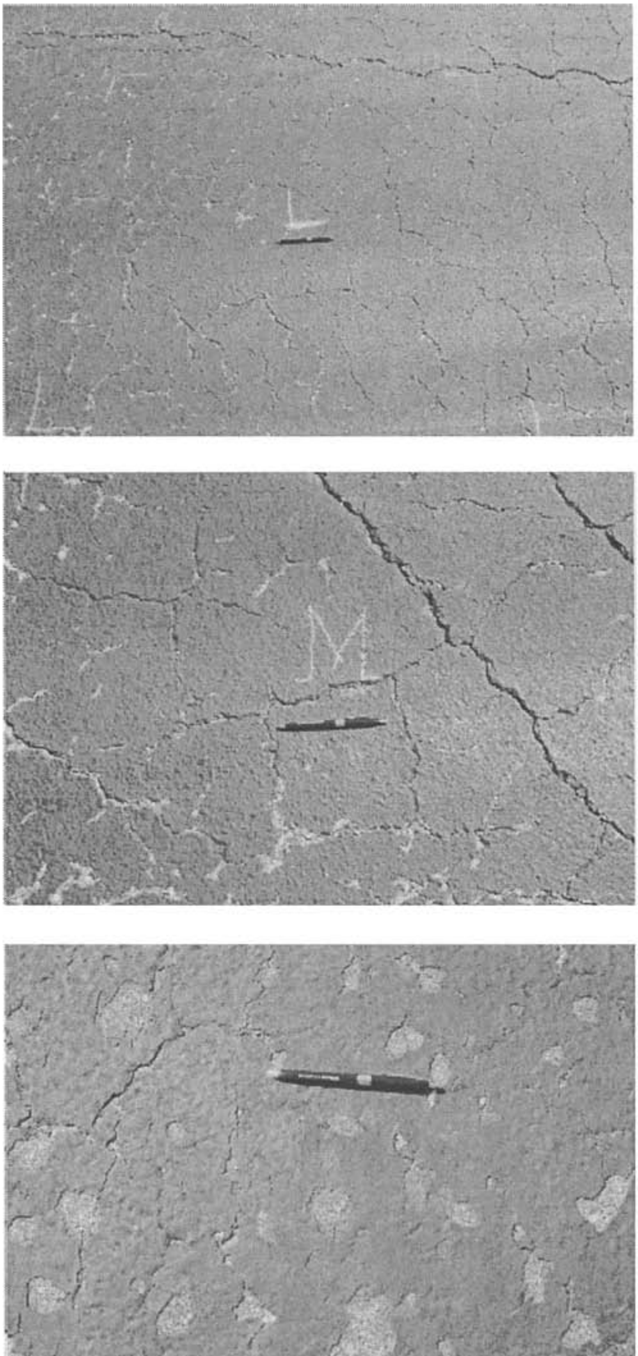
L—(1) Scaled area is less than 1%.(2) In case of coal tar where pattern cracking has developed, the tar surface cracks are less than ¼ inch ((6 mm) wide.

M—(1) Scaled area is between 1 and 1%.(2) In case of coal tar where pattern cracking has developed, the cracks are ¼ inch ((6 mm) wide or greater.

H—(1) Scaled area is over 10%.(2) In case of coal tar the surface is peeling off.

How to Measure

Raveling and weathering are measured in square meter (square feet) or surface area. Mechanical damage caused by hook drags, tire rims, or snowplows is counted as areas of high-severity raveling and weathering.



LOW

MEDIUM

HIGH

Figure D-12b. Raveling/Weathering on a Porous Friction Course Surface.

Raveling and Weathering (52) Continued

Porous Friction Course Severity Levels (Fig. D–12c)

L—In a square meter (square yard) representative sample, the number of aggregate pieces missing is between 5 and 20 and/or the number of missing aggregate clusters (when more than one adjoining aggregate piece is missing) does not exceed 1.

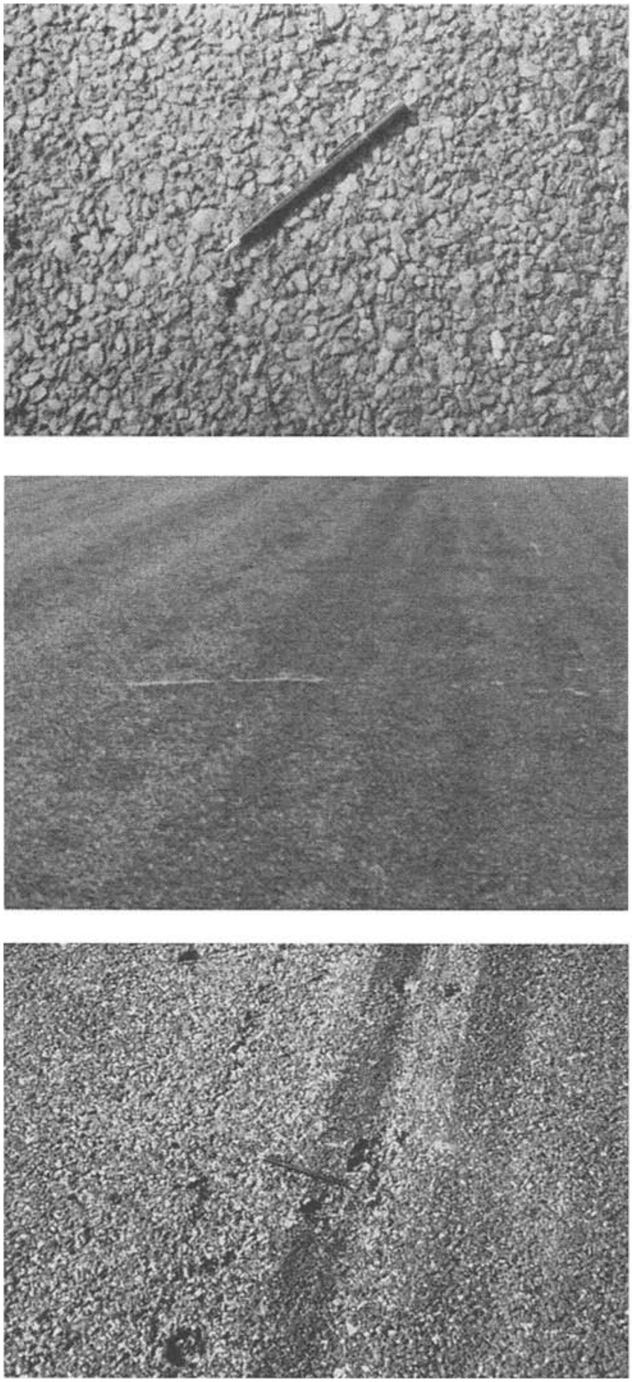
M—In a square meter (square yard) representative sample, the number of aggregate pieces missing is between 21 and 40 and/or the number of missing aggregate clusters is greater than 1 but does not exceed 25 percent of the square meter (square foot) area.

H—In a square meter (square yard) representative sample, the number of aggregate pieces missing is over 40 and/or the number of missing aggregate clusters is greater than 25 percent of the square meter (square foot) area.

How to Measure

Raveling and weathering are measured in square meter (square feet) or surface area. Mechanical damage caused by hook drags, tire rims, or snowplows is counted as areas of high-severity raveling and weathering.

[a]Improperly applied rejuvenators or surface seals may cause skid problems on high-speed surfaces.



LOW

MEDIUM

HIGH

Figure D-12c. Raveling/Weathering on a Porous Friction Course Surface.

Rutting (53)

Description

A rut is a surface depression in the wheel path. Pavement uplift may occur along the sides of the rut; however, in many instances ruts are noticeable only after a rainfall, when the wheel paths are filled with water. Rutting stems from a permanent deformation in any of the pavement layers or subgrade. It is usually caused by consolidation or lateral movement of the materials due to traffic loads. Significant rutting can lead to major structural failure of the pavement.

Severity Levels (Figure D–13)

Mean Rut Depth Criteria
Severity—All Pavement Sections

L—Less than 1/4 to 1/2 in. (<6.4 to 12.7 mm).

M—From 1/2 to 1 in. (12.7 to 25.4 mm).

H—Greater than 1 in. (25.4 mm).

How to Measure

Rutting is measured in square feet (square meters) of surface area, and its severity is determined by the depth of the rut. To determine the rut depth, a straightedge should be laid across the rut and the maximum depth measured. The mean depth in inches (mm) should be computed from measurements taken along the length of the rut.

Options for Repair

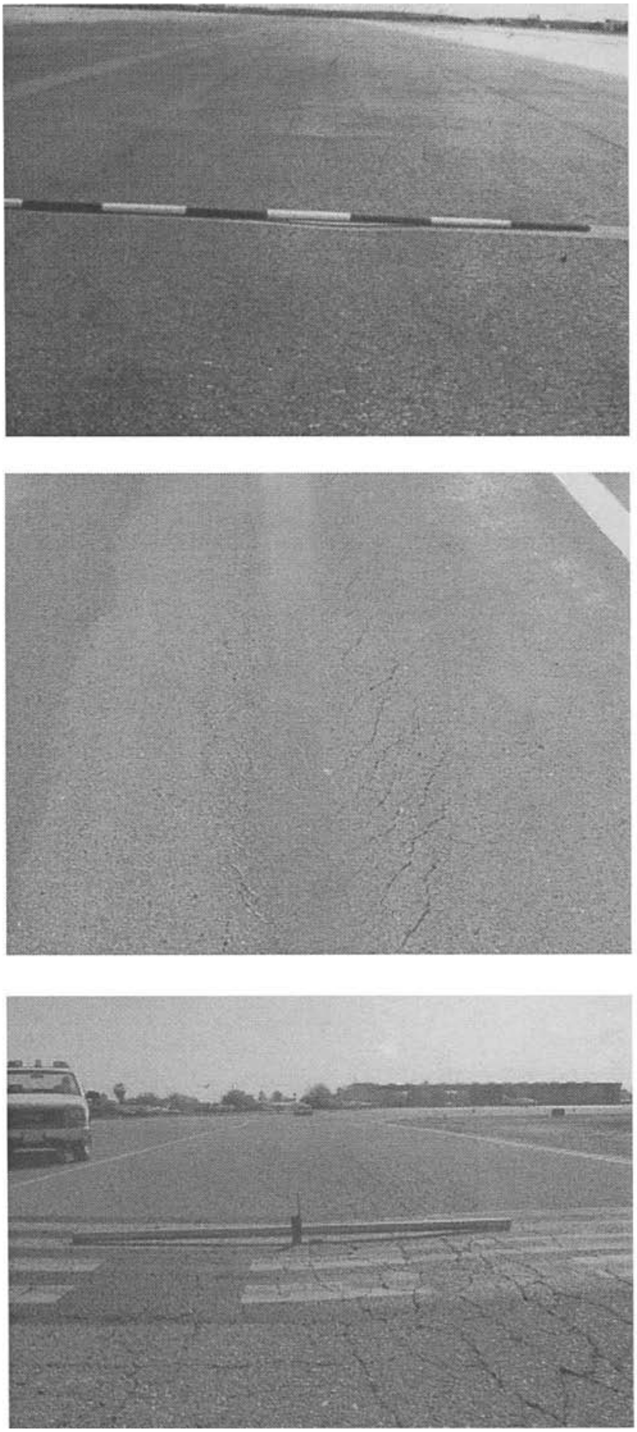
L—Do nothing.

M—Shallow[a], partial or full-depth patch; Partial or full-depth patch and overlay.

H—Shallow[a], partial or full-depth patch; Partial or full-depth patch and overlay.

---

[a]Shallow patching should not be used on runways where FOD is of concern.



LOW

MEDIUM

HIGH

Figure D-13. Rutting.

Shoving of Asphalt Pavement by PCC Slabs (54)

Description

PCC pavements occasionally increase in length at ends where they adjoin flexible pavements (commonly referred to as "pavement growth"). This "growth" shoves the asphalt or tar-surfaced pavements, causing them to swell and crack. The PCC slab "growth" is caused by a gradual opening of the joints as they are filled with incompressible materials that prevent them from reclosing.

Severity Levels (Figure D–14)

L—A slight amount of shoving has occurred, with little effect on ride quality and no breakup of the asphalt pavement.

M—A significant amount of shoving has occurred, causing moderate roughness or breakup of the asphalt pavement.

H—A large amount of shoving has occurred, causing severe roughness or breakup of the asphalt pavement.

How to Measure

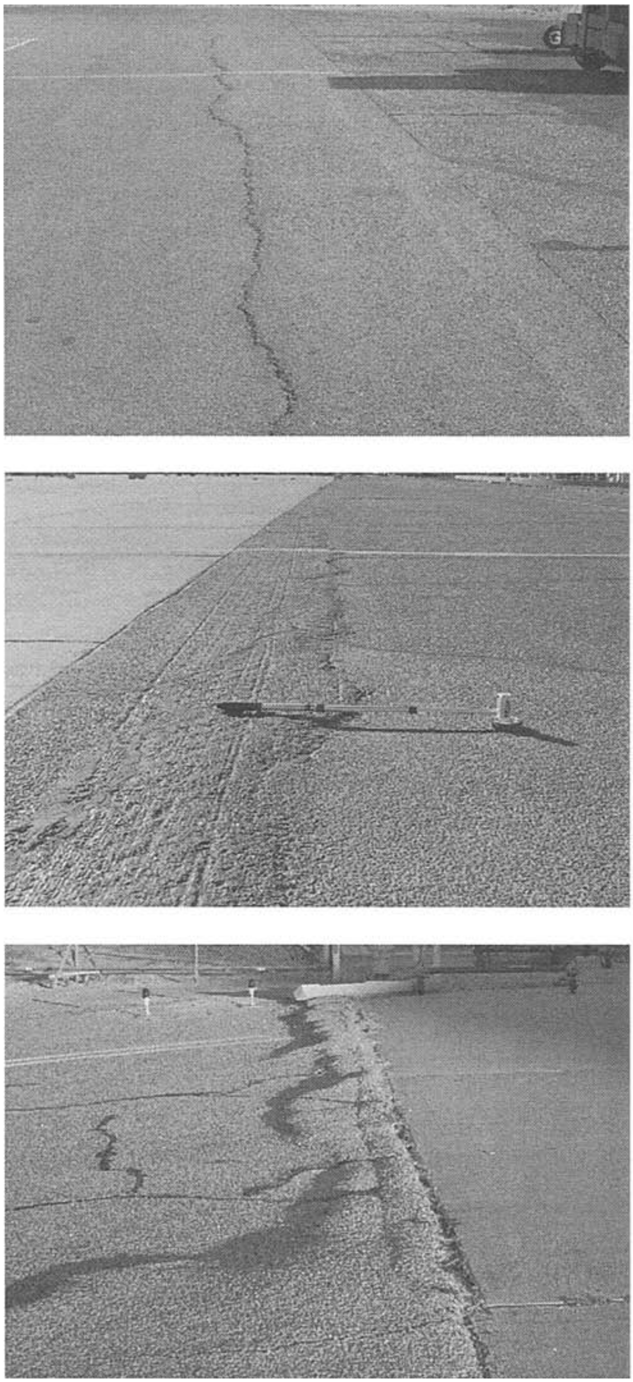
Shoving is measured by determining the area in square feet (square meters) of the swell caused by shoving.

Options for Repair

L—Do nothing.

M—Partial-depth patch; Full-depth patch.

H—Partial-depth patch; Full-depth patch.



LOW

MEDIUM

HIGH

Figure D-14. Shoving.

Slippage Cracking (55)

Description

Slippage cracks are crescent or half-moon shaped cracks having two ends pointed away from the direction of traffic. They are produced when braking or turning wheels cause the pavement surface to slide and deform. This usually occurs when there is a low-strength surface mix or poor bond between the surface and next layer of pavement structure.

Severity Levels (Figure D–15)

No degrees of severity are defined. It is sufficient to indicate that a slippage crack exists.

How to Measure

Slippage cracking is measured in square feet (square meters) of surface area.

Options for Repair

Do nothing; Partial or full-depth patch.

Figure D-15. Slippage Cracking.

Swell (56)

Description

A swell is characterized by an upward bulge in the pavement's surface. A swell may occur sharply over a small area or as a longer, gradual wave. Either type of swell can be accompanied by surface cracking. A swell is usually caused by frost action in the subgrade or by swelling soil, but a small swell can also occur on the surface of an asphalt overlay (over PCC) as a result of a blow- up in the PCC slab.

Severity Levels (Figure D–16)

L—Swell is barely visible and has a minor effect on the pavement's ride quality as determined at the normal aircraft speed for the pavement section under consideration. (Low-severity swells may not always be observable, but their existence can be confirmed by driving a vehicle over the section at the normal aircraft speed. An upward acceleration will occur if the swell is present).

M—Swell can be observed without difficulty and has a significant effect on the pavement's ride quality as determined at the normal aircraft speed for the pavement section under consideration.

H—Swell can be readily observed and severely affects the pavement's ride quality at the normal aircraft speed for the pavement section under consideration.

How to Measure

The surface area of the swell is measured in square feet (square meters). The severity rating should consider the type of pavement section (i.e., runway, taxiway, or apron). For example, a swell of sufficient magnitude to cause considerable roughness on a runway at high speeds would be rated as more severe than the same swell located on the apron or taxiway where the normal aircraft operating speeds are much lower. The following guidance is provided for runways:

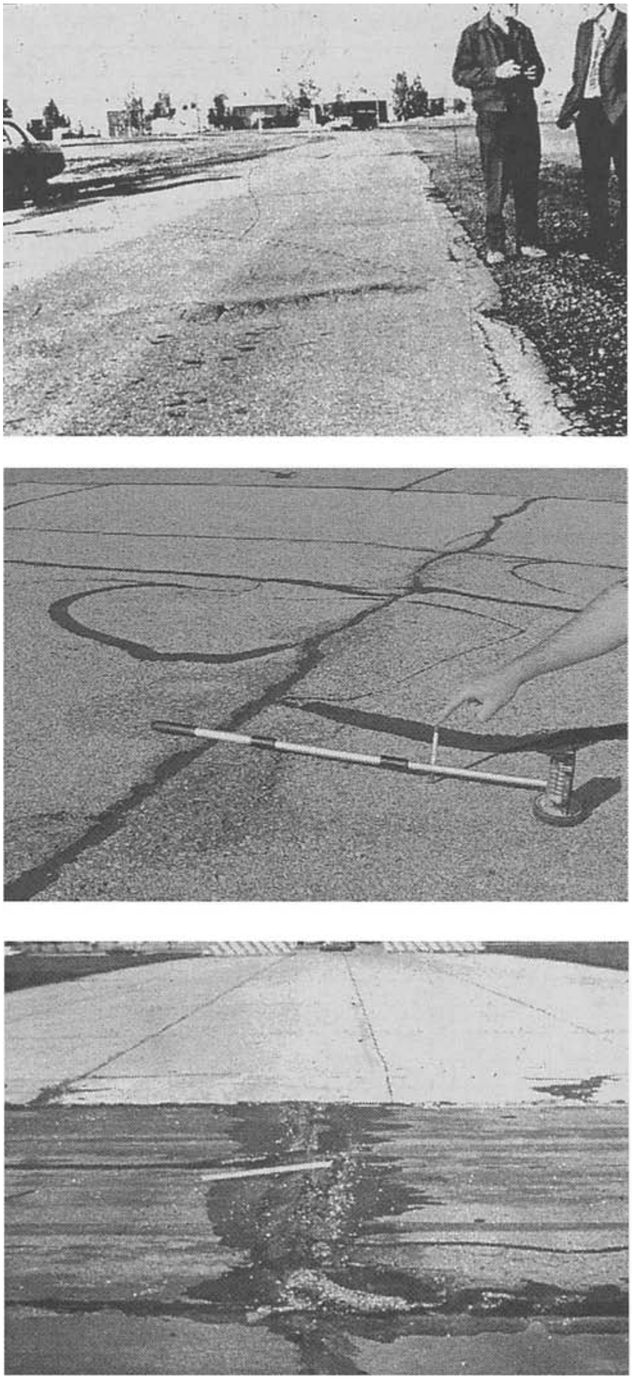
Severity	Height Differential
L	<3/4 in. (<19 mm)
M	3/4 to 1 1/2 in. (19 to 38 mm)
H	>1 1/2 in. (>38 mm)

Options for Repair

L—Do nothing.

M—Reconstruct.

H—Reconstruct.



LOW

MEDIUM

HIGH

Figure D-16. Swell.

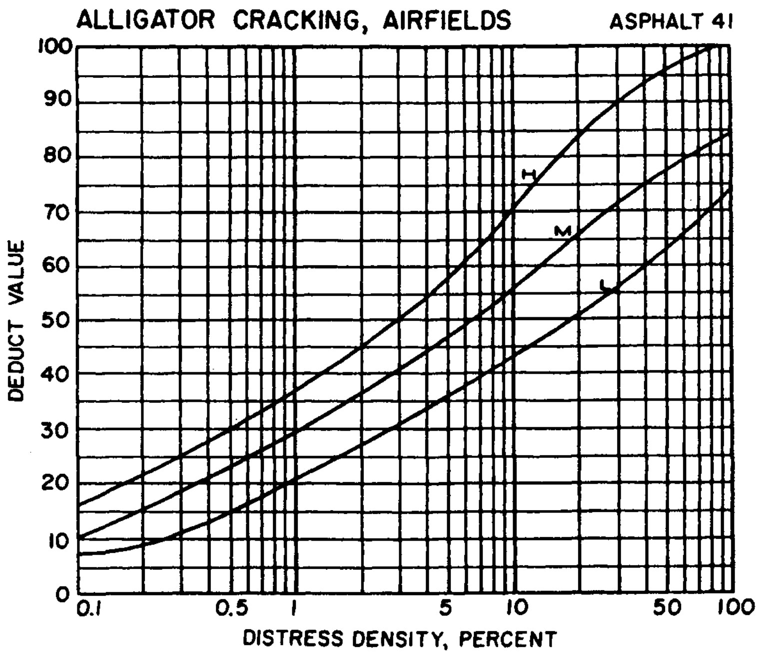


Figure D-17. Flexible Deduct Values, Distress 41, Alligator Cracking.

