GUIDELINES FOR EVALUATION OF ASPHALT-OVERLAID CONCRETE PAVEMENTS

By

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A Report of the Findings of:
Rehabilitation of Asphalt-Overlaid Concrete Pavements

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Guidelines for Evaluation of Asphalt-Overlaid Concrete Pavements

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Study was conducted in cooperation with the U. S. Department of Transportation Federal Highway Administration. Study title: Rehabilitation of Asphalt-Overlaid Concrete Pavements.

This report presents guidelines for selection of asphalt-overlaid concrete (AC/PCC) pavements for rehabilitation, collection of distress data, recognition of AC/PCC pavement distress modes, nondestructive deflection testing, materials sampling, and overall project-level evaluation.

Key condition indicators (distress, rutting, roughness, serviceability, and CRS) were examined to assess their usefulness in AC/PCC rehabilitation project selection. Critical levels for these indicators were identified, and available prediction models were investigated for use in rehabilitation programming.

The project-level evaluation procedure includes guidelines for field and laboratory data collection and analysis for structural evaluation, functional evaluation, drainage evaluation, and AC surface material evaluation. A procedure was developed for backcalculation of AC/PCC pavement layer elastic moduli from deflection measurements, and guidelines were developed for practical interpretation of the backcalculation results.

The recommended evaluation procedure relies on conventional testing and evaluation methods (visual surveying, deflection testing, coring, materials testing, etc.), new technologies (e.g., ground-penetrating radar, infrared thermography) are developing which have potential for use in pavement evaluation. These technologies are also reviewed in this report.
ACKNOWLEDGMENTS

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GUIDELINES FOR EVALUATION OF ASPHALT-OVERLAID CONCRETE PAVEMENTS

1.0 INTRODUCTION

The Illinois Interstate highway system consists of about 1750 two-way miles of heavily trafficked multiple-lane pavements which were constructed largely between 1957 and 1980. About one third of these pavements were originally constructed as 10-in jointed reinforced concrete pavement, and about two thirds were originally constructed as continuously reinforced concrete pavement (CRCP) ranging in thickness from 7 to 10 inches.

As of 1991, about 60 percent of these pavements had been overlaid with asphalt concrete (AC) ranging in thickness from 1.5 to 6 inches. By some estimates nearly all of the Illinois Interstate highway system will be overlaid by the year 2000. Similar trends are seen in other states as well. As the mileage of bare concrete highway pavement decreases and the mileage of asphalt-overlaid concrete (AC/PCC) highway pavement increases, evaluation and rehabilitation of AC/PCC pavements become increasingly prominent and pressing concerns to state highway engineers.

Rehabilitation of concrete pavements is done by resurfacing with asphalt more commonly than by any other method. Yet, to date the performance of AC overlays of PCC pavements has been very inconsistent and in many cases very poor. A large and growing mileage of AC/PCC pavements is deteriorating, some of it rapidly, for reasons which are not fully understood. Despite the fact that AC/PCC pavements make up such a large percentage of the highway mileage of the United States, much less is known about their performance, evaluation, and rehabilitation than is known about other pavement types. The purpose of this study is to investigate AC/PCC pavement performance and behavior in depth and produce practical guidelines for their evaluation and rehabilitation.

This report presents guidelines for project-level evaluation of AC/PCC pavement, for the purposes of rehabilitation programming and design. Rehabilitation programming involves deciding when a project should be rehabilitated, and rehabilitation design involves deciding what should be done. A careful project-level evaluation must be conducted in order to select the rehabilitation type and timing which will achieve the best and most cost-effective performance. Figure 1 illustrates the evaluation activities which are described in this report.

Project selection for the multiyear rehabilitation program: Rehabilitation programming requires identifying pavements which will be in need of rehabilitation within the next five years. Condition indicators such as CRS, roughness, distress, and rut depth may be used to identify pavement sections that will become unacceptable in the next five years. In this study, the various condition indicators available were
### Selection for Multiyear Rehabilitation Program

#### Project-Level Evaluation

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Figure 1. Components of AC/PCC pavement evaluation.
examined and compared to assess their usefulness in identifying AC/PCC pavement projects needing rehabilitation. Critical levels for these condition indicators were also suggested.

One advantage to targeting pavements in advance is that they can be scheduled for nondestructive deflection testing (NDT) and materials testing, so that these data will be available when it is time for a detailed project-level evaluation and rehabilitation design. Otherwise, a project may be on hold for a year or more waiting for NDT and coring, while its condition becomes worse and the cost of its rehabilitation increases.

An important indicator of pavement condition is the Condition Rating Survey or CRS rating, given biannually to every pavement section on the Interstate network. For a given pavement section, its current CRS may be used to predict the number of years of "remaining life" of the section, i.e., the time when CRS will reach a critical level triggering rehabilitation.

Distress survey: The major distress modes in AC/PCC pavements are described in detail in this report. These distress modes are responsible for the specific distress types which should be recorded in a distress survey of an AC/PCC pavement. Although AC/PCC pavements may exhibit many different types of distress, the emphasis here is on the specific distress types and quantities needed for purposes of rehabilitation type selection, rehabilitation timing, and rehabilitation design.