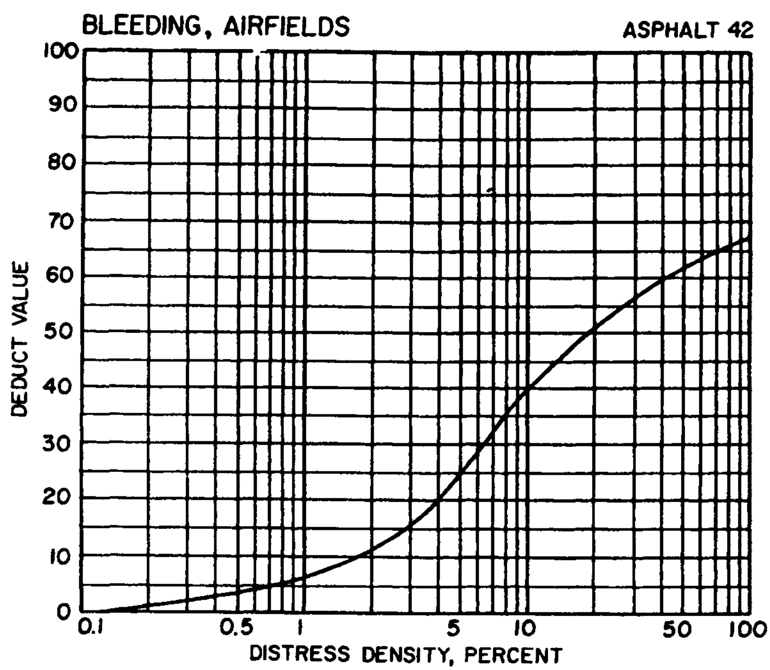


Figure D-18. Flexible Pavement Deduct Values, Distress 42, Bleeding.

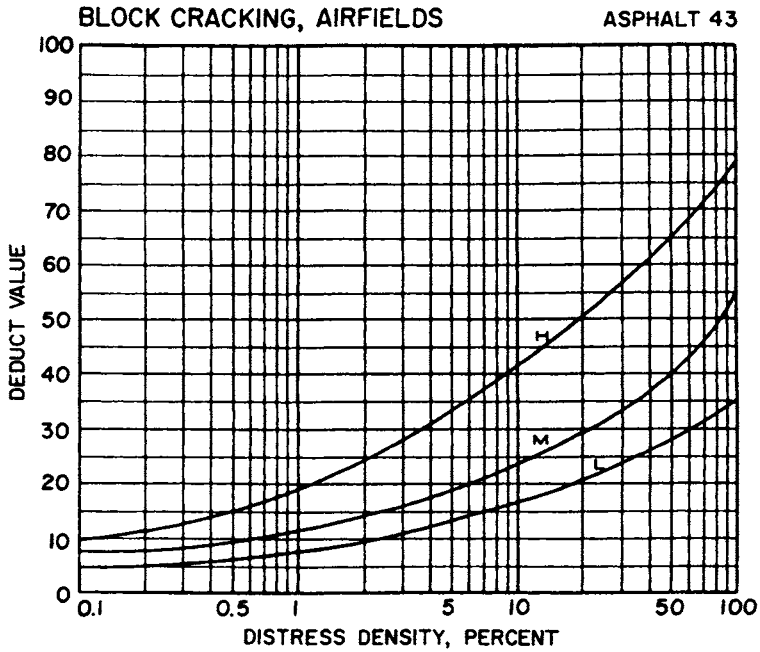


Figure D-19. Flexible Pavement Deduct Values. Distress 43. Block Cracking.

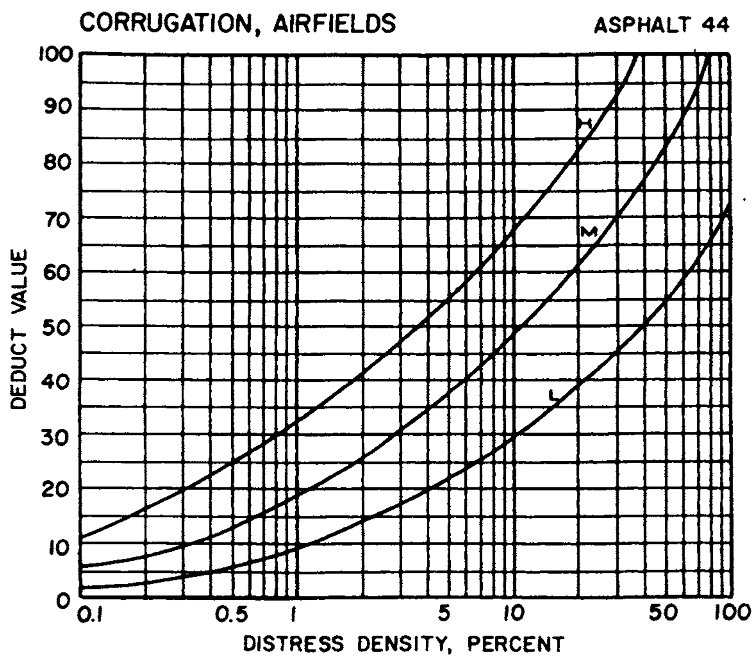


Figure D-20. Flexible Pavement Deduct Values, Distress 44, Corrugation.

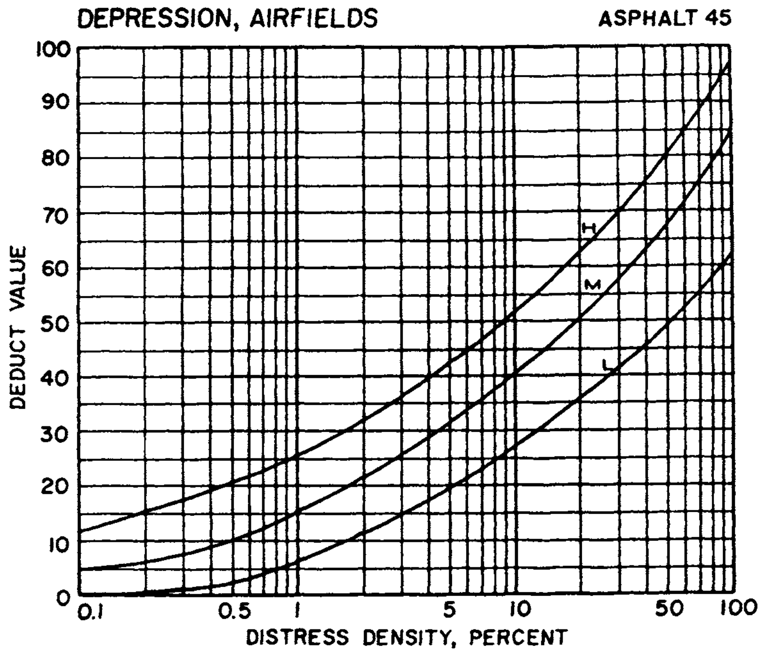


Figure D-21. Flexible Pavement Deduct Values. Distress 45 Depression.

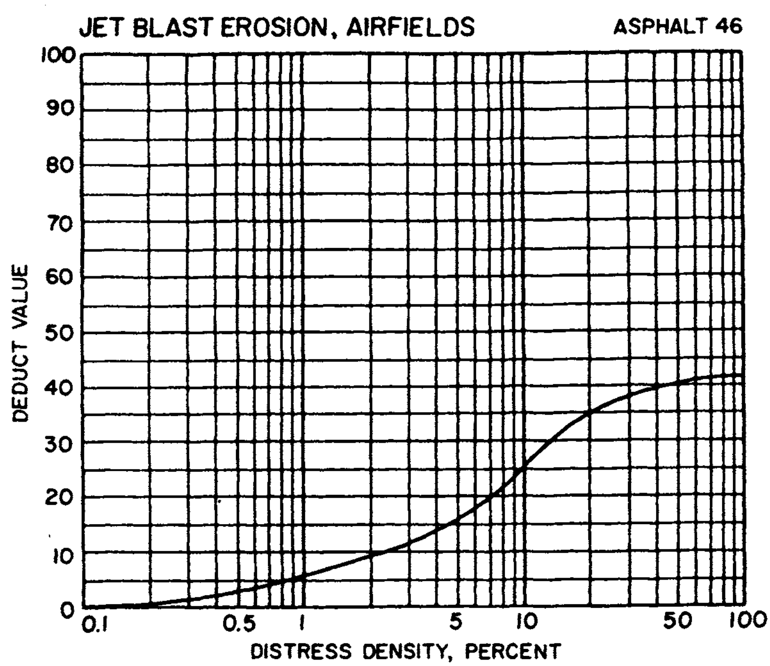
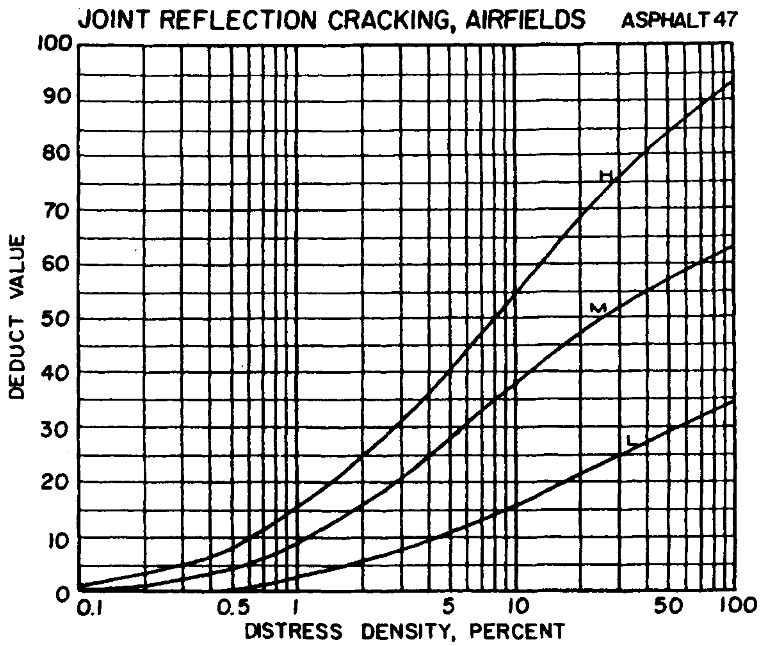


Figure D-22. Flexible Pavement Deduct Values, Distress 46, Joint Erosion.



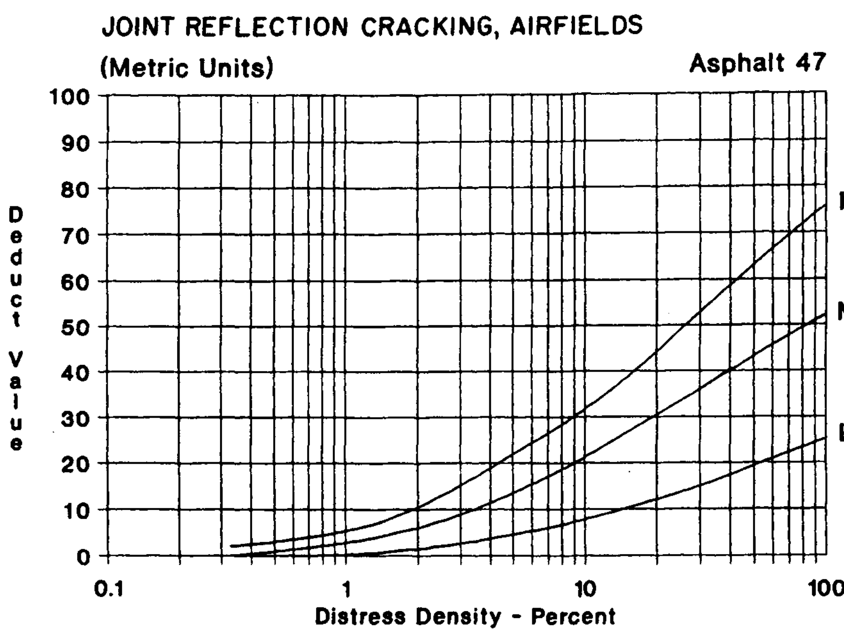
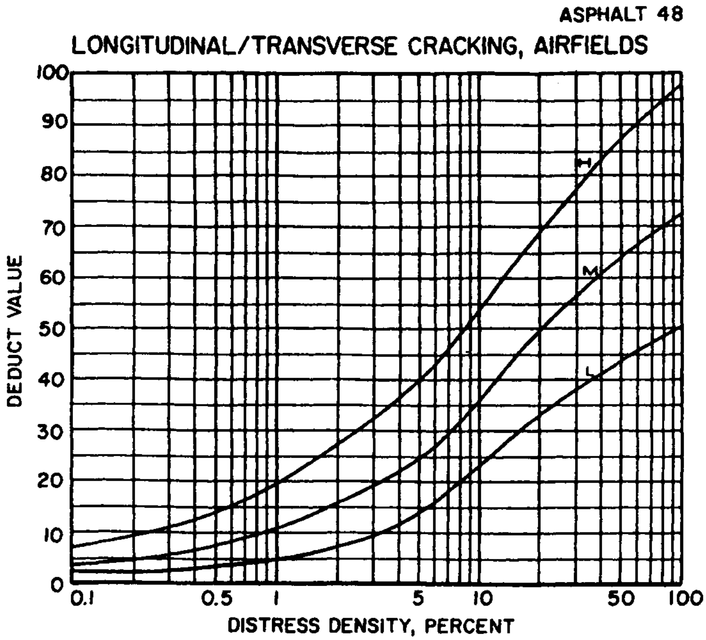


Figure D-23. Flexible Pavement Deduct Values, Distress 47, Joint Reflection Cracking.



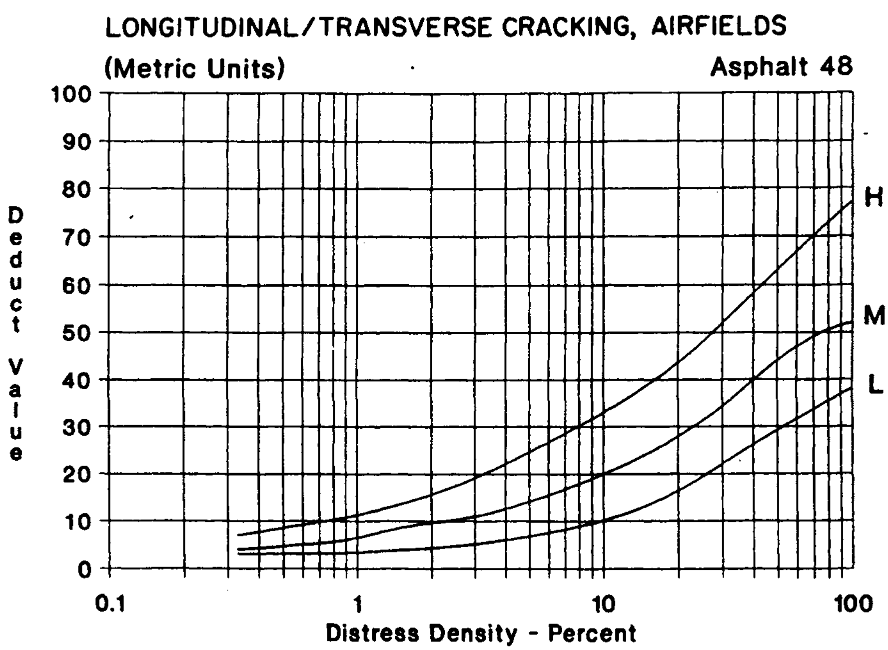


Figure D-24. Flexible Pavement Deduct Values. Distress 48. Longitudinal and Transverse Cracking.

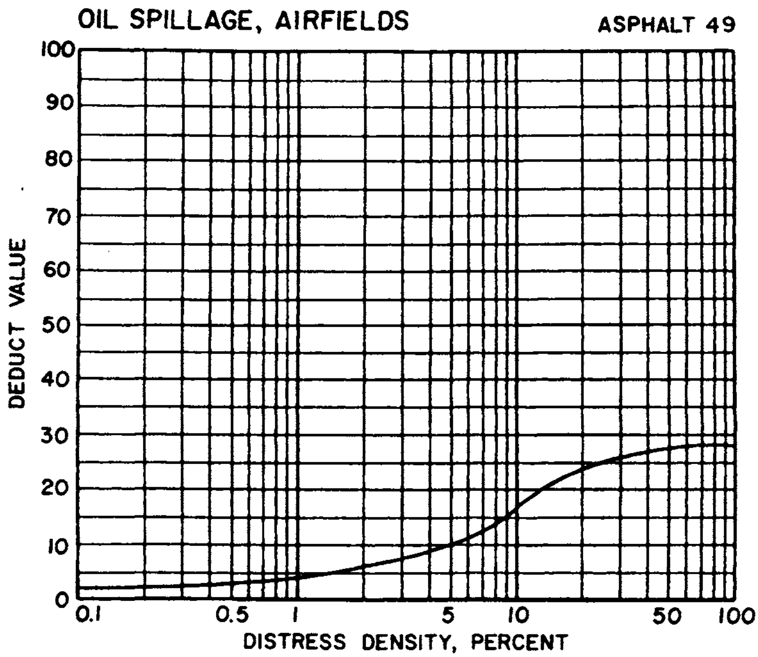


Figure D-25. Flexible Pavement Deduct Values, Distress 49, Oil Spillage.

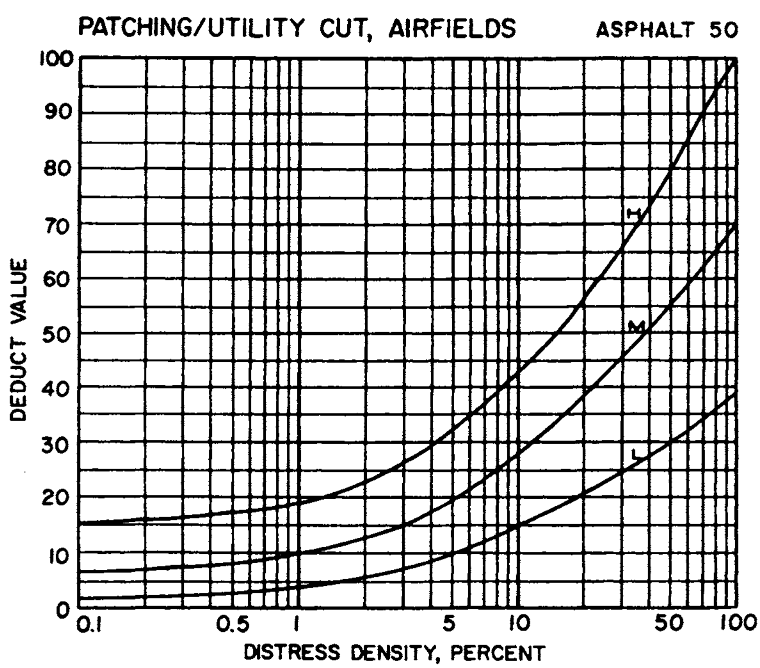


Figure D-26. Flexible Pavement Deduct Values, Distress 50, Patching and Utility Cut.

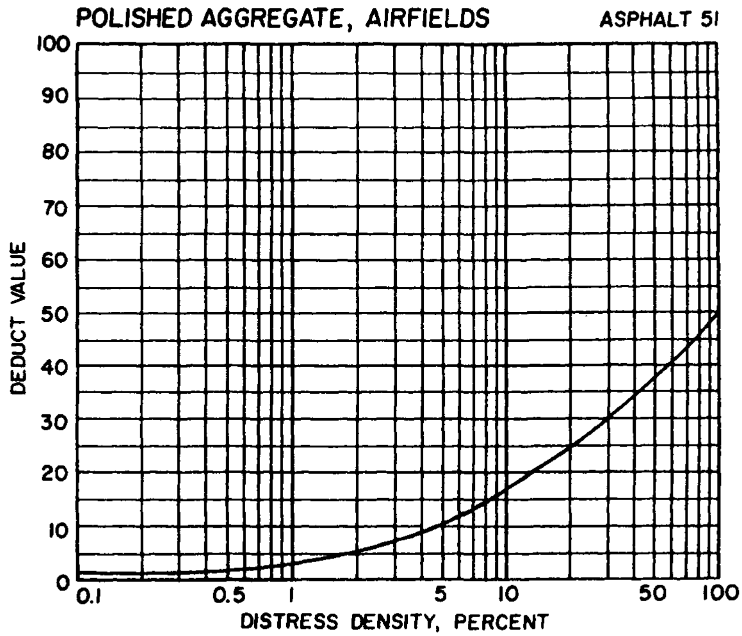


Figure D-27. Flexible Pavement Deduct Values. Distress 51. Polished Aggregate.

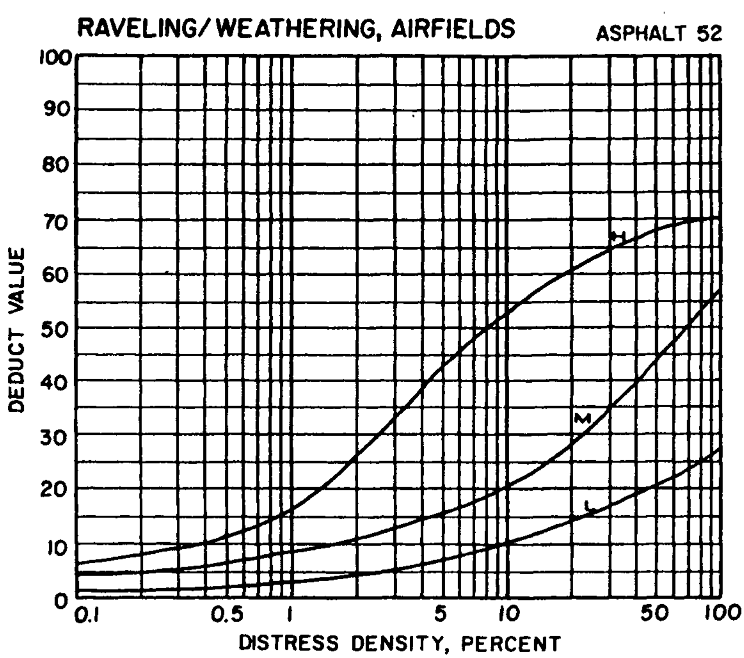


Figure D-28. Flexible Pavement Deduct Values. Distress 52, Raveling/Weathering.