Nondestructive deflection testing: Detailed guidelines are given for nondestructive deflection testing (NDT) of AC/PCC pavements using the Falling Weight Deflectometer (FWD). These guidelines are specifically for highway FWD testing, and address such topics as traffic control, load level, sensor positions, and testing frequency along a project. The primary purpose of the NDT testing procedures described is to obtain information needed for structural evaluation of the AC/PCC pavement system.

Materials sampling and testing: Guidelines are given for obtaining, inspecting, and testing core samples of the AC surface, PCC slab, and (if present) stabilized base of an AC/PCC pavement.

It is possible to select and design rehabilitation alternatives using only design, traffic, and visible distress data. However, it is far preferable, whenever possible, to conduct deflection testing and materials testing to better characterize the pavement's load-carrying capacity and extent of deterioration. This is particularly true for AC/PCC pavement, since the extent of deterioration in the PCC slab is difficult to assess from visual observations of the AC surface.
Backcalculation of AC/PCC pavement layer moduli: An efficient and reliable method is presented for backcalculation of AC/PCC pavement layer moduli from measured deflections. The elastic modulus of the PCC slab and the k-value of the foundation may be backcalculated quickly using the equations presented. This backcalculation is well suited for spreadsheet analysis and is thus far more efficient than any other backcalculation method available for AC/PCC pavements.

This or any other appropriate, reliable, and convenient backcalculation method may be used to analyze AC/PCC pavement deflection data. Whatever method is used, the results must be interpreted properly to complete the structural evaluation. The subject of interpretation of backcalculation results is addressed in this report. Structural evaluation serves a variety of purposes in rehabilitation decision-making, including assessment of extent and severity of load-related deterioration, selection of repair areas, identification of a need for structural improvement, and obtaining inputs needed for rehabilitation design.

**Project-level evaluation:** The four key components of a project-level evaluation of an AC/PCC pavement are:

1. Functional evaluation,
2. Structural evaluation,
3. Drainage evaluation, and
4. AC material evaluation.

The condition, deflection, and materials testing data collected must be examined to determine whether or not the pavement has a deficiency in any of these four key areas. The deficiencies identified will play a major role in selection of appropriate rehabilitation alternatives.

A thorough project-level evaluation should be documented in a report which summarizes the project design, present condition, and anticipated rehabilitation needs. The evaluation report provides planners and designers with the key information which they need to make decisions concerning selection of rehabilitation alternatives, rehabilitation timing, and rehabilitation design.
PART ONE -- NETWORK-LEVEL EVALUATION

2.0 SELECTION OF AC/PCC PAVEMENTS FOR MULTIYEAR REHABILITATION PROGRAM

Selection of individual pavement sections for rehabilitation is done within the framework of management of a State's entire pavement network. Every agency has its own unique process for assessing highway network needs, prioritizing projects, and allocating funds among the projects, and this process usually involves both planners and engineers. Typically, however, this network-level selection process is characterized by consideration of a few key condition indicators related to pavement condition, ride quality, and safety.

The pavement condition data examined in the network-level selection process is much less detailed and less comprehensive than would be required for a project-level evaluation. This type of condition information is therefore inadequate to design rehabilitation alternatives and prepare detailed plans, bid estimates, and specifications. Furthermore, the limitations of the condition data used in network-level project selection may sometimes be responsible for an individual pavement section receiving a rehabilitation treatment which is not truly the "best" alternative, or which is not done at the "best" time to maximize its long-term performance and cost-effectiveness. More often, however, less-than-optimal rehabilitation activities are done primarily because of insufficient funds.

This section describes and examines the condition indicators available for use in selection of AC/PCC pavements for rehabilitation. The condition indicators which show the most promise for AC/PCC rehabilitation project selection are identified.

Current IDOT Rehabilitation Programming Process

The process of selecting Interstate pavement sections for rehabilitation begins in the District offices. The four major selection criteria considered by the Districts are the pavement's CRS values (from the biyearly Condition Rating Survey, described below), average daily traffic (ADT), accident history, and public input. Other factors which are considered by District personnel in selecting projects for rehabilitation include the priority assigned to the project by IDOT's Interstate Review Team in its biyearly survey (described below), traffic capacity requirements, funding availability, and bridge rehabilitation requirements. Due to Federal requirements that bridge safety standards be met before or in conjunction with pavement rehabilitation, bridge improvement needs often constrain pavement rehabilitation. [2]

The Condition Rating Survey (CRS) is conducted in even-numbered years on all Interstate pavement sections by a team of experienced engineers from IDOT's Central office and District offices. Pavement sections are subjectively rated on a 1 to 9 scale, based on the distress observed. [3] A target CRS value of 6.5 or less is used for
programming purposes. Observations of AC/PCC pavements showed that when the CRS was below 6, extensive maintenance was needed to keep them in service.

The Pavement Review Team (PRT), also made up of experienced IDOT engineers, has in the past surveyed all Interstate pavements in odd-numbered years. Each section was assigned one of the following four "priorities": [3]

PRT 1-2: Pavement will need rehabilitation in one or two years.
PRT 3-5: Pavement will need rehabilitation in three to five years.
PRT 5-10: Pavement will need rehabilitation in five to ten years.
PRT > 10: Pavement will not need rehabilitation in the next ten years.