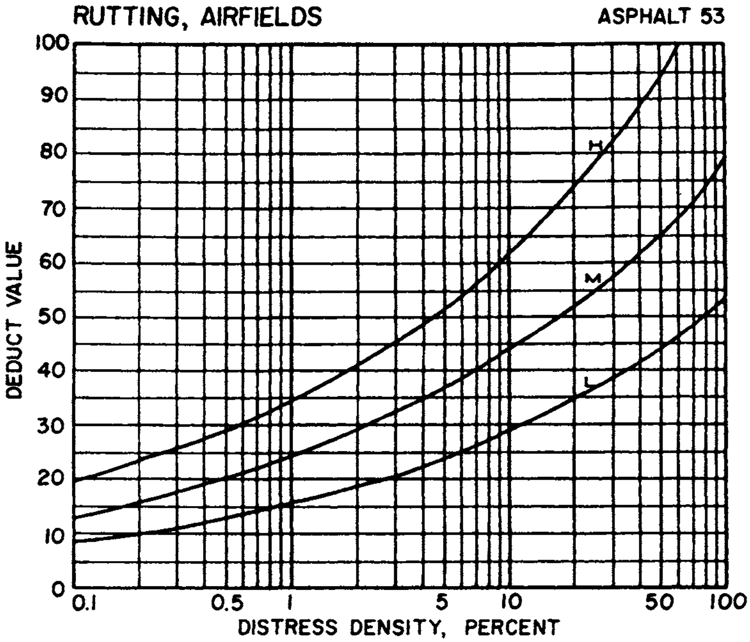


Figure D-29. Flexible Pavement Deduct Values, Distress 53, Rutting.

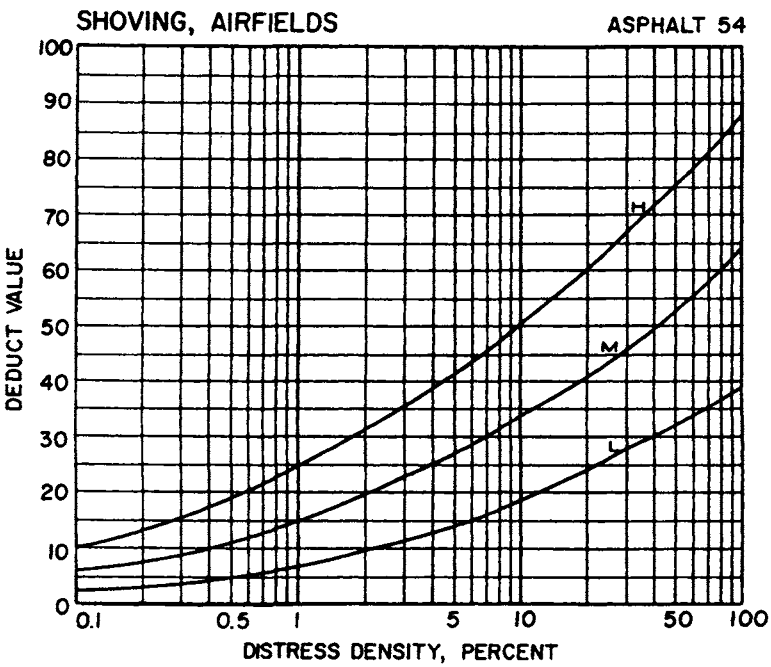


Figure D-30. Flexible Pavement Deduct Values, Distress 54, Shoving of Flexible Pavement by PCC Slabs.

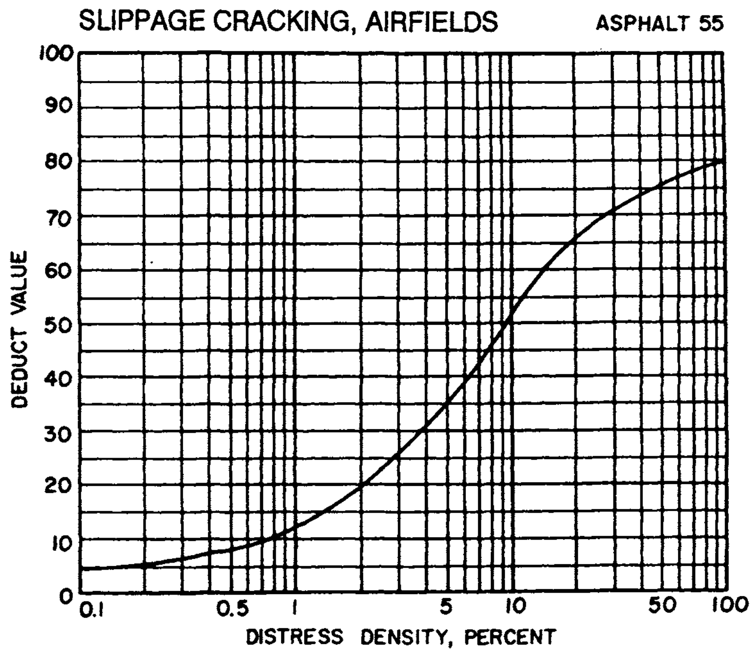


Figure D-31. Flexible Pavement Deduct Values, Distress 55, Slippage Cracking.

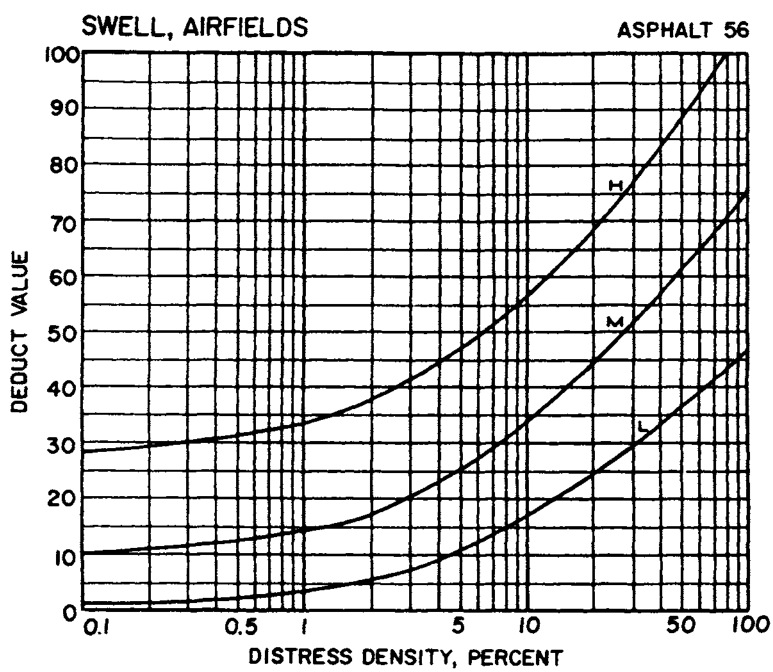


Figure D-32. Flexible Pavement Deduct Values, Distress 56, Swell.

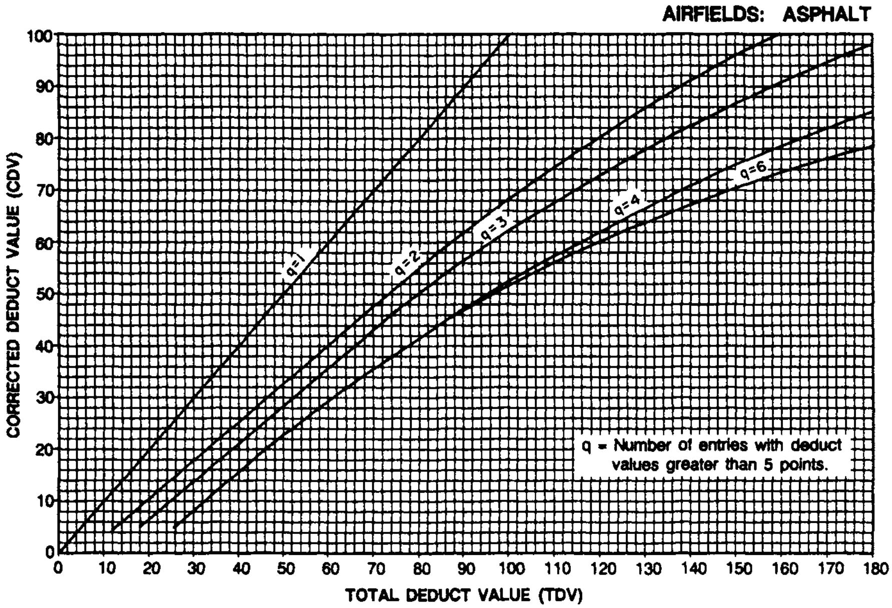


Figure D-33. Corrected Deduct Values for Asphalt or Tar-surfaced Pavements.

APPENDIX E

Portland Cement Concrete Airfields: Distress Definitions and Deduct Value Curves

Blowup (61)

Description

Blowups occur in hot weather, usually at a transverse crack or joint that is not wide enough to permit expansion by the concrete slabs. The insufficient width is usually caused by infiltration of incompressible materials into the joint space. When expansion cannot relieve enough pressure, a localized upward movement of the slab edges (buckling) or shattering will occur in the vicinity of the joint. Blowups can also occur at utility cuts and drainage inlets. This type of distress is almost always repaired immediately because of severe damage potential to aircraft. Blowups are included for reference when closed sections are being evaluated for reopening.

Severity Levels (Figure E–1)

L—Buckling or shattering has not rendered the pavement inoperative, and only a slight amount of roughness exists.

M—Buckling or shattering has not rendered the pavement inoperative, but a significant amount of roughness exists.

H—Buckling or shattering has rendered the pavement inoperative.

Note: For pavements to be considered operational, all foreign material from blowups must have been removed.

How to Count

A blowup usually occurs at a transverse crack or joint. At a crack, it is counted as being in one slab, but at a joint, two slabs are affected and the distress should be recorded as occurring in two slabs.

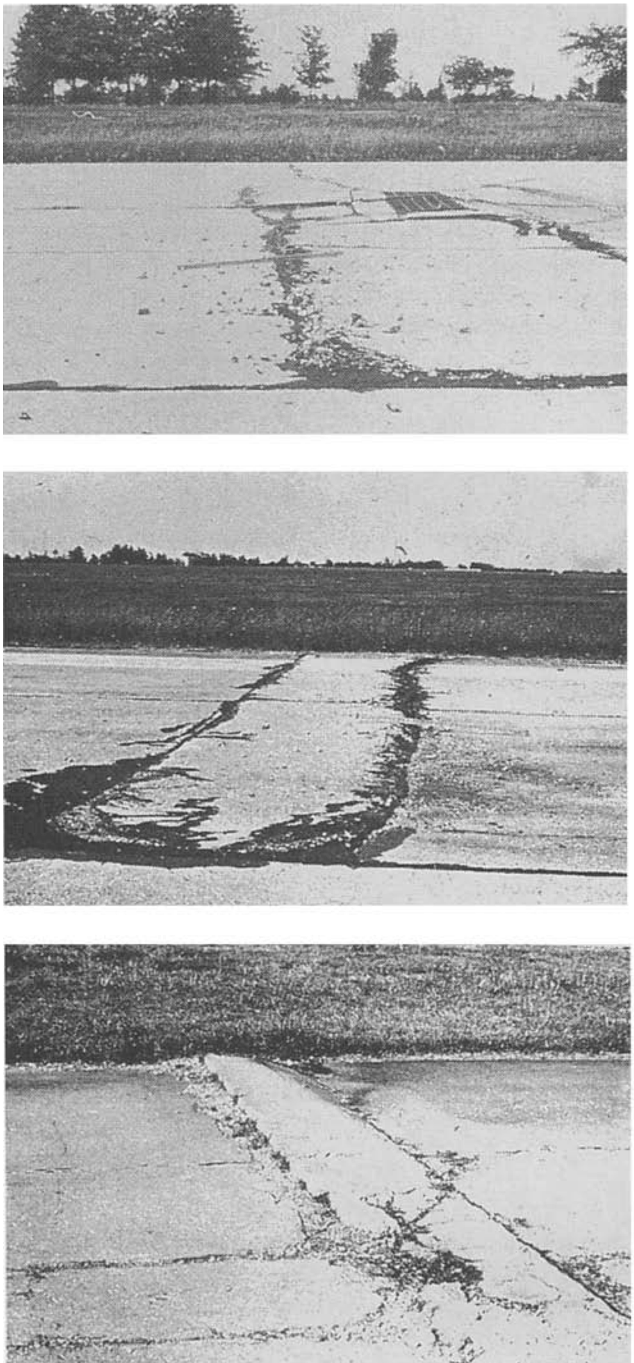
Options for Repair

L[a]—Partial, or full-depth patch; Slab replacement.

M[a]—Partial or full-depth patch; Slab replacement.

H[a]—Full-depth patch; Slab replacement.

[a]Must provide expansion joints.



LOW

MEDIUM

HIGH

Figure E-1. Blowup.

Corner Break (62)

Description

A corner break is a crack that intersects the joints at a distance less than or equal to one-half the slab length on both sides, measured from the corner of the slab. For example, a slab with dimensions of 25 by 25 ft (7.5 by 7.5 m) that has a crack intersecting the joint 5 ft (1.5 m) from the corner on one side and 17 ft (2.1 m) on the other side is not considered a corner break; it is a diagonal crack. However, a crack that intersects 7 ft (2.1 m) on one side and 10 ft (3 m) on the other is considered a corner break. A corner break differs from a corner spall in that the crack extends vertically through the entire slab thickness, while a corner spall intersects the joint at an angle. Load repetition combined with loss of support and curling stresses usually causes corner breaks.

Severity Levels (Figure E–2)

L—Crack has either no spalling or minor spalling (no foreign object damage (FOD) potential). If non-filled, it has a mean width less than approximately 1/8 in.(3.2 mm); a filled crack can be of any width, but the filler material must be in satisfactory condition. The area between the corner break and the joints is not cracked.

M—One of the following conditions exists:

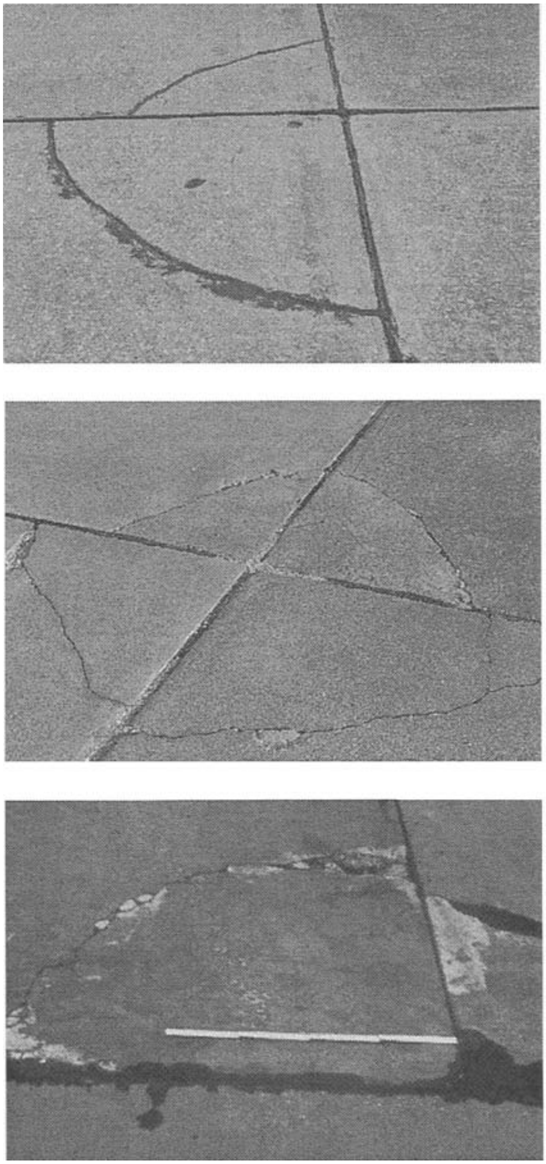
1. Filled or non-filled crack is moderately spalled (some FOD potential).
2. A non-filled crack has a mean width between 1/8 in. (3.2 mm) and 1 in. (25.4 mm).
3. A filled crack is not spalled or only lightly spalled, but the filler is in unsatisfactory condition.
4. The area between the corner break and the joints is lightly cracked with loose or missing particles.

H—One of the following conditions exists:

1. Filled or non-filled crack is severely spalled, causing definite FOD potential.
2. A non-filled crack has a mean width greater than approximately 1 in.(25.4 mm) creating a tire damage potential.
3. The area between the corner break and the joints is severely cracked.

How to Count

A distressed slab is recorded as one slab if it (1) contains a single corner break, (2) contains more than one break of a particular severity, or (3) contains two or more breaks of different severities. For two or more breaks, the highest level of severity should be recorded. For example, a slab containing both light and medium-severity corner breaks should be counted as one slab with a medium-severity corner break.



LOW

MEDIUM

HIGH

Figure E-2. Corner Break.

Options for Repair

L[a]—Do nothing; Seal cracks.

M[a]—Seal cracks; Full-depth patch; Slab replacement.

H[a]—Seal cracks; Full-depth patch; Slab replacement.

[a]Check for voids; consider undersealing project.

Cracks: Longitudinal, Transverse, and Diagonal (63)

Description

These cracks, which divide the slab into two or three pieces, are usually caused by a combination of load repetition, curling stresses, and shrinkage stresses. (For slabs divided into four or more pieces, see Shattered Slab/Intersecting Cracks.) Low-severity cracks are usually warping or friction-related and are not considered major structural distresses. Medium or high-severity cracks are usually working cracks and are considered major structural distresses.

Note: Hairline cracks that are only a few feet long and do not extend across the entire slab are rated as shrinkage cracks.

Non-reinforced PCC Severity Levels (Figure E–3)

L—(1) Crack has no spalling or minor spalling (no FOD potential). If non-filled, it is less than 1/8 in. (3.2 mm) wide; a filled crack can be of any width, but its filler material must be in satisfactory condition; or (2) the slab is divided into three pieces by low-severity cracks.

M—One of the following conditions exists:

1. A filled or non-filled crack is moderately spalled (some FOD potential).
2. A non-filled crack has a mean width between 1/8 in. (3.2 mm) and 1 in. (25.4 mm).
3. A filled crack has no spalling or minor spalling, but the filler is in unsatisfactory condition.
4. The slab is divided into three pieces by two or more cracks, one of which is at least medium severity.

H—One of the following conditions exists:

1. A filled or non-filled crack is severely spalled (definite FOD potential).
2. A non-filled crack has a mean width approximately greater than 1 in. (25.4 mm), creating tire damage potential.

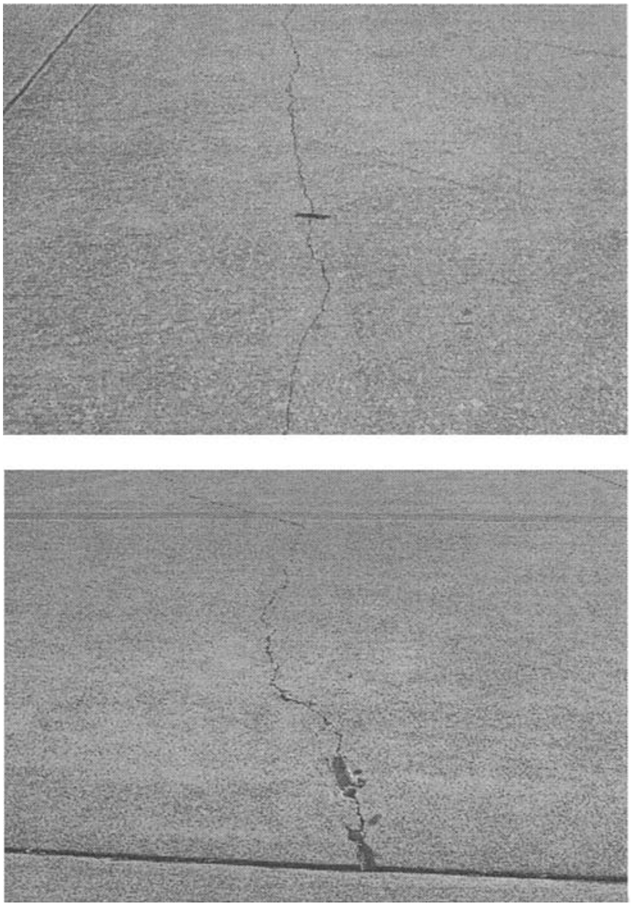


Figure E-3. LongitudinalCrack.

Reinforced Concrete Severity Levels

L—One of the following conditions exists:

1. Non-filled crack, 1/8 in. (3.2 mm) to 1/2 in. (12.7 mm) wide, with no faulting or spalling.
2. Filled or non-filled cracks of any width < 1/2 in. (12.7 mm), with low-severity spalling.
3. Filled cracks of any width (filler satisfactory), with no faulting or spalling.

Note: Crack less than 1/8 in. (3.2 mm) wide with no spalling or faulting should be counted as shrinkage cracking.

M—One of the following conditions exists:

1. Non-filled cracks, 1/2 in. (12.7 mm) to 1 in. (25.4 mm) wide, no faulting or spalling.
2. Filled cracks of any width, with faulting < 3/8 in. (9.6 mm) or medium-severity spalling.

H—One of the following conditions exists:

1. Non-filled cracks of width > 1 in. (25.4 mm).
2. Non-filled cracks of any width, with faulting > 3/8 in. (9.6 mm) or medium-severity spalling.
3. Filled cracks of any width, with faulting > 3/8 in. (9.6 mm) or high-severity spalling.

How to Count

Once the severity has been identified, the distress is recorded as one slab. If a crack is repaired by a narrow patch [e.g., 4 to 10 in. (102 to 254 mm)wide], only the crack and not the patch should be recorded at the appropriate severity level.
Slabs longer than 30 ft. (9.1m) are divided into approximately equal length "slabs" having imaginary joints assumed to be in perfect condition.

Options for Repair

L—Do nothing; Seal cracks.

M—Seal cracks.

H—Seal cracks; Full-depth patch; Slab replacement.

HIGH

Figure E-3. Severity Crack.

Durability ("D") Cracking (64)

Description

Durability cracking is caused by the inability of the concrete to withstand environmental factors such as freeze-thaw cycles. It usually appears as a pattern of cracks running parallel to a joint or linear crack. A dark coloring can usually be seen around the fine durability cracks. This type of cracking may eventually lead to disintegration of the concrete within 1 to 2 ft (0.3 to 0.6 m) of the joint or crack.

Severity Levels (Figure E–4c)

L—"D" cracking is defined by hairline cracks occurring in a limited area of the slab, such as one or two corners or along one joint. Little or no disintegration has occurred. No FOD potential.

M—(1) "D" cracking has developed over a considerable amount of slab area with little or no disintegration or FOD potential; or (2) "D" cracking has occurred in a limited area of the slab, such as in one or two corners or along one joint, but pieces are missing and disintegration has occurred. Some FOD potential.

H—"D" cracking has developed over a considerable amount of slab area with disintegration of FOD potential.

How to Count

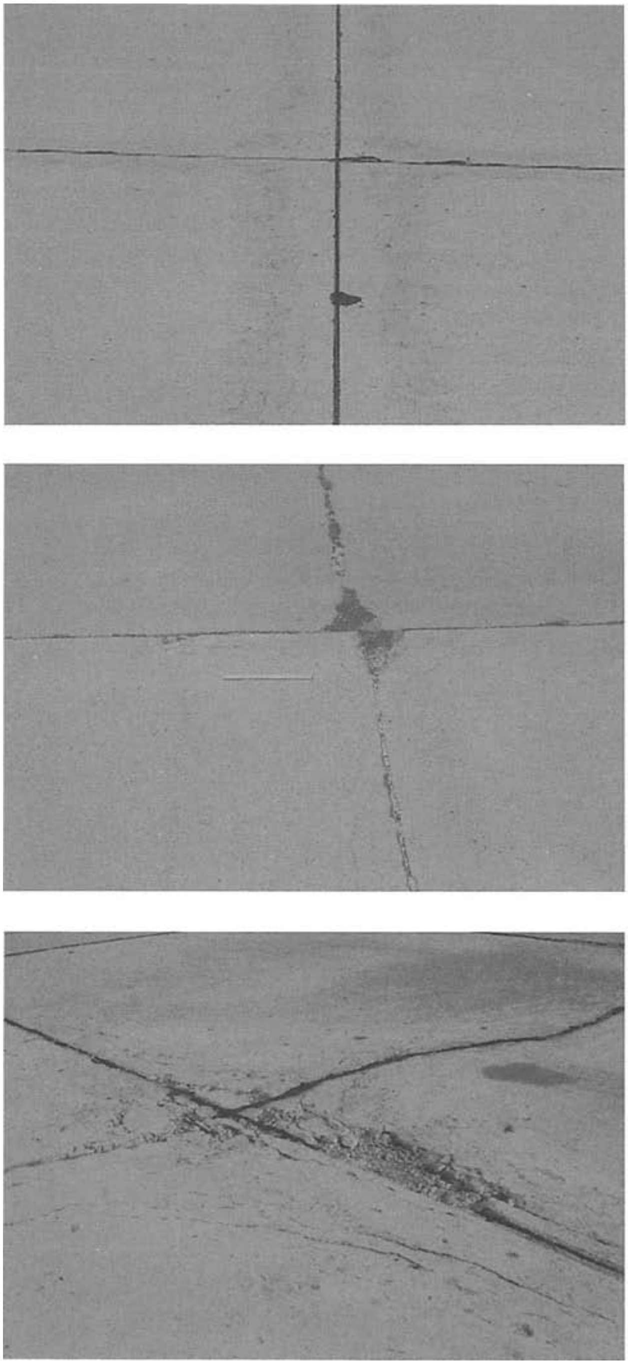
When the distress is located and rated at one severity, it is counted as one slab. If more than one severity level is found, the slab is counted as having the higher severity distress. If "D" cracking is counted, scaling on the same slab should not be recorded.

Options for Repair

L—Do nothing; Seal joints.

M—Full-depth patch; Reconstruct joints.

H—Full-depth patch; Reconstruct joints; Slab replacement.



LOW

MEDIUM

HIGH

Figure E-4. Durability Cracking.

Joint Seal Damage (65)

Description

Joint seal damage is any condition which enables soil or rocks to accumulate in the joints or allows significant infiltration of water. Accumulation of incompressible materials prevents the slabs from expanding and may result in buckling, shattering, or spalling. A pliable joint filler bonded to the edges of the slabs protects the joints from accumulation of materials and also prevents water from seeping down and softening the foundation supporting the slab.

Typical types of joint seal damage are (1) stripping of joint sealant, (2) extrusion of joint sealant, (3) weed growth, (4) hardening of the filler (oxidation), (5) loss of bond to the slab edges, and (6) lack or absence of sealant in the joint.

Severity Levels (Figure E–5)

L—Joint sealer is in generally good condition throughout the section. Sealant is performing well, with only a minor amount of any of the above types of damage present.

M—Joint sealer is in generally fair condition over the entire surveyed section, with one or more of the above types of damage occurring to a moderate degree. Sealant needs replacement within 2 years.

H—Joint sealer is in generally poor condition over the entire surveyed section, with one or more of the above types of damage occurring to a severe degree. Sealant needs immediate replacement.

How to Count

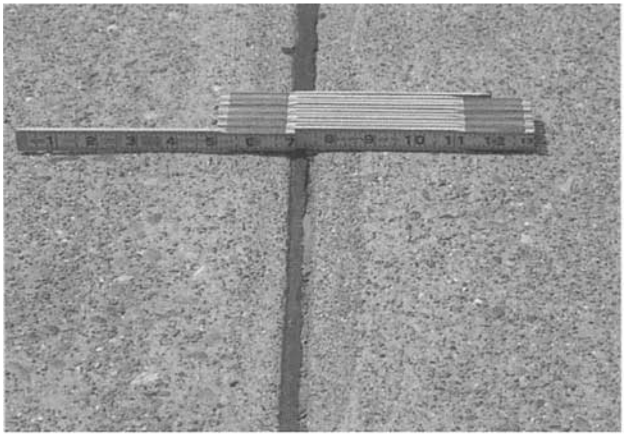
Joint seal damage is not counted on a slab-by-slab basis but is rated based on the overall condition of the sealant in the sample unit.

Options for Repair

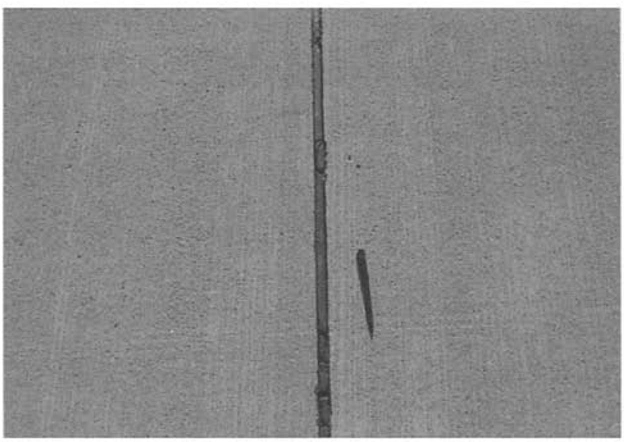
L—Do nothing.

M—Seal joints.

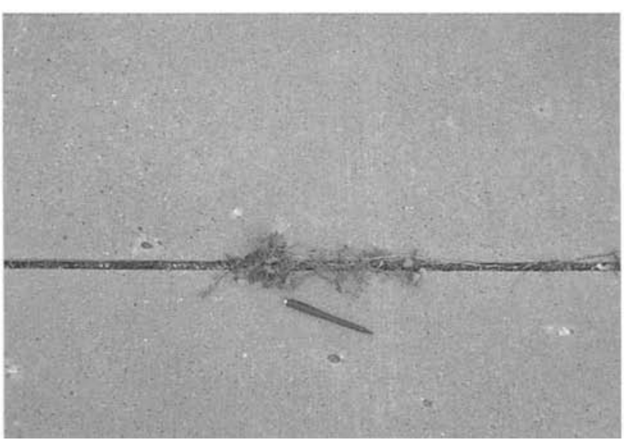
H—Seal joints.



LOW



MEDIUM



HIGH

Figure E-5. Joint Seal Damage.

Patching, Small [Less than 5 ft^2 (1.5 m^2)] (66)

Description

A patch is an area where the original pavement has been removed and replaced by a filler material. For condition evaluation, patching is divided into two types: small [less than 5 ft^2 (1.5 m^2)] and large [over 5 ft^2 (1.5 m^2)]. Large patches are described in the next section.

Severity Levels (Figure E–6)

L—Patch is functioning well, with little or no deterioration.

M—Patch has deteriorated, and/or moderate spalling can be seen around the edges. Patch material can be dislodged, with considerable effort (minor FOD potential).

H—Patch has deteriorated, either by spalling around the patch or cracking within the patch, to a state which warrants replacement.

How to Measure

If one or more small patches having the same severity level are located in a slab, it is counted as one slab containing that distress. If more than one severity level occurs, it is counted as one slab with the higher severity level being recorded.

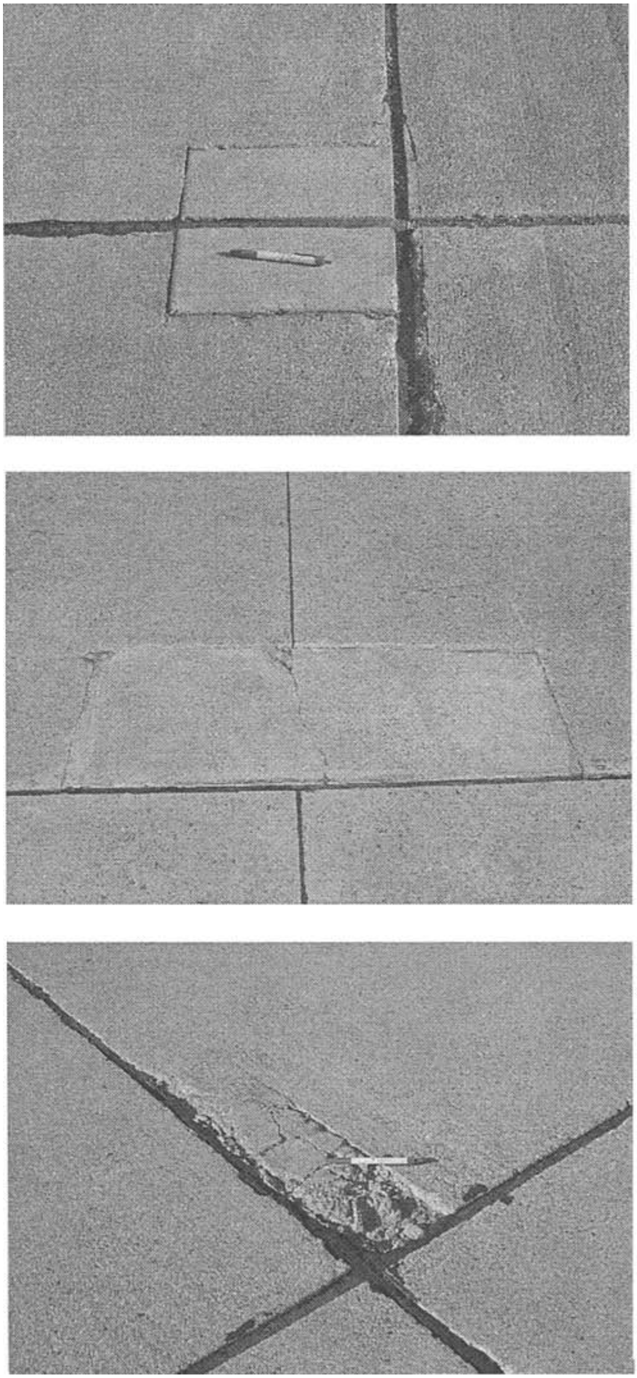
If a crack is repaired by a narrow patch [e.g., 4 to 10 in. (102 to 254 mm) wide], only the crack and not the patch should be recorded at the appropriate severity level. If the original distress of a patch is more severe than the patch itself, the original distress type should be recorded.

Options for Repair

L—Do nothing.

M—Replace patch.

H—Replace patch.



LOW

MEDIUM

HIGH

Figure E-6. Patching,Small.

Patching, Large [Over 5 ft^2 (0.45 m^2)] and Utility Cuts (67)

Description

Patching is the same as defined in the previous section. A utility cut is a patch that has replaced the original pavement because of placement of underground utilities. The severity levels of a utility cut are the same as those for regular patching.

Severity Levels (Figure E–7)

L—Patch is functioning well with very little or no deterioration.

M—Patch has deteriorated and/or moderate spalling can be seen around the edges. Patch material can be dislodged with considerable effort, causing some FOD potential.

H—Patch has deteriorated to a state which causes considerable roughness and/or high FOD potential. The extent of the deterioration warrants replacement of the patch.

How to Count

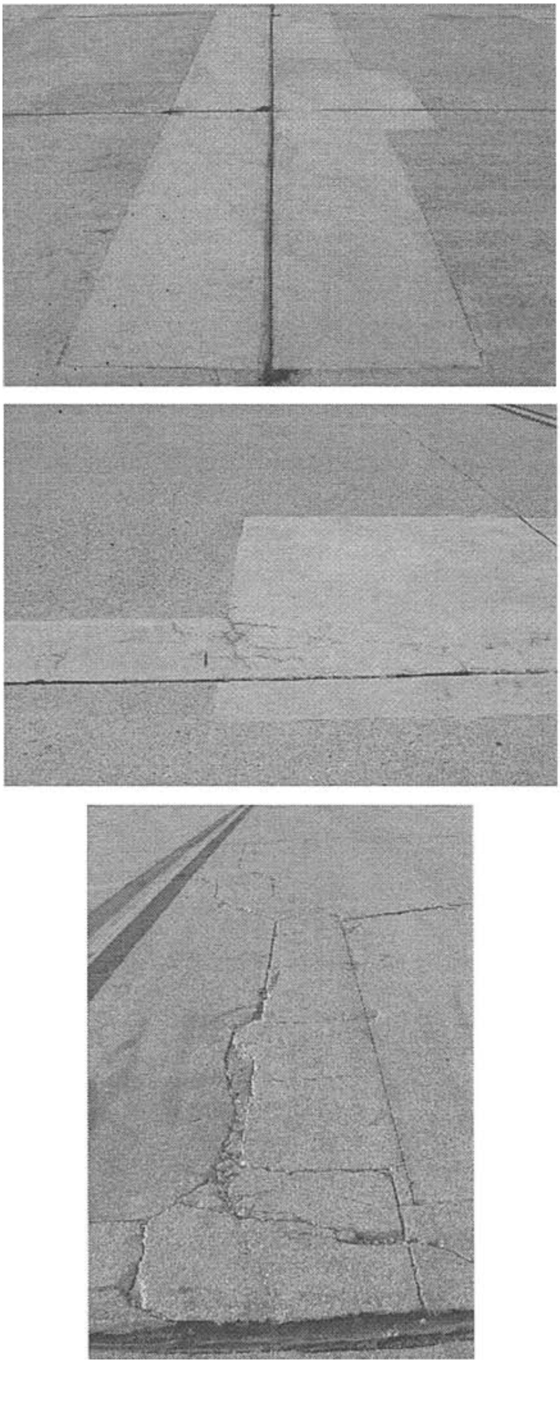
The criteria are the same as for small patches.

Options for Repair

L—Do nothing.

M—Seal cracks; Repair distressed area; Replace patch.

H—Replace patch; Slab replacement.



LOW

MEDIUM

HIGH

Figure E-7. Patching, Large.

Popouts (68)

Description

A popout is a small piece of pavement that breaks loose from the surface due to freeze-thaw action in combination with expansive aggregates. Popouts usually range from approximately 1 to 4 in. (25 to 102 mm) in diameter and from 1/2 to 2 in. (13 to 51 mm) deep.

Severity Levels (Figure E–8)

No degrees of severity are defined for popouts. However, popouts must be extensive before they are counted as a distress, that is, average popout density must exceed approximately three popouts per square yard over the entire slab area.

How to Count

The density of the distress must be measured. If there is any doubt about the average being greater than three popouts per square yard (per square meter), at least three, random, 1-square yard (1 m2) areas should be checked. When the average is greater than this density, the slab is counted.

Options for Repair

Do nothing.

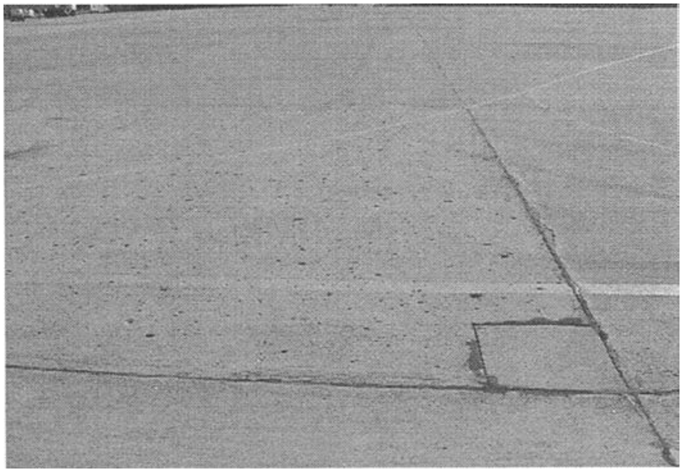


Figure E-8. Popouts.

Pumping (69)

Description

Pumping is the ejection of material by water through joints or cracks caused by deflection of the slab under passing loads. As the water is ejected, it carries particles of gravel, sand, clay, or silt and results in a progressive loss of pavement support. Surface staining and base or subgrade material on the pavement close to joints or cracks are evidence of pumping. Pumping near joints indicates poor joint sealer and loss of support which will lead to cracking under repeated loads.

Severity Levels (Figure E–9)

No degrees of severity are defined. It is sufficient to indicate that pumping exists.

How to Count

Slabs are counted as follows: one pumping joint between two slabs is counted as two slabs. However, if the remaining joints around the slab are also pumping, one slab is added per additional pumping joint.

Options for Repair

Seal cracks and joints; Underseal.

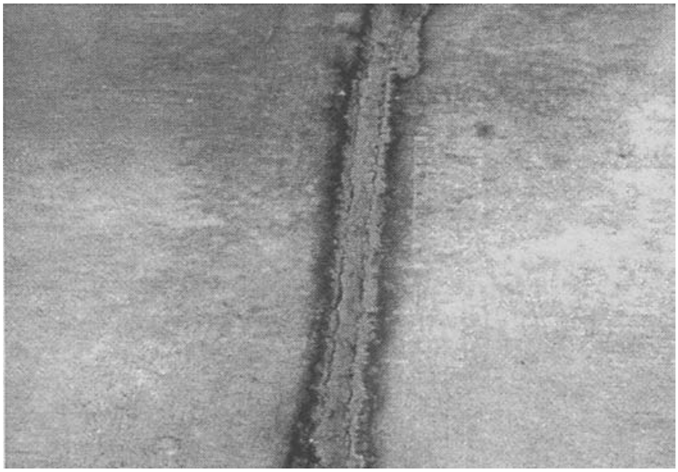


Figure E-9. Pumping.

Scaling, Map Cracking, and Crazing (70)

Description

Map cracking or crazing refers to a network of shallow, fine, or hairline cracks which extend only through the upper surface of the concrete. The cracks tend to intersect at angles of 120 degrees. Map cracking or crazing is usually caused by over-finishing the concrete and may lead to scaling of the surface. Scaling is the breakdown of the slab surface to a depth of approximately 1/4 to 1/2 in. (6 to 13 mm). Scaling may also be caused by deicing salts, improper construction, freeze-thaw cycles, and poor aggregate. Another recognized source of distress is the Alkali Silica Reaction (ASR) which is the reaction between the alkalies (Na_2O and K_2O) in some cements and certain minerals in some aggregates the gel from the reaction is white. Products formed by the reaction between the alkalies and aggregate result in expansions that cause a breakdown in the concrete and may affect adjacent structures. Cracks near the joints will also tend to be perpendicular to the joints as compared to "D" cracking which is parallel to the joints.

Severity Levels Applicable to Scaling (Not Applicable to ASR) (Figure E–10)

L—Crazing or map cracking exists over most of the slab area; the surface is in good condition with no scaling.

Note: The low-severity level is an indicator that scaling may develop in the future. A slab should only be counted if, in the judgment of the pavement inspector, future scaling is likely to occur within 2 to 3 years.

M—Scaling covers approximately 5 percent or less of the surface, causing some FOD potential.

H—Slab is severely scaled, causing a high FOD potential. Usually more than 5 percent of the surface is affected.

Severity Levels Applicable to ASR

L—ASR is noted on only a small portion of the slab and produces no FOD.

M—ASR is noted over the entire slab, but cracks are tight and no loose aggregate exists; or ASR covers 5 percent or less of the surface and causes some FOD potential.

H—ASR covers more than 5 percent of the surface and causes a high FOD potential.

How to Count

If two or more levels of severity exist on a slab, the slab is counted as one slab having the maximum level of severity. For example, if both low-severity crazing and medium scaling exist on one slab, the slab is counted as one slab containing medium scaling. If "D" cracking is counted, scaling is not counted.