Pavement priorities based on the 1989 survey are available in the 1991 version of IDOT's Pavement Management File (PMF). In selecting projects for rehabilitation, District personnel are particularly interested in whether or not the priorities assigned to the pavement sections by the Pavement Review Team are consistent with their own assessment of rehabilitation priorities.

The Districts generally do not consider alternative rehabilitation strategies at this early point in the rehabilitation programming process, except to assess whether particular pavement sections should be rehabilitated or reconstructed. For example, a need for substantial geometric or safety improvements or extensive patching may dictate reconstruction.

Based on the criteria listed above, each of the Districts develops a prioritized list of projects for rehabilitation and submits the list to IDOT's Office of Planning and Programming. This office compiles a multiyear program for rehabilitation projects, ranked according to CRS, PRT priority, and ADT level. Timing of rehabilitation projects within the multiyear program is decided based on pavement condition, District priority, plan availability, and geographic balance.

Pavement and structure rehabilitation activities take first priority in funding, followed by other items such as rest areas and interchange modifications. Pavements with CRS below the minimum acceptable level receive funding first. The Office of Planning and Programming cooperates with the Districts and the central Highway Office to develop the annual program. [3]

The above description of the rehabilitation programming process applies to pavements on the Interstate system only. Non-Interstate pavements, for which less information is generally available, are handled differently. Traffic, functional classification, and pavement condition are used to prioritize these projects.
Other Available Condition Information

For this study, additional pavement condition data for Illinois Interstate AC/PCC pavements were obtained. These data are described in this section.

Distress Data

IDOT began conducting pavement distress surveys on the Interstate in 1985, and repeated the distress surveys in 1987 and 1989. A 500-foot-long sample unit is surveyed at each milepost, which represents a sampling of about ten percent of the Interstate highway mileage. For AC/PCC pavements, the following key distress types are frequently noted.

1. Overlaid patch deterioration (sq ft, low/medium/high severity),
2. Potholes and localized distress (number, L/M/H),
3. Patch joint reflection cracks (number, L/M/H),
4. Transverse joint reflection cracks (number, L/M/H), and
5. Transverse reflection cracks (number, L/M/H).

Some other less serious AC/PCC pavement distresses, such as block cracking and ravelling, are also observed.

During the distress surveys, two or three rut depth measurements are taken in each wheelpath for each sample unit. Rut depth is measured in 0.05-inch increments by a hand-held graduated measurement device which is slid under a 6-foot straightedge.

Distress data from these surveys are compiled and entered into the IPFS NOMAD2 database [4], which also includes project limits, design and construction information, and traffic data. This database is currently under development and is far from error-free. To minimize the likelihood of errors in the database affecting the analyses described here, the actual field survey sheets for one entire District were obtained from IDOT for use in this study.

South Dakota Profiler Data

IDOT began using a South Dakota Profiler for monitoring rutting and International Roughness Index on Interstate pavements in 1989. The Profiler data for the first District tested (in December 1989) were obtained from IDOT for this study. The remaining Districts were tested in 1990, and all of the Districts were tested again in 1991.

The South Dakota profiler measures rut depth continuously along a pavement with three non-contact sensors: one in each wheelpath, and one halfway between the two wheelpath sensors. The rut depth is defined as the average of the two differences between the wheelpath sensors and the center sensor. [5]
The International Roughness Index (IRI) is an indicator of pavement roughness. The IRI (inches/mile) is obtained from measurements of the pavement profile. The profile is computed continuously along the pavement from the output of an accelerometer, which measures the vertical acceleration of the vehicle with respect to an inertial reference, a displacement transducer, which measures the displacement between the vehicle and the pavement, and a distance transducer, which measures the distance travelled along the pavement. Pavement profile measurement is described in ASTM Standard Test Method E 950.

The computed profile may also be used to simultaneously produce, by simulation, the outputs of other measuring devices as if those devices had been used to measure the surface. [6, 7] Simulation of vehicle responses from profile measurements is described in ASTM Standard Test Method E 1170. Devices that can be simulated include the Bureau of Public Roads (BPR) Roughometer, the CHLOE Profilometer, the Mays Ride Meter, the PCA Road Meter, and various straightedge devices. [7, 8] Either a quarter-car or half-car model is used to simulate the vehicle response for the measurement method of interest. The IRI is obtained from a quarter-car model simulation using standardized vehicle parameter constants as specified in ASTM E 1170 for a "ride meter, vehicle-mounted."

**PSR Survey**

In the summer of 1989, University of Illinois civil engineering department staff conducted PSR surveys of all of the Interstate pavements in District 5. The pavements on I-57, I-70, I-72, and I-74 in District 5 total 526 two-lane miles in 126 construction sections (by direction). Of this mileage, 232 miles in 54 sections are AC/PCC pavement.