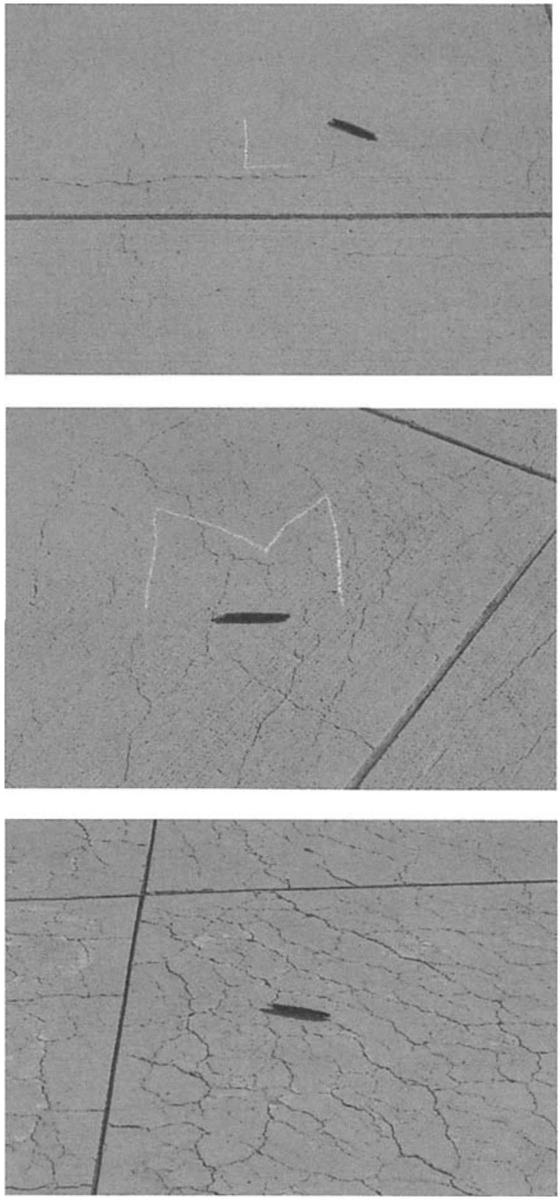


Figure E–10. Scaling, Map Cracking, and Crazing.

Options for Repair

L—Do nothing

M—Partial-depth patch; Slab replacement.

H—Slab replacement.

Settlement or Faulting (71)

Description

Settlement or faulting is a difference of elevation at a joint or crack caused by upheaval or consolidation.

Severity Levels (Figure E–11)

Severity levels are defined by the difference in elevation across the fault and the associated decrease in ride quality and safety as severity increases.

Difference in Elevation:	Runways/Taxiways	Aprons
L	<1/4 in. (6.4 mm)	1/8 to 1/2 in. (3.2 to 12.7 mm) (Fig. E-11a).
M	1/4 to 1/2 in. (6.4 to 12.7 mm)	1/2 to 1 in. (12.7 to 25.4 mm) (Fig. E-11b).
H	>1/2 in. (12.7 mm)	>1 in. (25.4 mm) (Fig. E-11c).

How to Count

In counting settlement, a fault between two slabs is counted as one slab. A straightedge or level should be used to aid in measuring the difference in elevation between the two slabs.

Options for Repair

L[a]—Do nothing.

M[a]—Slab grinding.

H[a]—Slab grinding; Slab replacement.

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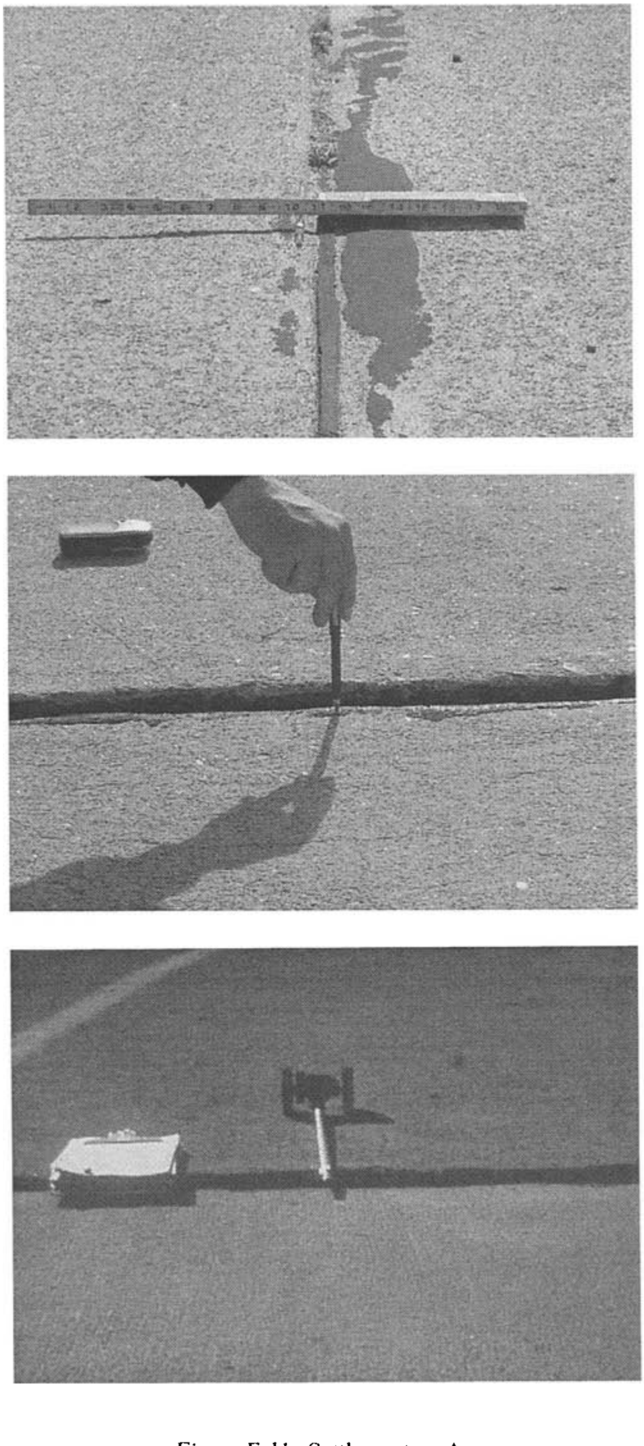
[a]Check for joint seal damage and voids. Consider undersealing and joint seal project.



LOW

MEDIUM

HIGH

Figure E-11. Settlement on Apron.

Shattered Slab Intersecting Cracks (72)

Description

Intersecting cracks are cracks that break into four or more pieces because of overloading and/or inadequate support. The high-severity level of this distress type, as defined below, is referred to as a shattered slab. If all pieces or cracks are contained within a corner break, the distress is categorized as a severe corner break.

Severity Levels (Figure E–12)

L—Slab is broken into four or five pieces with the vast majority of the cracks (over 85 percent) of low-severity.

M—(1) Slab is broken into four or five pieces with over 15 percent of the cracks of medium severity (no high-severity cracks); or (2) slab is broken into six or more pieces with over 85 percent of the cracks of low- severity.

H—At this level of severity, the slab is called shattered:

1. Slab is broken into four or five pieces with some or all of the cracks of high severity.
2. Slab is broken into six or more pieces with over 15 percent of the cracks of medium- or high-severity.

How to Count

No other distress such as scaling, spalling, or durability cracking should be recorded if the slab is medium or high-severity level, since the severity of this distress would affect the slab's rating substantially.

Options for Repair

L—Seal cracks.

M—Seal cracks; Full-depth patch; Slab replacement.

H—Full-depth patch; Slab replacement.

LOW

MEDIUM

HIGH

Figure E-12. Intersecting Cracks.

Shrinkage Cracks (73)

Description

Shrinkage cracks are hairline cracks that are usually only a few feet long and do not extend across the entire slab. They are formed during the setting and curing of the concrete and usually do not extend through the depth of the slab.

Severity Levels (Figure E–13)

No degrees of severity are defined. It is sufficient to indicate that shrinkage cracks exist.

How to Count

If one or more shrinkage cracks exist on one particular slab, the slab is counted as one slab with shrinkage cracks.

Options for Repair

Do nothing.

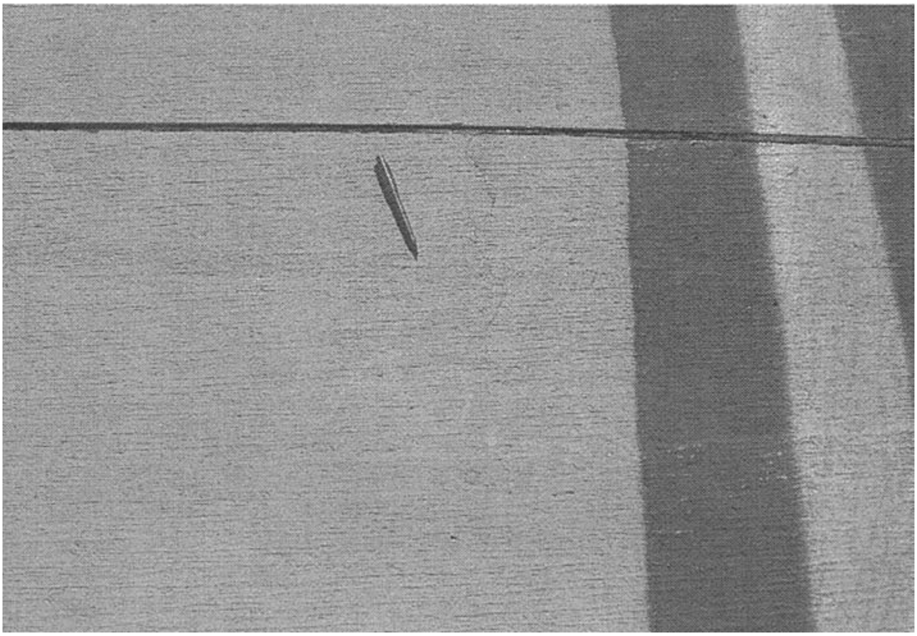


Figure E-13. Shrinkage Cracks.

Spalling (Transverse and Longitudinal Joints) (74)

Description

Joint spalling is the breakdown of the slab edges within 2 ft (0.6 m) of the side of the joint. A joint spall usually does not extend vertically through the slab but intersects the joint at an angle. Spalling results from excessive stresses at the joint or crack caused by infiltration of incompressible materials or traffic loads. Weak concrete at the joint (caused by overworking) combined with traffic loads is another cause of spalling.

Severity Levels (Fig E–14).

L—(1) Spall over 2 ft (0.6 m) long: (a) spall is broken into no more than 3 pieces defined by low or medium-severity cracks; little or no FOD potential exists; or (b) joint is lightly frayed; little or no FOD potential exists (Fig. E-14a). (2) Spall is less than 2 ft (0.6m) long: spall is broken into pieces or fragmented, little FOD or tire damage potential exists.

M—(1) Spall over 2 ft (0.6 m) long: (a) spall is broken into more than three pieces defined by light or medium cracks; (b) spall is broken into no more than three pieces with one or more of the cracks being severe with some FOD potential existing; or (c) joint is moderately frayed, with some FOD potential (Fig. E-14b). (2) Spall less than 2 ft (0.6 m) long: spall is broken into pieces or fragmented, with some of the pieces loose or absent, causing considerable FOD or tire damage potential.

H—(1) Spall greater than 2 ft (0.6 m) long: (a) spall is broken into more than three pieces defined by one or more high-severity cracks with high FOD potential; or (b) joint is severely frayed, with high FOD potential. *Note:* If greater than 2 ft (0.6 m) of the joint is lightly frayed, the spall should not be counted.

How to Count

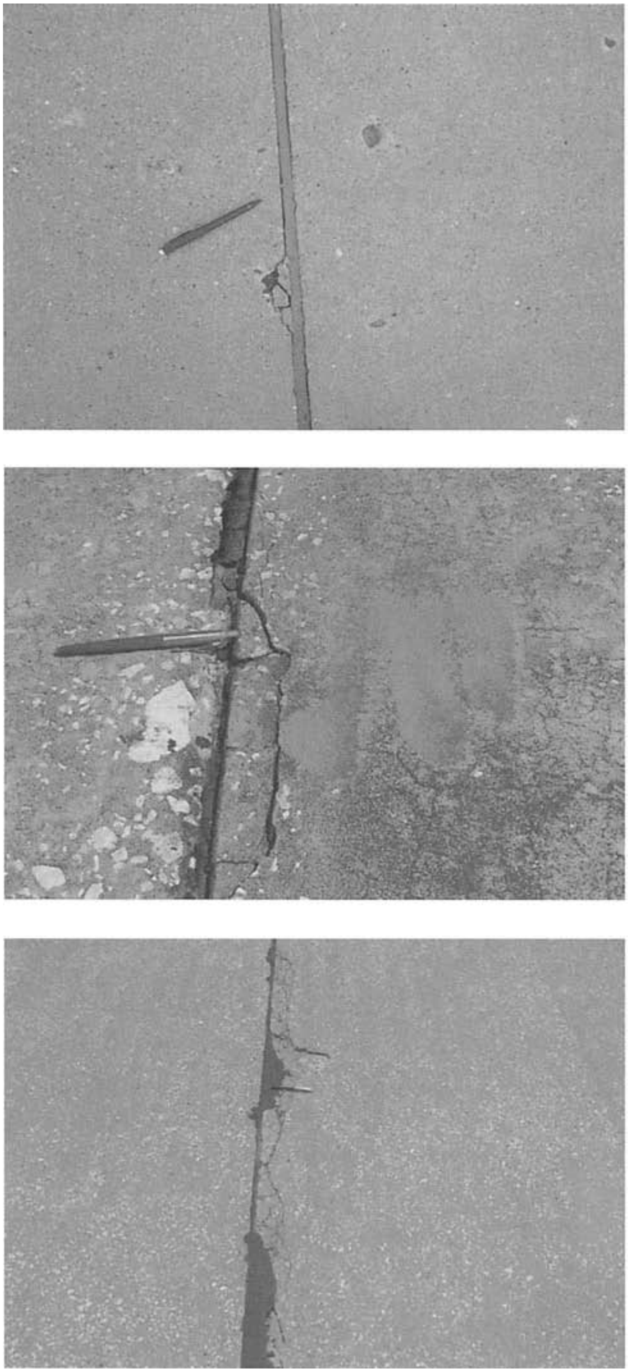
If the joint spall is located along the edge of one slab, it is counted as one slab with joint spalling. If spalling is located on more than one edge of the same slab, the edge having the highest severity is counted and recorded as one slab. Joint spalling can also occur along the edges of two adjacent slabs. If this is the case, each slab is counted as having joint spalling. If a joint spall is small enough to be filled during a joint seal repair, it should not be recorded.

Options for Repair

L—Do nothing.

M—Partial-depth patch.

H—Partial-depth patch.



LOW

MEDIUM

HIGH

Figure E-14. Joint Spall

Spalling, Corner (75)

Description

Corner spalling is the raveling or breakdown of the slab within approximately 2 ft (0.6 m) of the corner. A corner spall differs from the corner break in that the spall angles downward to intersect the joint, while a break extends vertically through the slab.

Severity Levels (Figure E–15)

L—One of the following conditions exists:

1. Spall is broken into one or two pieces defined by low-severity cracks (little or no FOD potential).
2. Spall is defined by one medium-severity crack (little or no FOD potential).

M—One of the following conditions exists:

1. Spall is broken into two or more pieces defined by medium-severity crack(s), and a few small fragments may be absent or loose.
2. Spall is defined by one severe, fragmented crack that may be accompanied by a few hairline cracks.
3. Spall has deteriorated to the point where loose material is causing some FOD potential.

H—One of the following conditions exists:

1. Spall is broken into two or more pieces defined by high-severity fragmented crack(s), with loose or absent fragments.
2. Pieces of the spall have been displaced to the extent that a tire damage hazard exists.
3. Spall has deteriorated to the point where loose material is causing high FOD potential.

How to Count

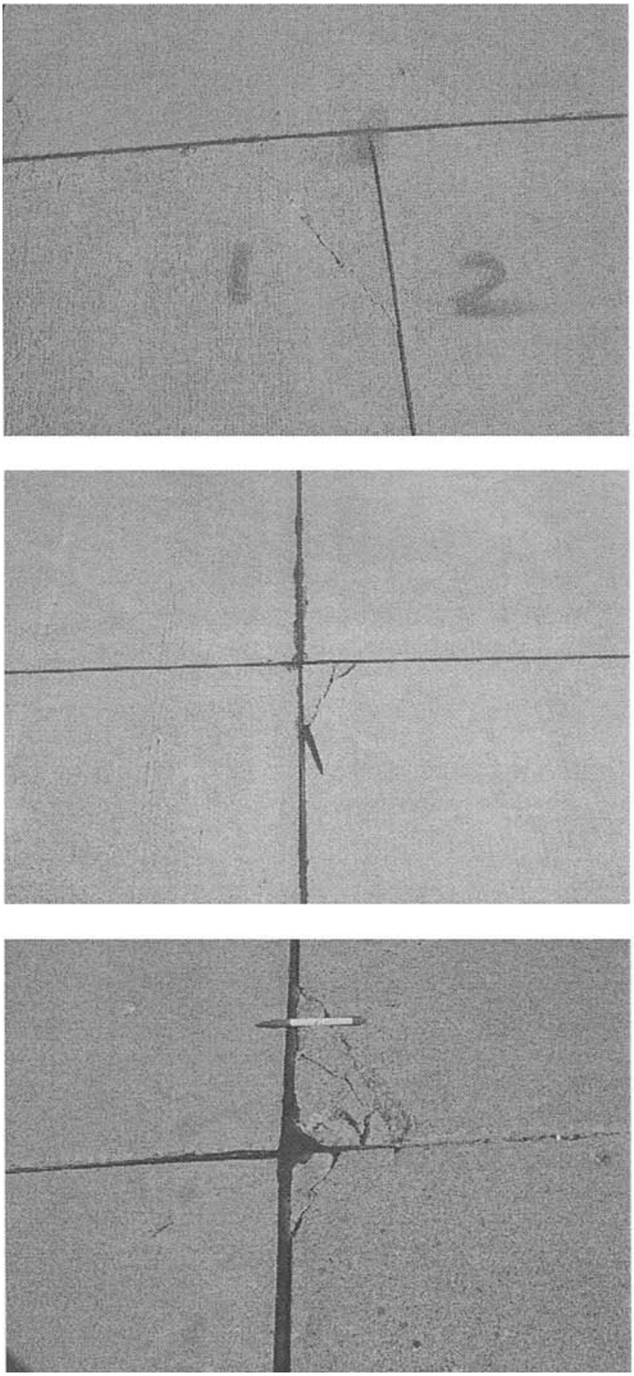
If one or more corner spalls having the same severity level are located in a slab, the slab is counted as one slab with corner spalling. If more than one severity level occurs, it is counted as one slab having the higher severity level.

Options for Repair

L—Do nothing.

M—Partial-depth patch.

H—Partial depth patch.



LOW

MEDIUM

HIGH

Figure E-15. CornerSpall.

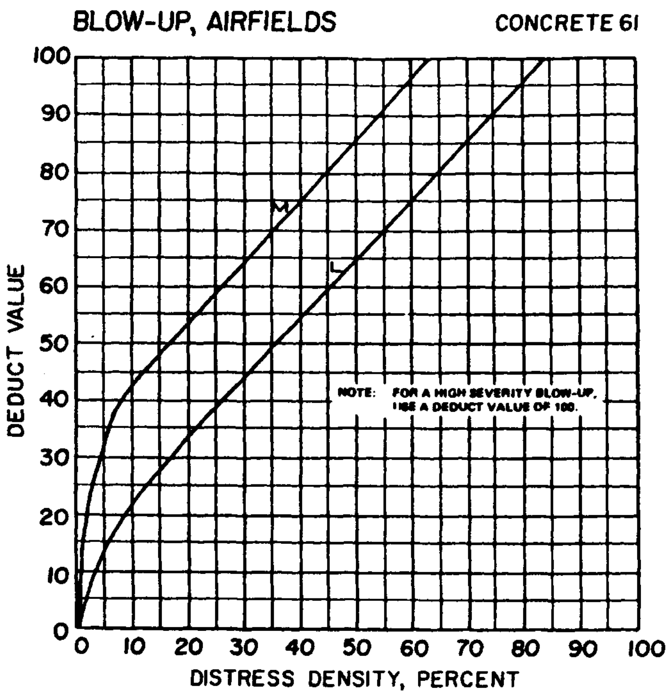


Figure E-16. Rigid Pavement Values, Distress 61, Blow-up.

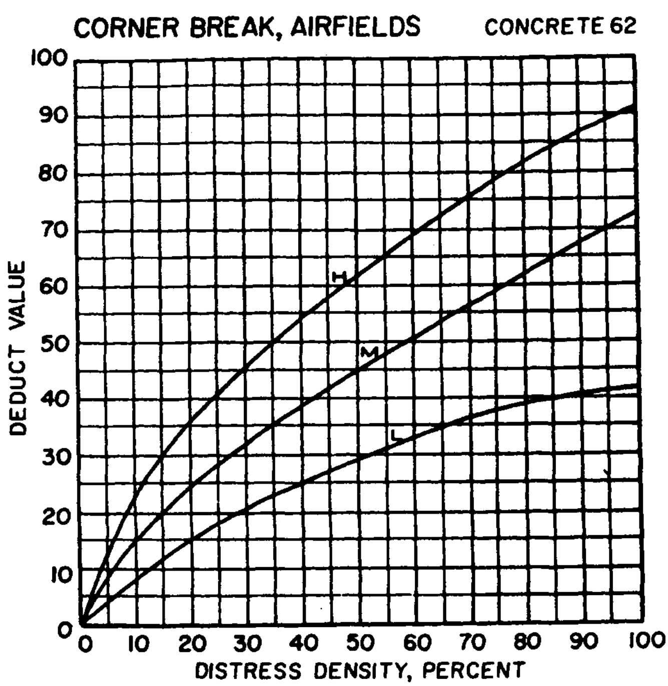


Figure E-17. Rigid Pavement Deduct Values, Distress 62, Corner Break.