A Present Serviceability Rating (PSR), on a scale of 0 to 5, was given to each section on the basis of its ride quality only:

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The direction in which each section was surveyed was noted on the survey form. Some of the sections were rated in only one direction. The ratings were conducted while driving in the outer lane. Serviceability ratings were given by three persons and the average of the three ratings was used in later analyses.
Subjective Rating (CRS)

The CRS is currently the dominant condition factor influencing when Illinois Interstate pavements are scheduled for rehabilitation. Figure 2 illustrates the CRS history for an Interstate PCC pavement section which was overlaid in 1985. The future CRS is predicted from the following equation:

$$\log_{10} \left( 9.0 - CRS \right) = 0.4185 - 0.1458 \log_{10} THICK$$

$$+ 0.5732 \log_{10} AGE + 0.1431 \log_{10} CESAL$$  \hspace{1cm} (1)

THICK = AC overlay thickness, inches  
AGE = age of overlay, years  
CESAL = cumulative 18-kip ESALs on overlay, millions  
$$R^2 = 58$$ percent  
$$\sigma_y = 0.76$$  
n = 509

The steadily declining trend in CRS from 1985 to 1990 indicates that the CRS will be less than 6.0 by 1994, and will reach 5.0 by 1998.

Figure 2. Prediction of remaining life of an AC/PCC pavement section.
The importance of CRS as a tool in selecting rehabilitation projects was also examined by comparing it with other available condition indicators.

**CRS versus PSR**

CRS values for AC/JRCP and AC/CRCP pavement sections in District 5 were compared to the present serviceability ratings (PSR) obtained in the UI survey. The results are shown in Figure 3. The best-fit line between PSR and CRS for AC/CRCP is given by Equation 2.

![Graph showing comparison of PSR and CRS](image)

**Figure 3.** Comparison of serviceability and CRS ratings.

\[
PSR = -0.355 + 0.555 \times CRS
\]  

(2)

- \( R^2 = 74 \) percent
- \( n = 27 \)
- \( \sigma_y = 0.312 \) (standard error of the estimate)
The best-fit line between PSR and CRS for AC/JRCP is given by Equation 3:

$$PSR = -1.956 + 0.700 \text{ CRS}$$

- $R^2 = 93$ percent
- $n = 12$
- $\sigma_y = 0.235$

These results suggest that CRS is correlated to ride quality for AC/PCC pavements. As Figure 3 shows, the relationships between PSR and CRS apparently follow significantly different trends for AC/JRCP and AC/CRCP. An AC/JRCP section is likely to have a higher CRS at any given PSR level than an AC/CRCP section at the same PSR level. For example, the trends indicate that a PSR of 3.0, commonly considered a critical level for Interstate pavements, is reached by an AC/CRCP section at a CRS of about 6, but is reached by an AC/JRCP section at a CRS of about 7.

CRS versus Distress

Models for predicting CRS for bare and overlaid pavements have also been developed. [3] The following CRS prediction equation for AC-overlaid pavements was initially derived based on judgment, and was modified to improve its fit to distress data:

$$CRS = 9 - (7 \times RUT + 0.05 \text{ FAIL})$$

where $RUT = \text{average rutting, inch}$
- $\text{FAIL = medium- and high-severity transverse reflection cracks and patches per mile}$

Serviceability (PSR)

Serviceability is an important indicator of pavement condition because it represents the user's assessment of the pavement's ability to provide a safe and comfortable ride. Naturally, serviceability is strongly related to pavement roughness, and therefore to pavement distresses which cause roughness.

PSR versus IRI

In a recent study, very good models were developed to correlate PSR to IRI, using a large set of data from pavements in six states. [9] Figure 4 illustrates the correlation between PSR and IRI for all pavement types. This correlation is expressed by the following equation:
\[ PSR = 5 \cdot e^{-0.0041 \cdot IRI} \]  

PSR = mean panel serviceability rating, 0 to 5 scale  
IRI = International Roughness Index, in/mile  
\( R^2 = 73 \% \)  
n = 332  
\( \sigma_Y = 0.39 \)

Figure 4. Correlation of PSR and IRI for all pavement types. [9]

The various pavement types (AC, PCC, and AC/PCC) showed very similar correlations between IRI and PSR, so all of the data for these different pavement types were used to develop the above model. Correlations were also developed for the specific pavement types. For AC/PCC, the following equation was obtained:

\[ PSR = 5 \cdot e^{-0.0046 \cdot IRI} \]  

PSR = mean panel serviceability rating, 0 to 5 scale  
IRI = International Roughness Index, in/mile  
\( R^2 = 70 \% \)  
n = 89  
\( \sigma_Y = 0.38 \)
PSR versus Distress

Work by Darter [10] suggests that loss of serviceability in AC-overlaid PCC pavement is correlated to the occurrence of deteriorated reflection cracks. Based on a field survey of a limited number of AC/PCC pavements, a relationship was derived between PSR and number of medium- and high-severity reflection cracks per mile.

This approach was used to compare the PSR ratings given to the AC/PCC pavements surveyed for this study to the distress data for the pavements, for those sections which showed any medium- or high-severity distress. Distress quantities recorded in 500-foot sample units at each milepost were averaged and extrapolated to obtain the number of medium- and high-severity reflection cracks, patches, and localized failures per mile. The results are illustrated in Figure 5. The curve shown in Figure 5 is given by Equation 7.

![Graph showing PSR versus M-H Cracks, Patches, Failures, no/mile](image)

Figure 5. PSR versus reflection cracks, patches, and failures in AC/PCC pavement.

\[
PSR = 4.7096 e^{-0.07117 \text{FAIL}^{0.57073}}
\]

\( R^2 = 68 \text{ percent} \)
\( n = 18 \)
\( \sigma_Y = 0.22 \)
The model has a reasonable shape, although it would be preferable to have more points at high distress levels to confirm that PSR is predicted to decline at an appropriate rate with respect to distress. The data and the model suggest that an unacceptable level of distress, corresponding to a PSR of 3.0, is reached at around 25 medium-high cracks, patches, and failures per mile. A PSR level of 2.5 corresponds to about 45 medium-high cracks, patches, and failures per mile.

Reflection Crack Deterioration

Distress is perhaps the most important of the condition indicators discussed so far. For AC/PCC pavements, a key distress category is transverse reflection cracking, which includes reflected joints, cracks, patches, and localized failures in the PCC slab. Reflection cracking is important because it relates directly to loss of serviceability, as described above, and also because it is an indicator of the extent of deterioration in the underlying PCC slab. For most AC/PCC rehabilitation alternatives, the quantity and severity of deteriorated reflection cracks, deteriorated patches, and localized failures present also directly influence the quantity of repair needed, and thus the cost of the rehabilitation alternatives.

Critical levels of distress and serviceability are very useful for identifying pavement sections which are in immediate need of rehabilitation. However, sections usually must be programmed for rehabilitation a few years in advance of when the funds for their rehabilitation will actually become available. In addition, rehabilitation projects are often delayed due to insufficient funds, and during the period of delay, their condition deteriorates further and their rehabilitation costs may increase. It is therefore useful to be able to predict medium- and high-severity reflection crack occurrence over time.

Reflection Crack Deterioration in AC/JRCP

A schematic illustration of the progression of reflection crack deterioration in AC/JRCP is shown in Figure 6. When the overlay is placed, it has no reflection cracks, but low-severity reflection cracks typically develop rapidly, within 1 to 2 years. Under typical Illinois Interstate traffic levels, most if not all of the regular transverse joints, unrepaired transverse cracks, and full-depth repairs will produce reflection cracks. The percentage of low-severity cracks reaches its peak at or near 100 percent within a few years, and starts to decline as the percentage which deteriorate to medium and high severity increases.

Figure 7 illustrates the percentage of reflection cracks, patches, and failures rated medium or high severity for 17 AC/JRCP pavement sections in District 5. The data confirm the trends illustrated in Figure 6. Reflection crack deterioration (to medium and severities) begins about two years after overlay, and progresses rapidly.
Figure 6. Schematic illustration of reflection crack progression in AC/JRCP.

Figure 7. Reflection crack deterioration versus overlay age for AC/JRCP.
According to the trend shown, fifty to sixty percent of the reflection cracks reach medium-high severity within about five years, and sixty to seventy percent are medium-high severity within about ten years. Eventually all of the reflection cracks will deteriorate to medium-high severity, although no data for older overlays were available in the data set to suggest how many years this would require. Crack sealing and other maintenance activities would probably affect these trends.

The number of reflected cracks, patches, and localized failures of all severities for the AC/JRCP sections analyzed are shown in Figure 8. With the exception of one section which had 168 reflection cracks per mile within three years, the sections tend to approach a level of about 125 cracks per mile in ten to twelve years. The section with the very high level of reflection cracking is rather unusual; the traffic lanes in the other direction of the same project had only 63 cracks per mile.

![Graph showing M-H Refl. Cracks, Patches, Failures / Mile vs Age of Overlay, years](image)

Figure 8. Reflection cracks, patches, and failures versus overlay age for AC/JRCP.

It is reasonable to expect that over the long term, an AC overlay of 100-foot JRCP would develop about 125 reflected cracks per mile. This level corresponds to an average reflection crack spacing of about 42 feet, which would result from reflection of regular transverse joints and slightly more than one midslab reflection crack per slab, on average.

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Figure 7 and Figure 8 together give an indication of the typical number of medium- and high-severity reflected cracks, patches, and localized failures that may be expected for an AC/JRCP of any given age. These curves are intended primarily as an example of an approach to prediction of medium- and high-severity distress, and should be verified with a larger data set of Illinois AC/JRCP pavements. Factors which may influence the shapes of the curves and scatter of the data include the accuracy of the distress data, the effect of crack sealing on distress severity level, the type of underlying patches, the annual traffic level, and the load transfer of the cracks and joints in the underlying JRCP.

Each of the data points actually represents the average of between 1 and 10 sample units, depending on the pavement section length. However, each of these sample units is a length of only 500 feet which is assumed to represent the condition of a mile of pavement. The curves were plotted with respect to age instead of traffic because the range of annual ESAL applications was fairly narrow for these pavements.