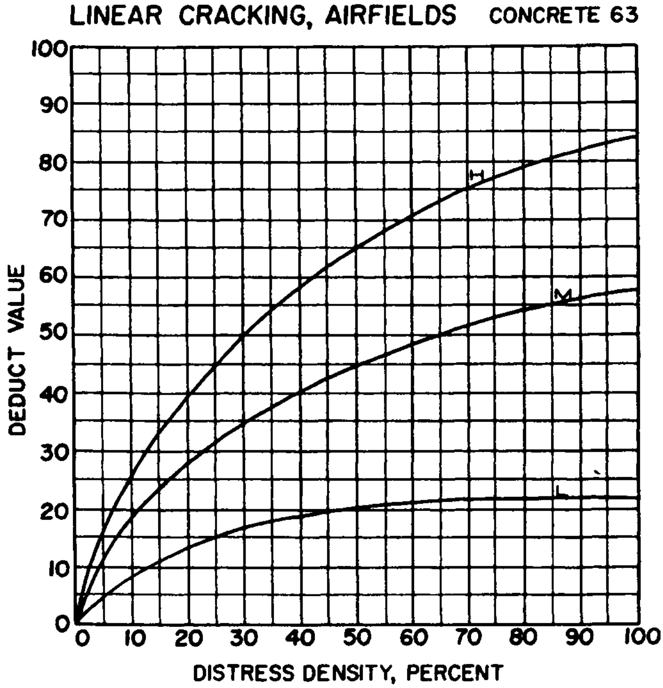


Figure E-18. Rigid Deduct Values, Distress 63, Longitudinal/Transverse/Diagonal Cracking.

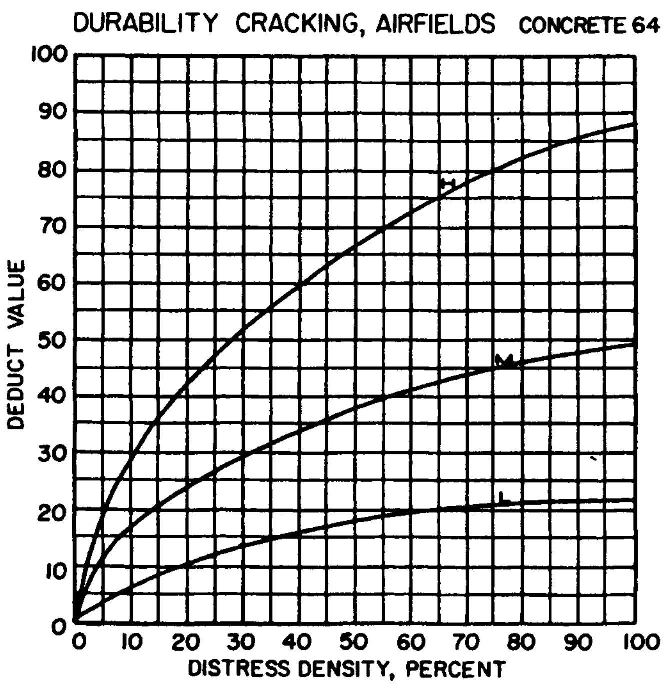


Figure E-19. Rigid Pavement Deduct Values, Distress 64, Durability Cracking.

JOINT SEAL DAMAGE CONCRETE 65

Joint seal damage is not rated by density. The severity of the distress is determined by the sealant's overall condition for a particular section.

The deduct values for the three levels of severity are as follows:

1. High Severity - 12 Points
2. Medium Severity - 7 Points
3. Low Severity - 2 Points

Figure E-20. Rigid Pavement Deduct Values, Distress 65, Joint Seal Damage.

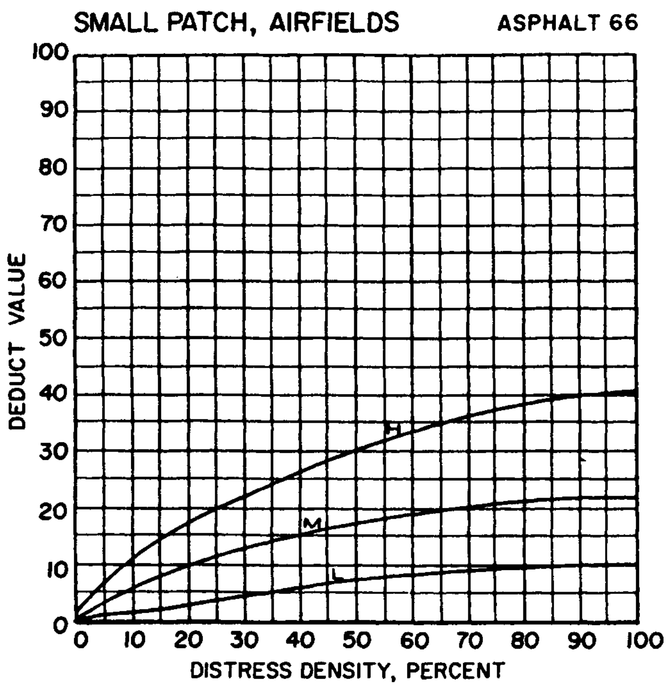


Figure E-21. Rigid Pavement Deduct Values, Distress 66, Small Patch.

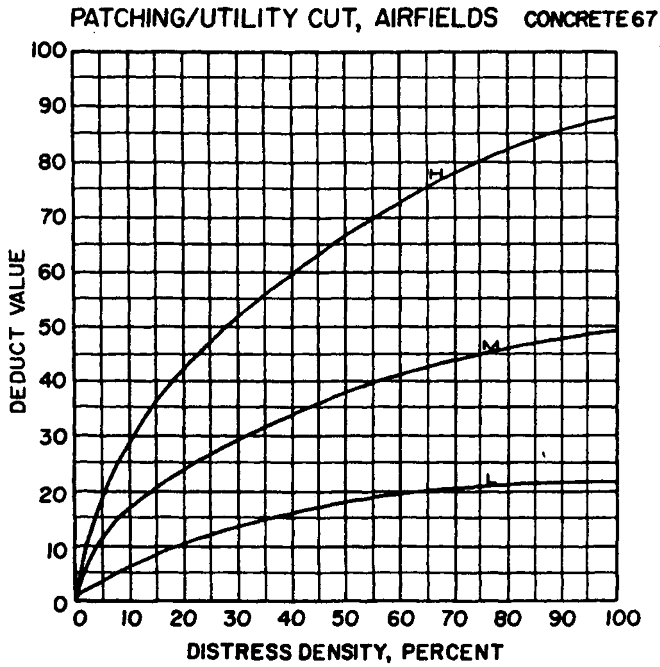


Figure E-22. Rigid Pavement Deduct Values, Distress 67, Patching/Utility Cut Defect.

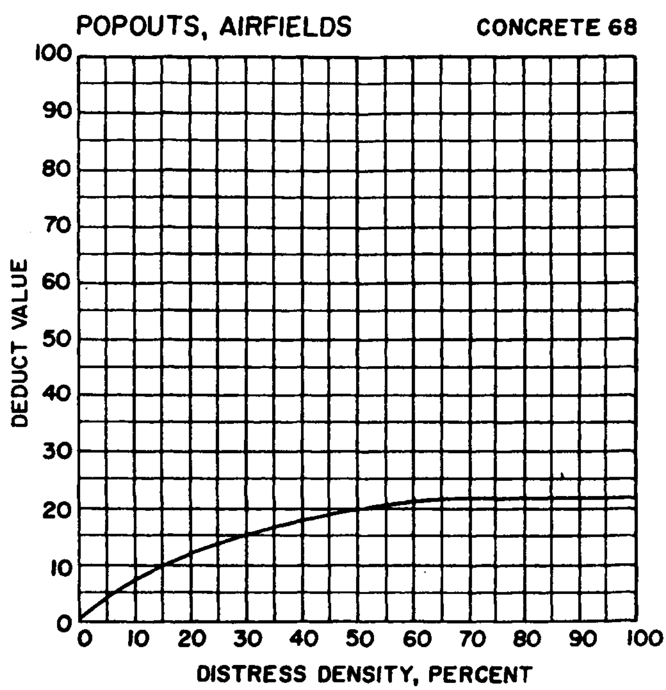


Figure E-23. Rigid Pavement Deduct Values, Distress 68, Popouts.

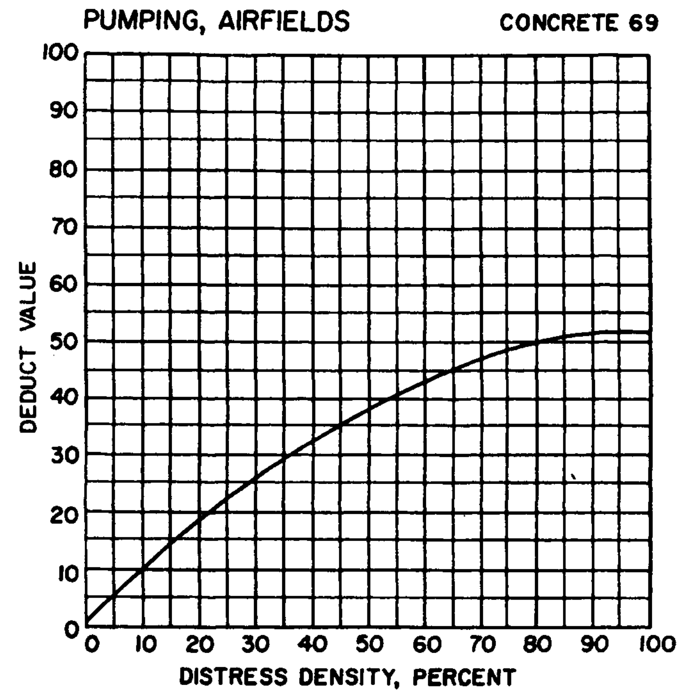


Figure E-24. Rigid Pavement Deduct Values, Distress 69. Pumping.

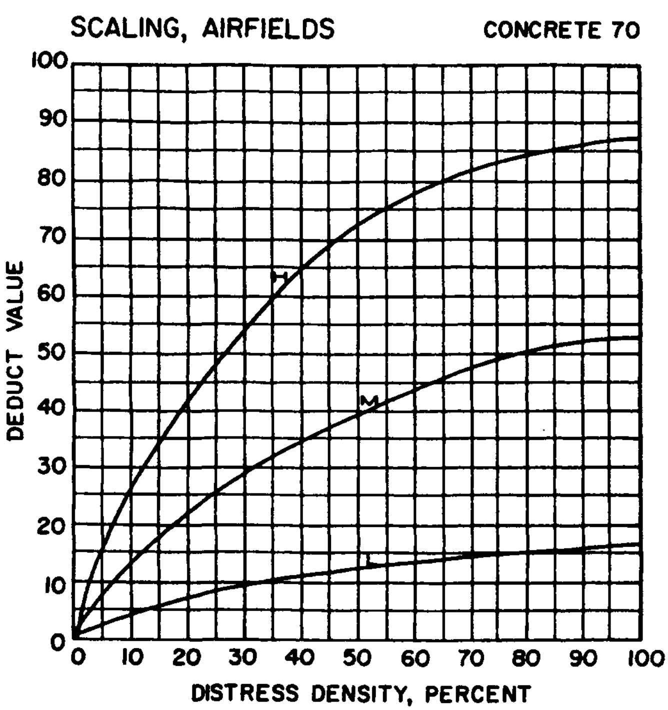


Figure E-25. Rigid Pavement Deduct Values, Distress 70, Scaling.

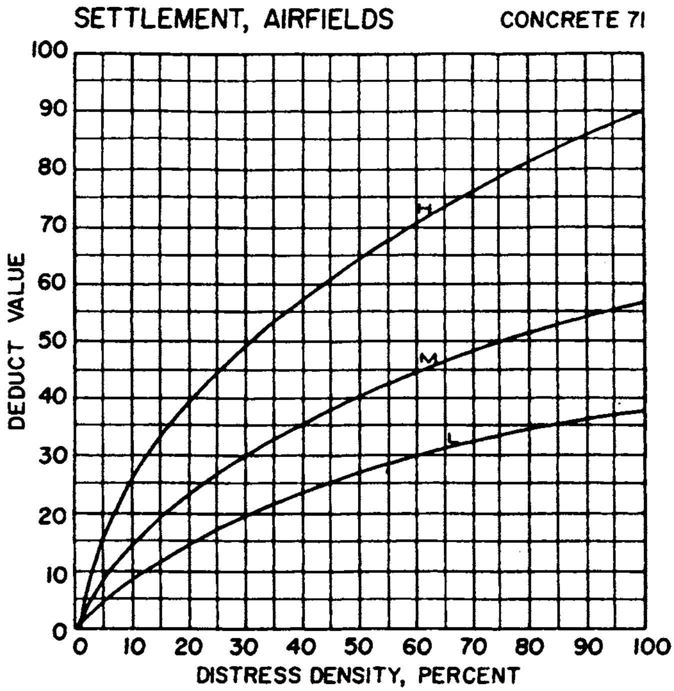


Figure E-26. Rigid Pavement Deduct Values. Distress 71, Settlement.

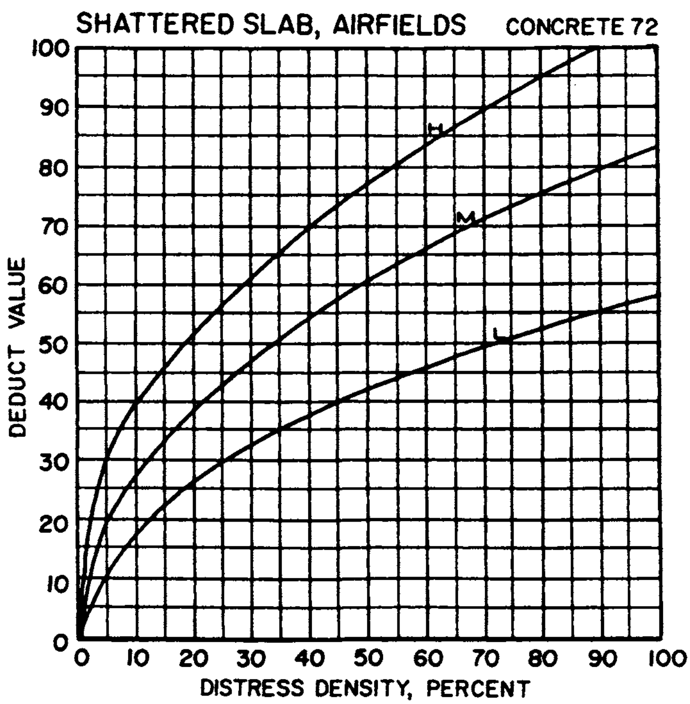


Figure E-27. Rigid Pavement Deduct Values, Distress 72, Shattered Slab.

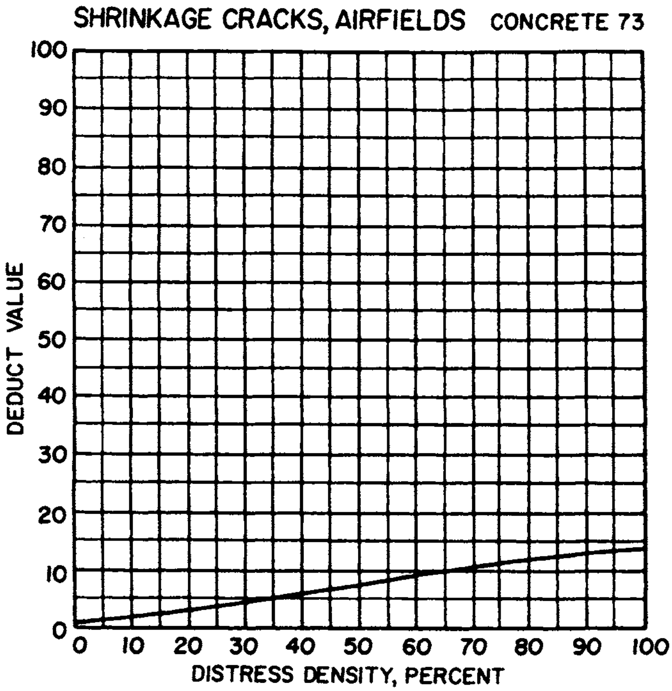


Figure E-28. Rigid Pavement Deduct Values, Distress 73, Shrinkage Cracks.

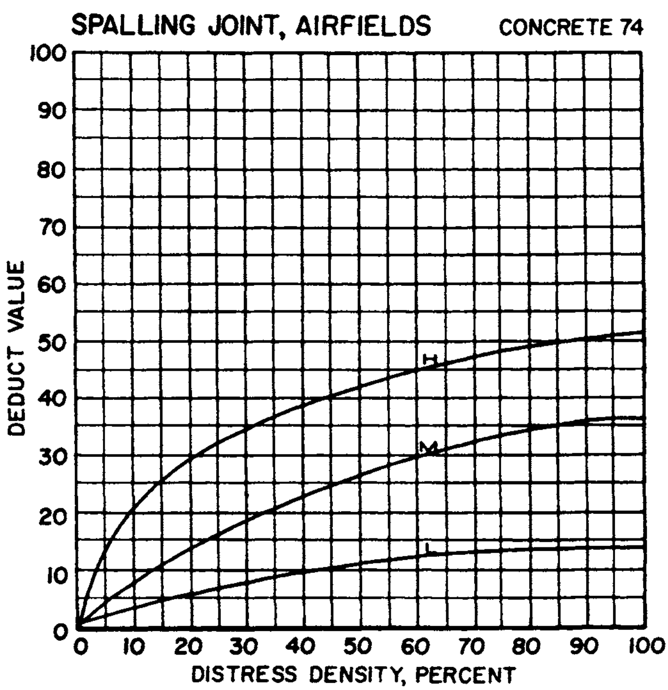


Figure E-29. Rigid Pavement Deduct Values, Distress 74, Spalling Along the Joints.

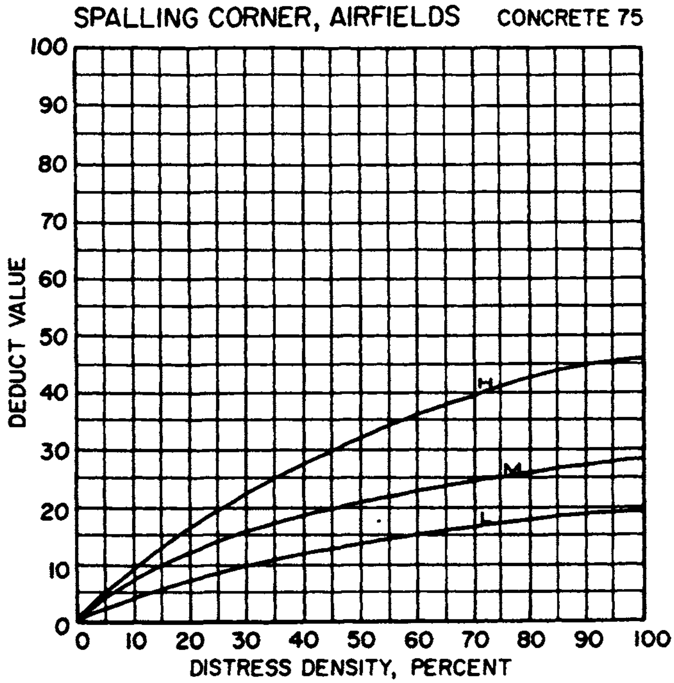


Figure E-30. Rigid Pavement Deduct Values. Distress 75. Spalling Corner.

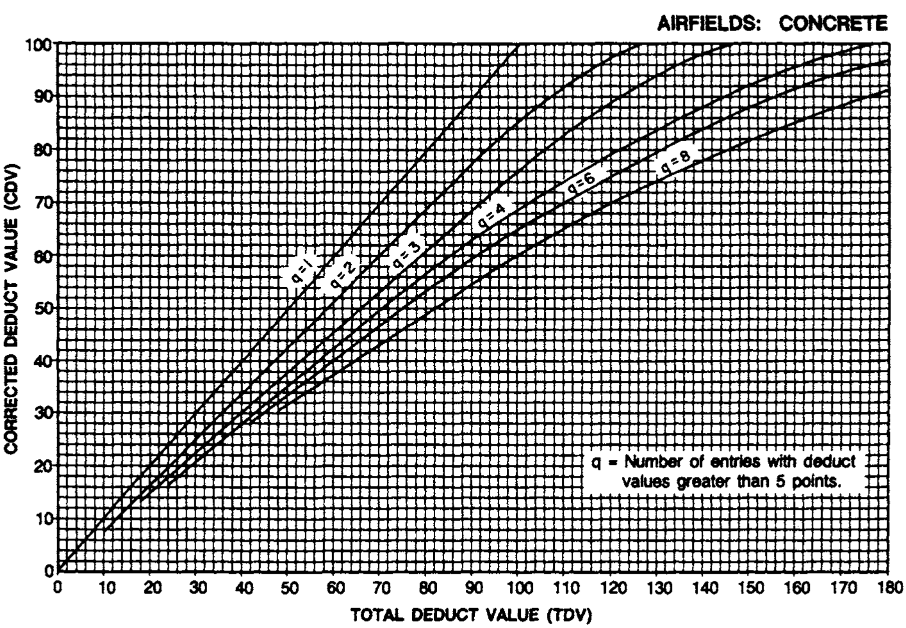


Figure E-31. Corrected Deduct Values for Jointed Concrete Pavements.

APPENDIX F

Unsurfaced Roads: Distress Definitions And Deduct Value Curves

Improper Cross Section

Description

An unsurfaced road should have a crown with enough slope from the centerline to the shoulder to drain all water from the road's surface. No crown is used on curves because they are usually banked. The cross section is improper when the road surface is not shaped or maintained to carry water to the ditches.

Severity Levels (Figure F–1)

L—Small amounts of ponding water or evidence of ponding water on the road surface; or the road surface is completely flat (no cross slope).

M—Moderate amounts of ponding water or evidence of ponding water on the road surface; or the road surface is bowl shaped.

H—Large amounts of ponding water or evidence of ponding water on the road surface; or the road surface contains severe depressions.

How to Measure

Improper cross section is measured in linear feet per sample unit (along the centerline or parallel to the centerline). The cross section runs from the outside shoulder break on one side of the road to the outside shoulder break on the other side. Different severity levels may exist within the sample unit. For example, there could be 60 ft with medium severity and 40 ft with low severity. The maximum length would be equal to the length of the sample unit.