Reflection Crack Deterioration in AC/CRCRCP

Reflection crack deterioration in the AC/CRCRCP sections studied did not, unfortunately, lend itself as well to modelling as did reflection crack deterioration in AC/JRCP. The percentage of total reflection cracks, patches, and failures rated medium or high severity did not follow any evident trend with respect to either age or accumulated ESAL.

Figure 9 illustrates the low-severity and medium- to high-severity reflection cracks, patches, and failures in 35 selected AC/CRCRCP sections. All but one of these pavements has "D" cracking in the CRCP, and all but two have thin AC overlays (less than 4 inches).

Eighteen of the AC/CRCRCP sections were less than five years old, and these sections had fairly little reflection cracking. However, the reflection cracking that was present was not all low-severity; in fact, three of the youngest overlaid sections had medium- and high-severity reflection cracking. Of the remaining seventeen sections, which were five to ten years old, fifteen had between 20 and 70 reflection cracks per mile, and two had even more. The percentage of reflection cracking rated medium or high severity does not appear to be related to overlay age.

The occurrence and deterioration of reflected distress in AC/CRCRCP is much less predictable than in AC/JRCP, without access to preoverlay condition and repair data. Previous research has demonstrated that occurrence and deterioration of reflection cracking in an AC overlay of CRCP without "D" cracking can be delayed for many years by repairing the CRCP prior to overlay with reinforced PCC full-depth repairs, tied or welded to the existing slab to maintain slab continuity. [11]
In the first several years of the life of an AC overlay of this type of pavement, reflection cracking is due to working (medium- and high-severity) cracks and patch joints which were not repaired with continuously reinforced PCC. In addition, it is reasonable to expect that some other cracks and full-depth repair joints will eventually start breaking down under heavy traffic and produce reflection cracks.

Unrepaired distress in "D"-cracked CRCP apparently causes more rapid distress reflection and deterioration in an AC overlay than unrepaired distress in non-"D"-cracked CRCP. This is exacerbated by the fact that "D"-cracked CRCP generally receives less preoverlay repair, in terms of both quality and quantity. Preoverlay repair of "D"-cracked CRCP is often done with full- or partial-depth AC patches, due to the difficulty of tying continuously reinforced PCC full-depth repairs into an unsound slab. Localized failures caused by "D" cracking are often left unrepaired, and other localized areas of slab disintegration may develop as the "D" cracking progresses.

Rutting

One of the reasons that rutting is an important AC/PCC pavement condition indicator is that it relates directly to safety, specifically, the risk of wet-weather accidents. Most highway agencies consider one third to one half inch of rutting in
AC-surfaced pavements unacceptable from the standpoint of safety. For the purpose of rehabilitation project selection, it is important to be able to obtain reliable rut depth measurements for a section of AC/PCC pavement and to assess whether rutting on the pavement has reached an unacceptable level.

Very few models exist for prediction of rutting in AC overlays of PCC pavements. The following model for prediction of rutting on AC/PCC pavements was recently developed for Illinois Interstates [3]:

\[
RUT = ESAL^{0.655} \times AGE^{0.138} \times (0.55 + 0.009 \times THICK) \tag{8}
\]

where
- \( RUT \) = average rut depth, inch
- \( ESAL \) = accumulated ESALs since overlay, millions
- \( AGE \) = age of overlay, years
- \( THICK \) = AC overlay thickness, inches
- \( R^2 = 74 \) percent

Other statistics for this model (i.e., standard error of the estimate and number of observations) are unavailable.

Another model, developed using field data from a nationwide survey of AC overlays of cracked and seated PCC pavements [12], is given by Equation 9:

\[
RUT = 0.084807 + 0.019208 \times ESAL + 0.012512 \times AGE + 0.001199 \times PTRUCK
- 0.004177 \times PRECIP + 0.002798 \times \left( \frac{FI}{OLTHICK} \right) + 0.0064465 \times ZONE \times OLTHICK \tag{9}
\]

where
- \( RUT \) = average wheelpath rutting, inch
- \( ESAL \) = accumulated 18-kip equivalent single-axle loads since overlay, millions
- \( AGE \) = years since overlay
- \( PTRUCK \) = percent trucks in average daily traffic
- \( PRECIP \) = annual precipitation, inches
- \( FI \) = Freezing Index, F degree-days
- \( OLTHICK \) = overlay thickness, inches
- \( ZONE \) = climatic zone, 1 to 9, from Equation 10:

\[
ZONE = -5.9531 + 0.14263 \times ANNTEMP
- 0.012123 \times PRECIP + 0.1955 \times TRANGE \tag{10}
\]
ANNTEMP = average annual temperature, °F
TRANGE = average monthly temperature range, °F

Statistics for RUT model:

\[ R^2 = 71 \text{ percent} \]
\[ n = 101 \]
\[ \sigma_Y = 0.06 \text{ inch} \]

For this study, a model of the same form as Equation 9 was derived from the same data, but using ESALs increased to reflect higher truck factors (ESAL/truck) than were assumed in development of the original model. The revised model is given by Equation 11. The revised traffic values result in minor changes in the coefficients obtained, and the fit of both models is about the same.

\[
RUT = 0.083803 + 0.013069 \text{ ESAL} + 0.012910 \text{ AGE} + 0.001201 \text{ PTRUCK} \\
- 0.004143 \text{ PRECIP} + 0.002801 \left( \frac{FI}{OLTHICK} \right) + 0.0064554 \text{ ZONE} \times OLTHICK \tag{11}
\]

\[ R^2 = 71 \text{ percent} \]
\[ n = 99 \]
\[ \sigma_Y = 0.05 \text{ inch} \]

This model form has the advantage that it contains terms reflecting the influence of climate on rutting. However, the model was developed from projects with wide ranges of AC mix properties, whereas Equation 8 was developed from Illinois projects only, overlaid either before or after 1984. The models have about the same goodness of fit to the data from which they were developed.