IMPROPER CROSS SECTION

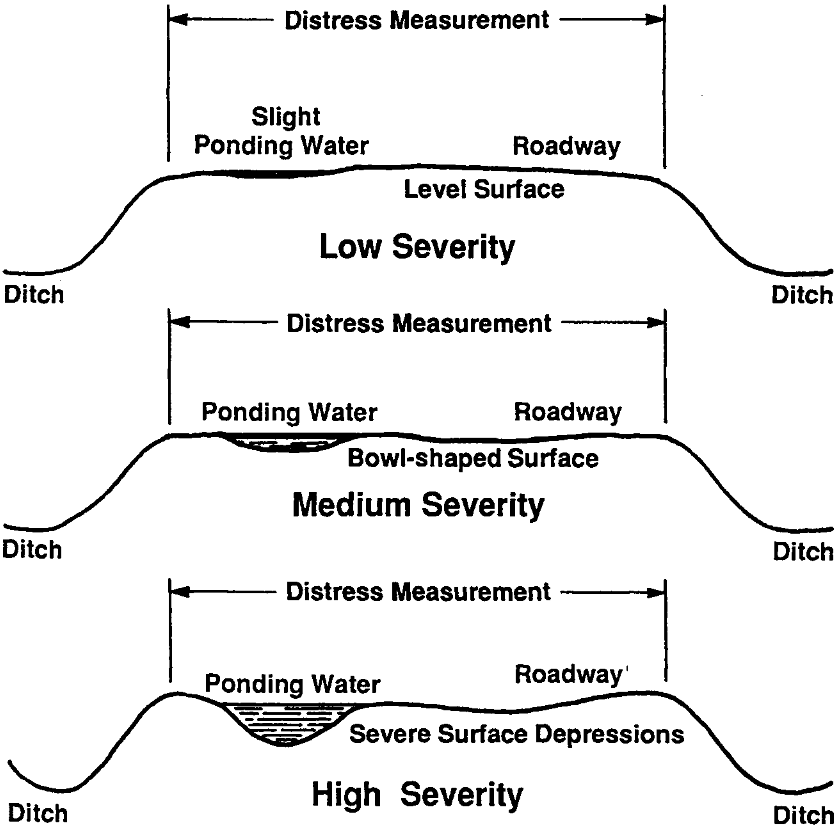


Figure F-1. Improper Cross Section.

Inadequate Roadside Drainage

Description

Poor drainage causes water to pond. Drainage becomes a problem when ditches and culverts are not in good enough condition to direct and carry runoff water because of improper shape or maintenance.

Severity Levels (Figure F–2)

L—Small amounts of ponding water or evidence of ponding water in the ditches; or overgrowth or debris in the ditches.

M—Moderate amounts of ponding water or evidence of ponding water in the ditches; or overgrowth or debris in the ditches; or erosion of the ditches into the shoulders or roadway.

H—Large amounts of ponding water or evidence of ponding water in the ditches; or water running across or down the road; or overgrowth or debris in the ditches; or erosion of the ditches into the shoulders or roadway.

How to Measure

Drainage problems are measured in linear feet per section parallel to the centerline. The maximum length is two times the length of the sample unit (two ditches for the total length of the sample unit). For example, a sample unit may have 120 ft with low severity and 35 ft with high severity.

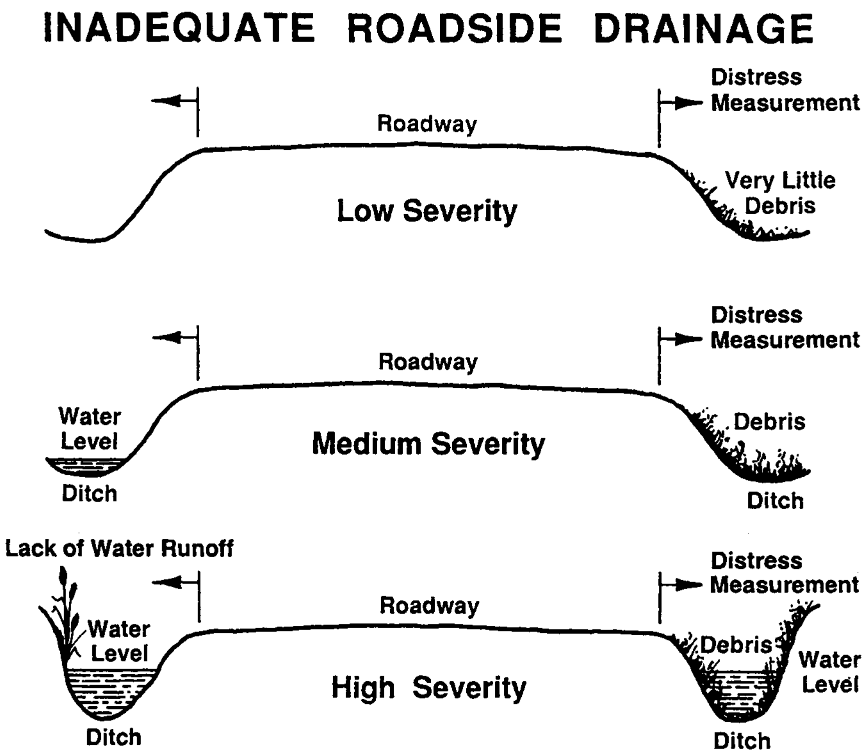


Figure F-2. Inadequate Roadside Drainage.

Corrugations

Description

Corrugations (also known as washboarding) are closely spaced ridges and valleys (ripples) at fairly regular intervals. The ridges are perpendicular to the traffic direction. This type of distress is usually caused by traffic and loose aggregate. These ridges usually form on hills, on curves, in areas of acceleration or deceleration, or in areas where the road is soft or potholed.

Severity Levels (Figure F–3)

L—Corrugations are <1 in. deep.

M—Corrugations are between 1 and 3 in. deep.

H—Corrugations are deeper than 3 in.

How to Measure

Corrugations are measured in square feet of surface area per sample unit. The amount cannot exceed the total area of the sample unit. For example, a sample unit may have 230 sq ft with moderate severity and 50 sq ft with high severity.

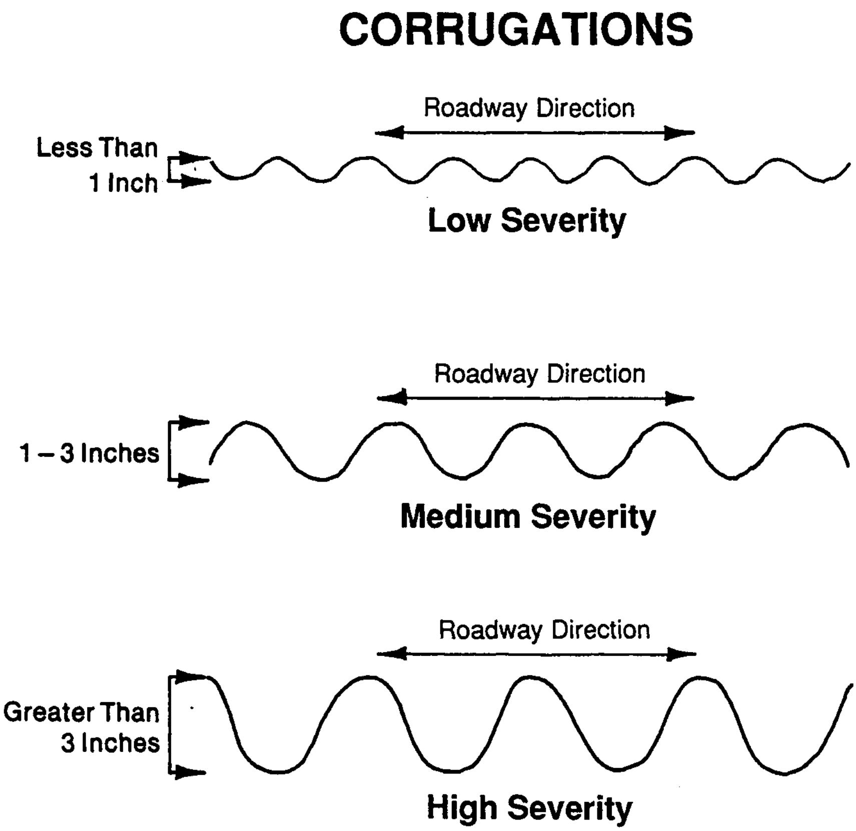


Figure F-3. Corrugations.

Dust

Description

The wear and tear of traffic on unsurfaced roads will eventually loosen the larger pa ticles from the soil binder. As traffic passes, dust clouds create a danger to trailing passing vehicles and cause significant environmental problems.

Severity Levels (Figure F–4)

L—Normal traffic produces a thin dust that does not obstruct visibility.

M—Normal traffic produces a moderately thick cloud that partially obstructs visibilit and causes traffic to slow down.

H—Normal traffic produces a very thick cloud that severely obstructs visibility an causes traffic to slow down significantly or stop.

How to Measure

Drive a vehicle at 25 mph and watch the dust cloud. Dust is measured as low-, mediun or high-severity for the sample unit.

DUST

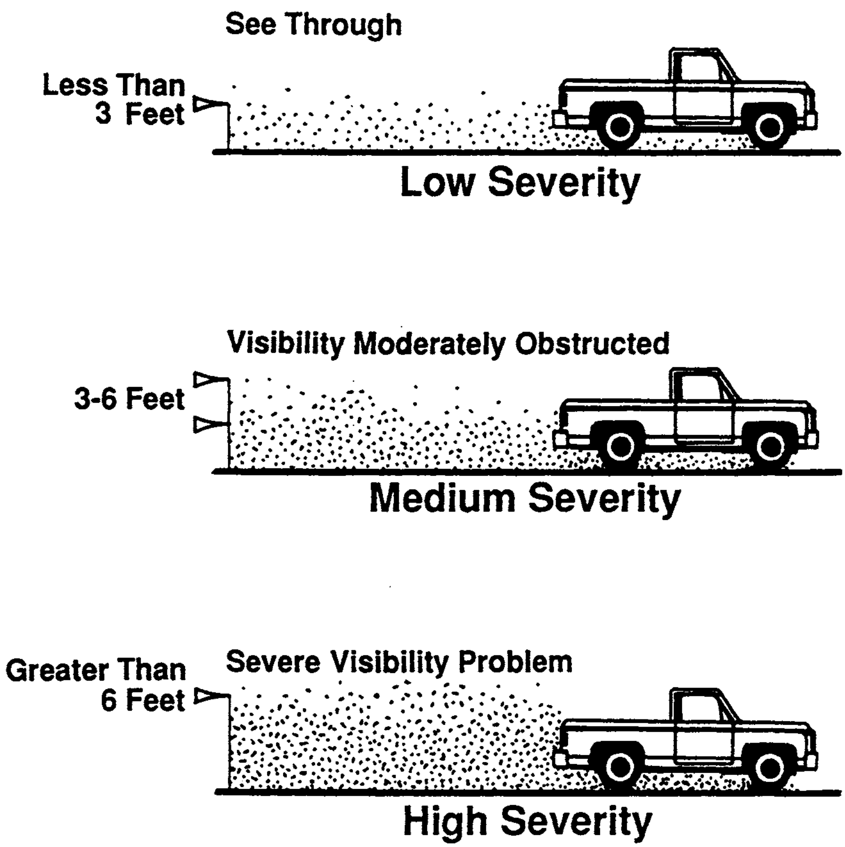


Figure F-4. Dust.

Potholes

Description

Potholes are bowl-shaped depressions in the road surface. They are usually <3 ft in diameter. Potholes are produced when traffic wears away small pieces of the road surface. They grow faster when water collects inside the hole. The road then continues to disintegrate because of loosening surface material or weak spots in the underlying soils.

Severity Levels (Figure F–5)

The levels of severity for potholes are based on both the diameter and the depth of the pothole according to the following table:

	Average Diameter			
Maximum Depth	Less than 1 ft	1 to 2 ft	2 to 3 ft	More than 3 ft[a]
1/2 to 2 inches	L	L	M	M
2 to 4 inches	L	M	H	H
More than 4 inches	M	H	H	H

[a]If the pothole is over 3 ft in diameter, the area should be determined in square feet and divided by 7 to find the equivalent number of potholes.

How to Measure

Potholes are measured by counting the number that are low-, medium- and high-severity in a sample unit and recording them separately by severity level. For example, there may be 14 potholes of medium severity and 8 potholes of low severity.

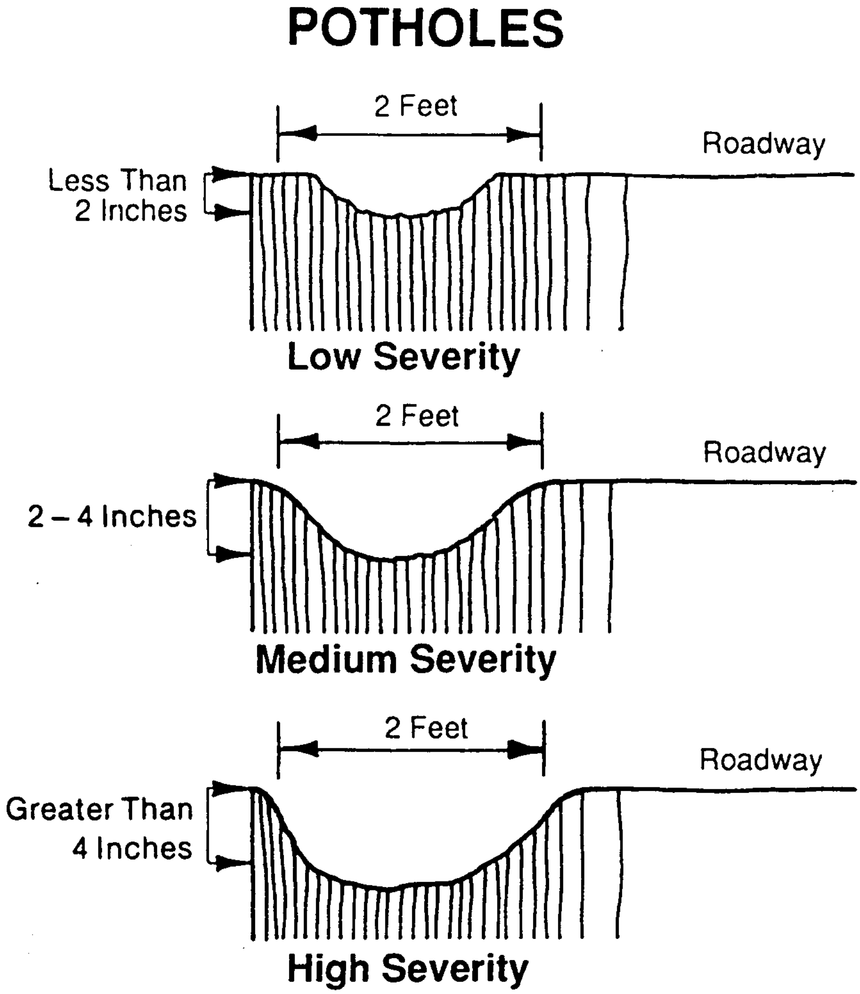


Figure F-5. Potholes.

Ruts

Description

A rut is a surface depression in the wheel path that is parallel to the road centerline. Ruts are caused by a permanent deformation in any of the road layers or subgrade. They result from repeated vehicle passes, especially when the road is soft. Significant rutting can destroy a road.

Severity Levels (Figure F–6)

L—Ruts are <1 in. deep.

M—Ruts are between 1 and 3 in. deep.

H—Ruts are deeper than 3 in.

How to Measure

Ruts are measured in square feet of surface area per sample unit. For example, a sample unit may have 75 sq ft with high severity and 240 sq ft with medium severity.

RUTS

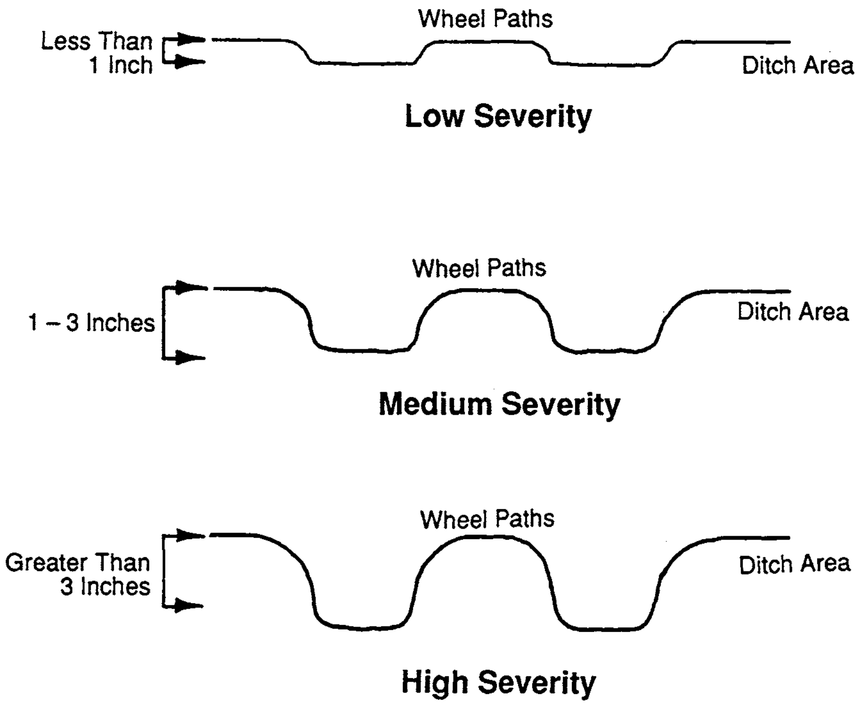


Figure F-6. Ruts.

Loose Aggregate

Description

The wear and tear of traffic on unsurfaced roads will eventually loosen the larger aggregate particles from the soil binder. This leads to loose aggregate particles on the road surface or shoulder. Traffic moves loose aggregate particles away from the normal road wheel path and forms a berm in the center or along the shoulder (the less-traveled areas).

Severity Levels (Figure F–7)

L—Loose aggregate on the road surface, or a berm of aggregate (<2 in. deep) on the shoulder or less-traveled area.

M—Moderate aggregate berm (between 2 to 4 in. deep) on the shoulder or less-traveled area. A large amount of fine soil particles is usually found on the roadway surface.

H—Large aggregate berm (>4 in. deep) on the shoulder or less-traveled area.

How to Measure

Loose aggregate is measured in linear feet parallel to the centerline in a sample unit. Each berm is measured separately. For example, if a sample unit that is 100 ft long has three berms of medium-severity loose aggregate—one on each side and one down the middle—then the measurement would be 300 ft at medium severity.

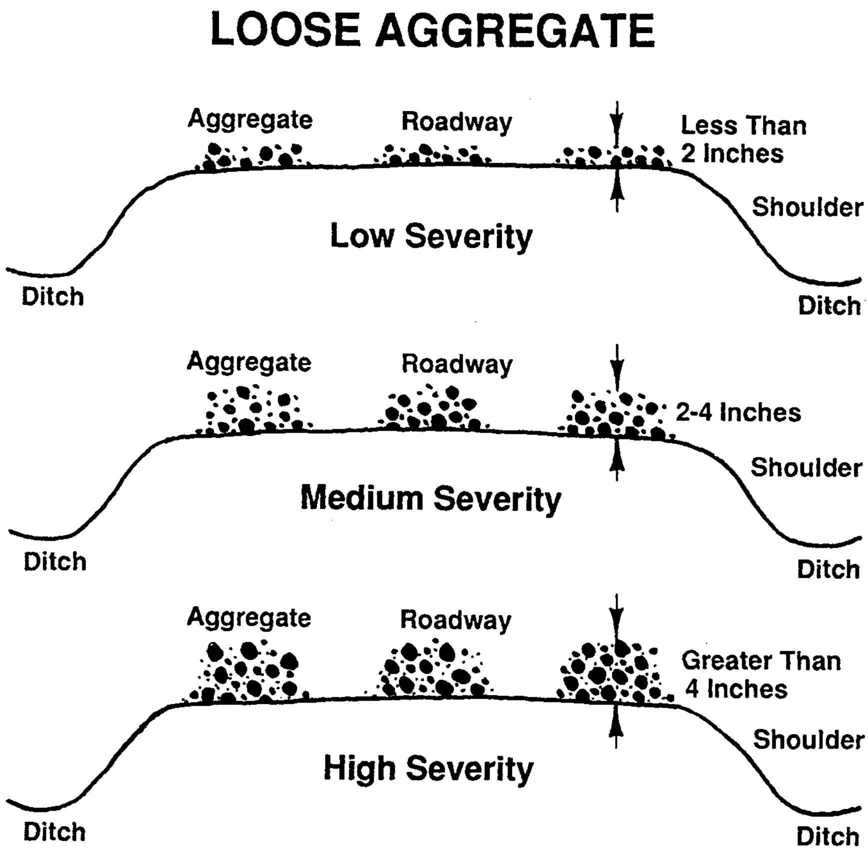


Figure F-7. Loose Gravel.

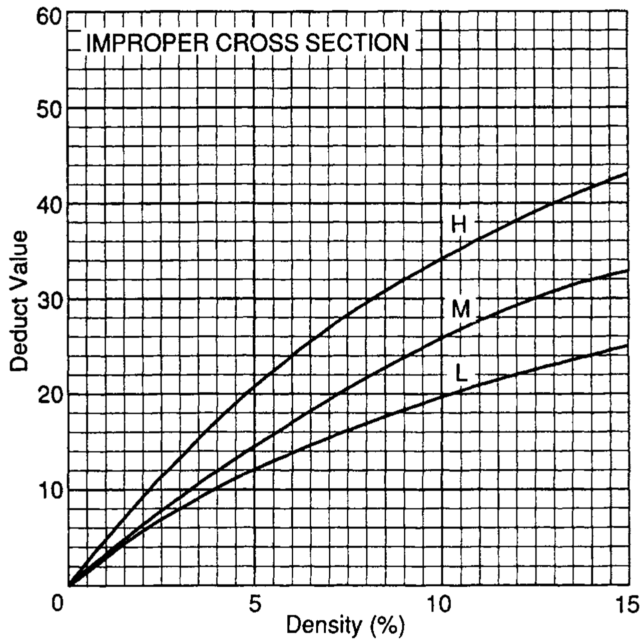


Figure F-8. Improper Cross Section.