In general, it is very difficult to reliably predict rutting of AC overlays of PCC pavements for the broad range of AC mix designs and climatic conditions throughout the United States. It is possible, however, to predict future rutting with improved accuracy for a particular project by calibrating a given rutting model to the current measured rutting.
Critical Condition Levels for Rehabilitation

The analyses presented in this section indicate that critical levels of CRS, roughness, and distress can be identified which correspond to a selected critical level of serviceability. Based on the samples of AC/PCC pavement sections considered in these analyses, the levels of CRS, IRI, and medium-high distress which correspond to a critical serviceability level of 3.0 are the following:

<table>
<thead>
<tr>
<th>Condition Indicator</th>
<th>Critical Level at PSR &lt; 3.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRS</td>
<td>&lt; 6.0</td>
</tr>
<tr>
<td>IRI</td>
<td>&gt; 110 inches/mile</td>
</tr>
<tr>
<td>M-H reflection cracks, patches, and failures</td>
<td>&gt; 25 per mile</td>
</tr>
</tbody>
</table>

In addition to these condition indicators, rutting should trigger a need for rehabilitation when it exceeds a level considered unacceptable, such as 0.4 inches. The same critical CRS of 6.0 is suggested for both AC/CRCP and AC/JRCP, although in the analysis done for this study using a small set of AC/JRCP sections, a CRS of 7.0 corresponded to a PSR of 3.0. These suggested critical condition levels are based on a sampling of AC/PCC pavement sections in downstate Illinois. Different critical levels may be more appropriate for Interstates with higher traffic volumes such as those in the Chicago area.

Trigger Condition Levels for Testing

The following levels of CRS, IRI, and medium-high distress are suggested for use in identifying AC/PCC pavements which should be scheduled for deflection testing and coring in preparation for rehabilitation:

<table>
<thead>
<tr>
<th>Condition Indicator</th>
<th>Trigger for Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRS</td>
<td>&lt; 7.0</td>
</tr>
<tr>
<td>IRI</td>
<td>&gt; 75 inches/mile</td>
</tr>
<tr>
<td>M-H reflection cracks, patches, and failures</td>
<td>&gt; 12 per mile</td>
</tr>
<tr>
<td>Rutting</td>
<td>&gt; 0.25 in</td>
</tr>
</tbody>
</table>
Cost-Effectiveness of Timely Rehabilitation

The critical levels given above indicate when rehabilitation is needed because an AC/PCC pavement's condition is unacceptable in terms of either ride quality, safety, or maintenance needs. However, earlier rehabilitation may provide better long-term performance and be more cost-effective. Research on rehabilitation of bare concrete pavements has shown that the initial and life-cycle costs of rehabilitation are strongly tied to pavement condition. [11, 13]

In particular, as the full-depth repair requirements increase, the costs of rehabilitation increase dramatically. It pays, therefore, to maintain pavements in good condition and to rehabilitate them before they exhibit substantial distress. Pavements with CRS less than 6.0 often require substantial maintenance efforts to keep them in service.
PART TWO -- PROJECT-LEVEL EVALUATION

3.0 AC/PCC PAVEMENT DISTRESS SURVEY

3.1 DISTRESSES IN AC/PCC PAVEMENTS

Distresses commonly seen in AC overlays of PCC pavements in Illinois include deteriorated reflection cracks, localized distress due to "D" cracking in the PCC layer, rutting, deterioration of underlying AC patches and expansion joints, and weathering and ravelling of the AC surface. Each of these is described in this section. Further information on distress types and severities in AC-surfaced pavements, including AC-overlaid PCC pavements, is available in Reference 14.

3.1.1 Reflection Cracking

The basic mechanism of reflection cracking in AC overlays of PCC pavements is strain concentration in the overlay due to movement in the vicinity of joints and cracks in the existing PCC slab. This movement may be either bending or shear induced by loads, or may be horizontal contraction induced by temperature changes. Load-induced movements are influenced by the overlay thickness and the thickness and stiffness of the existing pavement. Temperature-induced movements are influenced by daily and seasonal temperature variations, the coefficient of thermal expansion of the existing slab, and the spacing of joints and cracks. [15] A midslab reflection crack in AC/JRCP is shown in Figure 10.