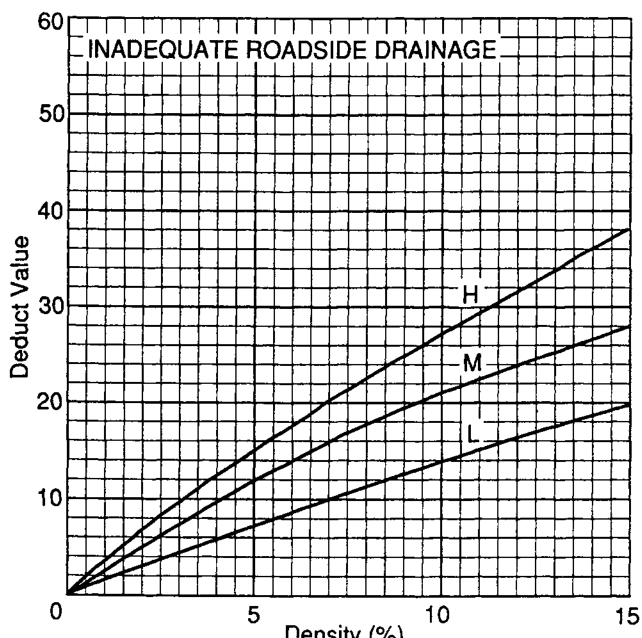


Figure F-9. Inadequate Roadside Drainage.

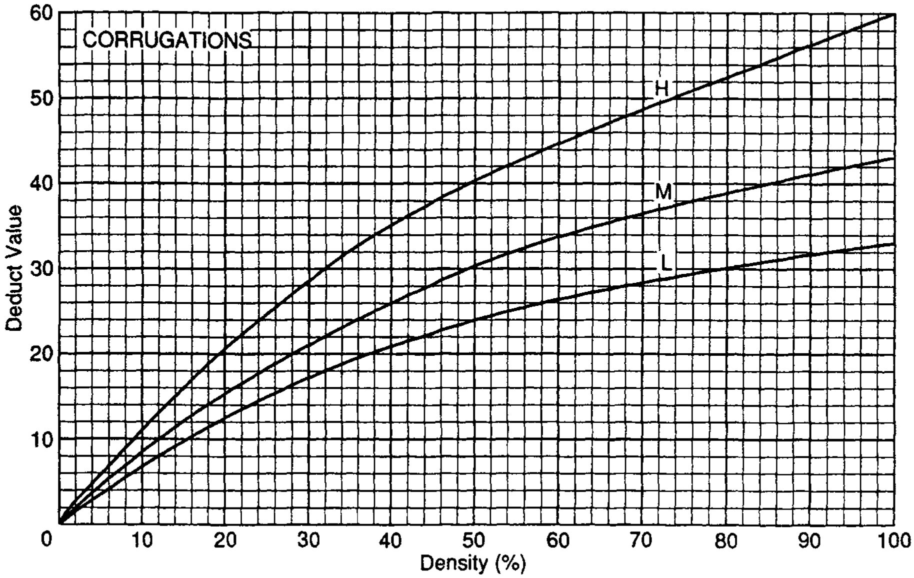


Figure F-10. Corrugations.

Dust

Dust is not rated by density. The deduct values for the levels of severity are:

LOW	2 points
MEDIUM	5 points
HIGH	15 points

Figure F-11. Dust.

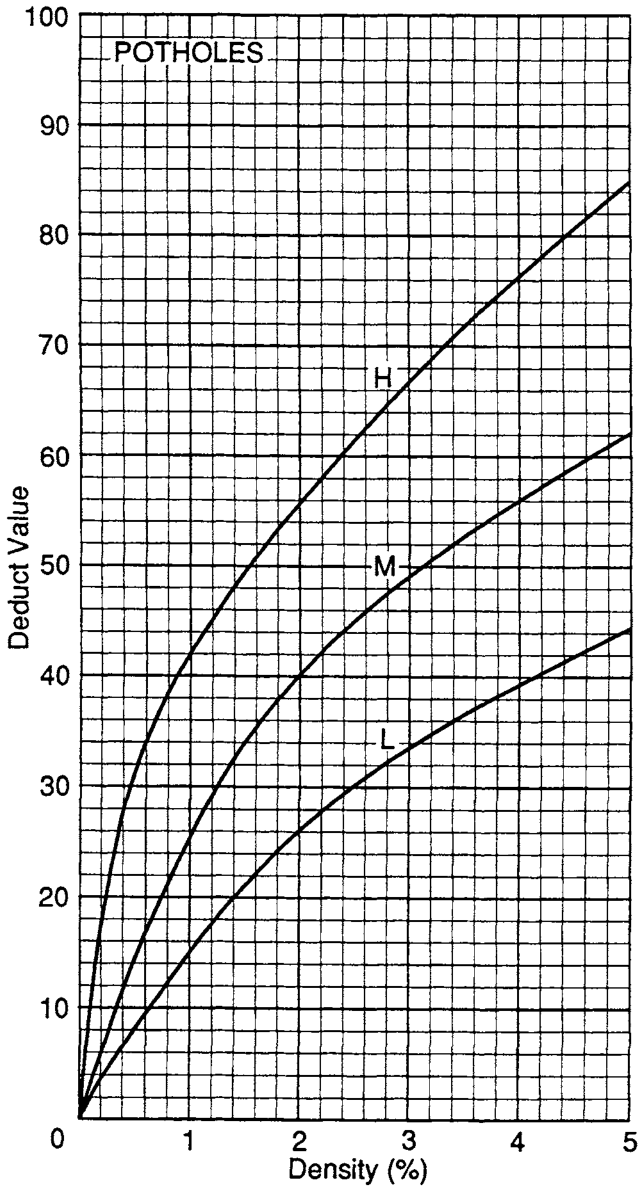


Figure F-12. Potholes.

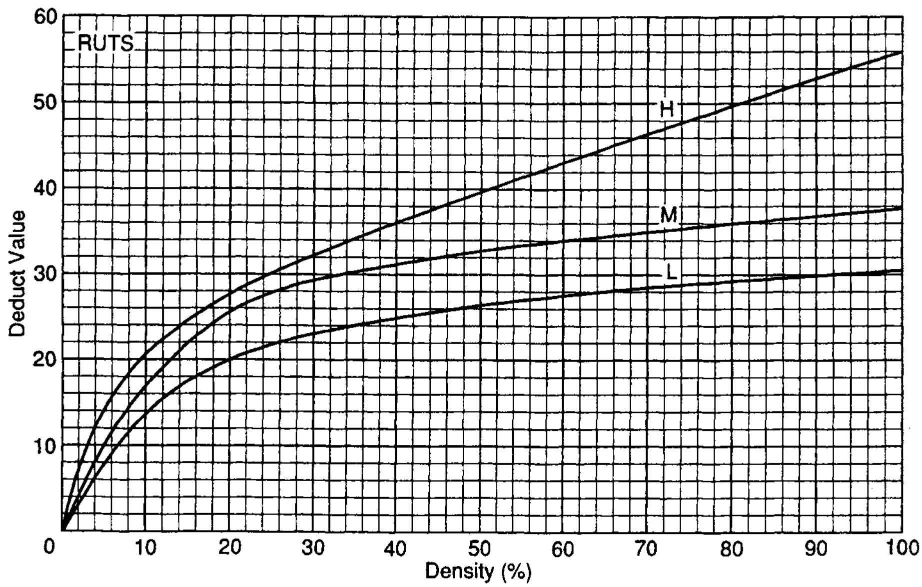


Figure F-13. Ruts.

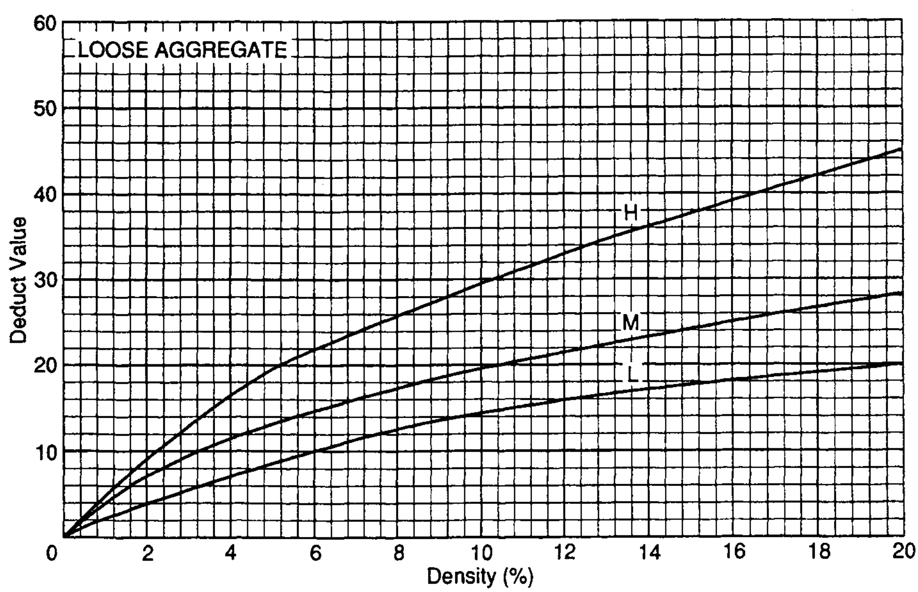


Figure F-14. Loose Aggregate.

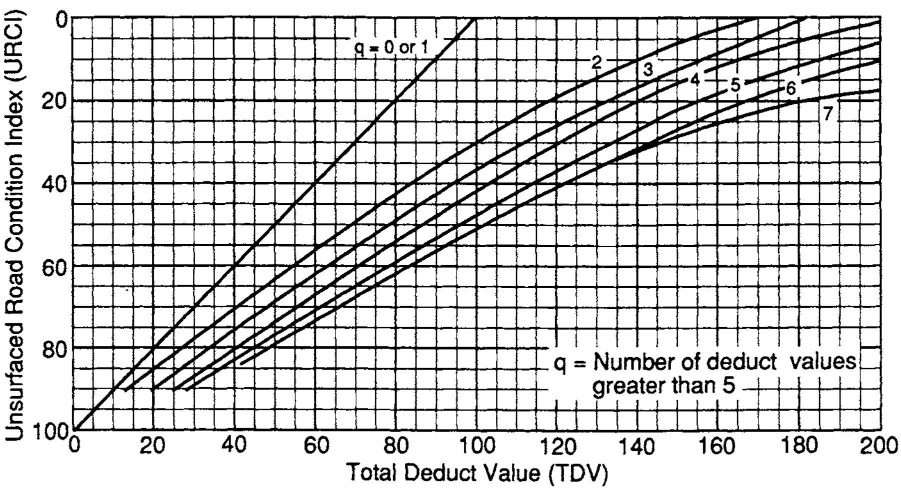

Unsurfaced Road Condition Index (URCI)
0
20
40
60
80
100
q = 0 or 1
2
3
4
5
6
7
q = Number of deduct values greater than 5
0
20
40
60
80
100
120
140
160
180
200
Total Deduct Value (TDV)

Figure F-15.

APPENDIX G

Computing Work Quantity from Distress Quantity

1. For slab replacement, work quantity = slab area

2. For all other; we try to look up the conversion by distress type, severity, and work unit type (Figure G–1). If a matching record is found, we take the amount given by the conversion type column (e.g., for distress 21, use slab width) and multiply it by the distress quantity and the value in the multiplier column to get the work quantity. The conversion type definitions are:
   a. *Slab Width*: Work quantity = distress quantity x slab width x multiplier
   b. *Slab Length:* Work quantity = distress quantity x slab length x multiplier
   c. *Slab Area*: Work quantity = (slab width x slab length) x distress quantity x multiplier
   d. *Constant*: Work quantity = distress quantity x multiplier
   e. *Joint Calculation*: Work quantity = Joint Length x (distress quantity/ number of slabs) x multiplier
   f. *Distress Area*: Work quantity = distress area x multiplier
   g. *Slab Length + Width*: Work quantity = (slab width + slab length) x distress quantity x multiplier
   h. *Patch Area*: Work quantity = distress quantity + (0.6096 x SquareRoot(Distress quantity/Multiplier) x (multiplier + 1)) + 0.3716

Please note that all equations and definitions in this appendix are based on using metric units. Therefore, the computed work quantity is either in linear meter or square meter.

Figure G-1. Work Conversion Table.

Distress Code	Description	Severity	Work Unit Type	Conversion Type	Multiplier
1	ALLIGATOR CRACKING	Any	Any	Patch Area	1.25
2	BLEEDING	Any	Any	Distress Quantity	1
3	BLOCK CRACKING	Any	Any	Distress Quantity	1
4	BUMPS/SAGS	Any	Any	Distress Quantity	1
5	CORRUGATION	Any	Any	Patch Area	1.25
6	DEPRESSION	Any	Any	Patch Area	1.25
7	EDGE CRACKING	Any	Area	Distress Quantity	0.5
7	EDGE CRACKING	Any	Linear	Distress Quantity	1
8	JOINT REFLECTION CRACKING	Any	Area	Distress Quantity	0.5
8	JOINT REFLECTION CRACKING	Any	Linear	Distress Quantity	1
9	LANE/SHOULDER DROP	Any	Any	Distress Quantity	1
10	LONGITUDINAL/TRANSVERSE CRACKING			Distress Quantity	1
11	PATCH/UTILITY CUT	Any	Any	Patch Area	1.25
12	POLISHED AGGREGATE	Any	Any	Distress Quantity	1
13	POTHOLE	H	Any	Constant	0.83613
13	POTHOLE	M	Any	Constant	0.55742
13	POTHOLE	L	Any	Constant	0.27871
14	RAILROAD CROSSING	Any	Any	Distress Quantity	1
15	RUTTING	Any	Any	Distress Quantity	1
16	SHOVING	Any	Any	Patch Area	1.25
17	SLIPPAGE CRACKING	Any	Any	Patch Area	1.25
18	SWELL	Any	Any	Patch Area	1.25
19	WEATHERING/RAVELING	Any	Any	Distress Quantity	1
21	BLOW-UP/SHATTER	H	Linear	Slab Width	1
21	BLOW-UP/SHATTER	H	Area	Slab Width	2
21	BLOW-UP/SHATTER	L	Linear	Slab Width	1
21	BLOW-UP/SHATTER	L	Area	Slab Width	1
21	BLOW-UP/SHATTER	M	Linear	Slab Width	1
21	BLOW-UP/SHATTER	M	Area	Slab Width	1.5
22	CORNER BREAK	L	Area	Constant	3
22	CORNER BREAK	H	Area	Constant	3
22	CORNER BREAK	L	Linear	Constant	2.5
22	CORNER BREAK	H	Linear	Constant	2.5
22	CORNER BREAK	M	Area	Constant	3
22	CORNER BREAK	M	Linear	Constant	2.5
23	DIVIDED SLAB	Any	Linear	Width	1
23	DIVIDED SLAB	Any	Area	Slab Area	1
24	DURABILITY CRACKING	L	Linear	Slab Width	1
24	DURABILITY CRACKING	H	Area	Slab Area	1
24	DURABILITY CRACKING	L	Area	Slab Width	1
24	DURABILITY CRACKING	M	Linear	Slab Width	1
24	DURABILITY CRACKING	H	Linear	Width	1
24	DURABILITY CRACKING	M	Area	Slab Width	1.25
25	FAULTING	Any	Any	Slab Width	1
26	JOINT SEAL DAMAGE	Any	Area	Joint Calculation	0.3048
26	JOINT SEAL DAMAGE	Any	Linear	Joint Calculation	1
27	LANE/SHOULDER DROP	Any	Area	SlabLength	1
27	LANE/SHOULDER DROP	Any	Linear	Slab Length	1
28	LINEAR CRACKING	Any	Linear	Width	0.5
28	LINEAR CRACKING	Any	Area	Slab Area	1
29	LARGE PATCH/UTILITY CUT	Any	Area	Slab Width	1.5
29	LARGE PATCH/UTILITY CUT	Any	Linear	Slab Width	1
30	SMALL PATCH	Any	Linear	Constant	0.5
30	SMALL PATCH	Any	Area	Constant	0.25
31	POLISHED AGGREGATE	Any	Linear	Slab Length	1
31	POLISHED AGGREGATE	Any	Area	Slab Area	1
32	POPOUTS	Any	Linear	Slab Length	1
32	POPOUTS	Any	Area	Slab Area	1
33	PUMPING	Any	Linear	Slab Width	1
33	PUMPING	Any	Area	Slab Width	0.3048
34	PUNCHOUT	Any	Area	Slab Width	1.5

Distress Code	Description	Severity	Work Unit Type	Conversion Type	Multiplier
34	PUNCHOUT	Any	Linear	Slab Width	1
35	RAILROAD CROSSING	Any	Linear	Slab Width	1
35	RAILROAD CROSSING	Any	Area	Slab Width	1.5
36	SCALING/CRAZING	M	Area	Slab Width	1.25
36	SCALING/CRAZING	M	Linear	Slab Width	1
36	SCALING/CRAZING	L	Linear	Slab Width	1
36	SCALING/CRAZING	L	Area	Slab Width	1.25
36	SCALING/CRAZING	H	Linear	Width	1
36	SCALING/CRAZING	H	Area	Slab Area	1.25
37	SHRINKAGE CRACKING	Any	Linear	Constant	1.5
37	SHRINKAGE CRACKING	Any	Area	Constant	0.305
38	CORNER SPALLING	Any	Area	Constant	0.25
38	CORNER SPALLING	Any	Linear	Constant	0.5
39	JOINT SPALLING	L	Linear	Constant	0.5
39	JOINT SPALLING	M	Area	Slab Width	0.5
39	JOINT SPALLING	H	Area	Slab Width	0.5
39	JOINT SPALLING	M	Linear	Slab Width	0.5
39	JOINT SPALLING	L	Area	Slab Width	0.25
39	JOINT SPALLING	H	Linear	Slab Width	1
41	ALLIGATOR CRACKING	Any	Any	Patch Area	1.25
42	BLEEDING	Any	Any	Distress Quantity	1
43	BLOCK CRACKING	Any	Any	Distress Quantity	1
44	CORRUGATION	Any	Any	Patch Area	1.25
45	DEPRESSION	Any	Any	Patch Area	1.25
46	JET BLAST	Any	Any	Distress Quantity	1
47	JOINT REFLECTION CRACKING	Any	Linear	Distress Quantity	1
47	JOINT REFLECTION CRACKING	Any	Area	Distress Quantity	0.5
48	LONGITUDINAL/TRANSVERSE CRACKING	Any	Any	Distress Quantity	1
49	OIL SPILLAGE	Any	Any	Patch Area	1.25
50	PATCHING	Any	Any	Patch Area	1.25
51	POLISHED AGGREGATE	Any	Any	Distress Quantity	1
52	WEATHERING/RAVELING	Any	Any	Distress Quantity	1
53	RUTTING	Any	Any	Distress Quantity	1
54	SHOVING	Any	Any	Patch Area	1.25
55	SLIPPAGE CRACKING	Any	Any	Patch Area	1.25
56	SWELLING	Any	Any	Patch Area	1.25
61	BLOW-UP/SHATTER	L	Linear	Slab Width	1
61	BLOW-UP/SHATTER	H	Area	Slab Width	2
61	BLOW-UP/SHATTER	M	Area	Slab Width	1.5
61	BLOW-UP/SHATTER	H	Linear	Slab Width	1
61	BLOW-UP/SHATTER	M	Linear	Slab Width	1
61	BLOW-UP/SHATTER	L	Area	Slab Width	1
62	CORNER BREAK	M	Linear	Constant	2.5
62	CORNER BREAK	H	Area	Constant	3
62	CORNER BREAK	M	Area	Constant	3
62	CORNER BREAK	L	Area	Constant	3
62	CORNER BREAK	L	Linear	Constant	2.5
62	CORNER BREAK	H	Linear	Constant	2.5
63	LINEAR CRACKING	Any	Any	Width	0.5
63	LINEAR CRACKING	Any	Area	Slab Area	1
64	DURABILITY CRACKING	H	Linear	Width	1
64	DURABILITY CRACKING	M	Area	Slab Width	1.25
64	DURABILITY CRACKING	M	Linear	Slab Width	1
64	DURABILITY CRACKING	L	Area	Slab Width	1
64	DURABILITY CRACKING	L	Linear	Slab Width	1
64	DURABILITY CRACKING	H	Area	Slab Area	1
65	JOINT SEAL DAMAGE	Any	Area	Joint Calculation	0.3048
65	JOINT SEAL DAMAGE	Any	Linear	Joint Calculation	1
66	SMALL PATCH	Any	Linear	Constant	0.5
66	SMALL PATCH	Any	Area	Constant	0.25
67	LARGE PATCH/UTILITY CUT	Any	Linear	Slab Width	1
67	LARGE PATCH/UTILITY CUT	Any	Area	Slab Width	1.5
68	POPOUTS	Any	Area	Slab Area	1

Distress Code	Description	Severity	Work Unit Type	Conversion Type	Multiplier
68	POPOUTS	Any	Linear	Slab Length	1
69	PUMPING	Any	Linear	Slab Width	1
69	PUMPING	Any	Area	Slab Width	0.3048
70	SCALING/CRAZING	L	Area	Slab Width	1.25
70	SCALING/CRAZING	H	Area	Slab Area	1.25
70	SCALING/CRAZING	H	Linear	Width	1
70	SCALING/CRAZING	M	Linear	Slab Width	1
70	SCALING/CRAZING	L	Linear	Slab Width	1
70	SCALING/CRAZING	M	Area	Slab Width	1.25
71	FAULTING	Any	Any	Slab Width	1
72	DIVIDED SLAB	Any	Linear	Width	1
72	DIVIDED SLAB	Any	Area	Slab Area	1
73	SHRINKAGE CRACKING	Any	Area	Constant	0.3048
73	SHRINKAGE CRACKING	Any	Linear	Constant	1.5
74	JOINT SPALLING	L	Linear	Constant	0.5
74	JOINT SPALLING	L	Area	Slab Width	0.25
74	JOINT SPALLING	M	Linear	Slab Width	0.5
74	JOINT SPALLING	M	Area	Slab Width	0.5
74	JOINT SPALLING	H	Linear	Slab Width	1
74	JOINT SPALLING	H	Area	Slab Width	0.5
75	CORNER SPALLING	Any	Linear	Constant	0.5
75	CORNER SPALLING	Any	Area	Constant	0.25

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