In an AC overlay of jointed PCC pavement (JPCP or JRCP), reflection cracks typically develop relatively soon after the overlay is placed (often in less than a year). The rate at which they deteriorate depends on the factors listed above as well as the traffic level. Thorough repair of deteriorated joints and working cracks with full-depth dowelled PCC repairs reduces the rate of reflection crack occurrence and deterioration, if good load transfer is obtained at the full-depth repair joints. Other preoverlay repair efforts which may discourage reflection crack occurrence and subsequent deterioration include subdrainage improvement, subs Sealing of slabs which have lost support, and restoring load transfer at joints and cracks with dowels grouted in slots. [15]

A variety of measures have been used to attempt to control the rates of reflection crack occurrence and deterioration. Any of the following treatments may be employed in an effort to control reflection cracking in an AC overlay of jointed PCC pavement [15]:

1. Placing a synthetic fabric or a stress-absorbing interlayer prior to or within the AC overlay. The effectiveness of these techniques has been variable.
2. Placing a large-stone, bituminous-stabilized crack relief layer in combination with placement of the AC overlay.

3. Sawing and sealing joints in the AC overlay at locations coinciding with joints in the underlying JPCP or JRCP. This technique has been very successful.

4. Breaking and seating JRCP and cracking and seating JPCP prior to placement of the AC overlay. These techniques reduce the size of PCC pieces and seats them in the underlying base material, which reduces horizontal (and possibly vertical movements) across the cracks. Breaking refers to rupturing of the steel reinforcement in JRCP.
5. Rubblizing and compacting the PCC slab prior to placement of the AC overlay. This reduces the size of PCC pieces to a maximum of about 12 inches, comparable to a high-strength granular base course. This technique is applicable to all types of PCC pavement.

6. Increasing AC overlay thickness. Reflection cracks will take more time to propagate through a thicker overlay and may deteriorate more slowly.

Reflection cracking can have a considerable (often controlling) influence on the life of an AC overlay of jointed PCC pavement. Deteriorated reflection cracks reduce a pavement's serviceability and also require frequent maintenance, such as sealing, milling, and patching. Reflection cracks also permit water to enter the pavement structure, which may result in loss of bond between the AC and PCC, stripping in the AC, progression of "D" cracking or reactive aggregate distress in PCC slabs with these durability problems, and softening of the base and subgrade. For this reason, reflection cracks should be sealed as soon as they appear and resealed periodically throughout the life of the overlay. Sealing low-severity reflection cracks may also be effective in retarding their progression to medium and high severity levels. [15]

Permanent patching of punchouts and working cracks with tied or welded reinforced PCC full-depth repairs will delay for many years the occurrence and deterioration of reflection cracks in an AC overlay of CRCP. [11] Improving subdrainage conditions and subsealing in areas where the slab has lost support will also discourage reflection crack occurrence and deterioration. Reflection crack control treatments are not necessary for AC overlays of CRCP, except at longitudinal joints, as long as continuously reinforced PCC repairs are used for deteriorated areas and cracks. [11, 15]

The importance of preoverlay repair to the prevention of reflection cracking in an AC overlay of CRCP is demonstrated by the performance of the experimental CRCP rehabilitation project on I-57 at Manteno, Illinois. [11] Every patch, crack, and joint on this project was mapped before the AC overlay was placed. Distress surveys and nondestructive deflection tests were performed on the overlaid pavement immediately after overlay placement, and after one year, five years, and ten years. More than 90 percent of the tied PCC patches which were placed before the overlay could not be detected in the AC overlay by reflection cracks even after ten years of service. The patch locations are known by station, however, and also by patch numbers stamped in the overlay. A more detailed description of the performance of this project can be found in Reference 11.
3.1.2 "D" Cracking

"D" cracking is progressive deterioration of Portland cement concrete which occurs as a result of freeze-thaw damage in the large aggregates. While "D" cracking is not caused by traffic loads, it eventually results in severe deterioration of the PCC, particularly at the outer slab edges, along the centerline, and in the wheelpaths near joints and cracks.

"D" cracking occurs frequently in PCC pavements in the northeastern, north central, and south central regions of the United States. [16] The major factors which influence the likelihood that a pavement will develop "D" cracking are availability of moisture (including quality of base drainage) [17], occurrence of freeze-thaw cycles [18], coarse aggregate composition (sedimentary rocks such as limestone and dolomite are generally most susceptible) [19], pore size distribution of the coarse aggregate [20, 21, 22, 23], and maximum aggregate size. [24, 25] The fine aggregate does not influence the likelihood of "D" cracking. [18, 26] The level of air entrainment, likewise, does not affect "D" cracking [27] (although air entrainment does improve resistance to scaling caused by freeze-thaw damage in the cement mortar). The composition or brand of cement has little or no influence on "D" cracking. [27]

"D" cracking occurs in Illinois in PCC pavements constructed with both gravel and crushed stone. The Illinois DOT found in a 1979 survey that only 42 percent of the Interstate system was free of "D" cracking. Low-severity "D" cracking was present on 40 percent of the system, medium-severity "D" cracking on 12 percent, and high-severity "D" cracking on 6 percent. [28] It has also been Illinois' experience that "D" cracking results in more severe deterioration in CRCP than in JRCP. One explanation proposed for this is that the many fine cracks which naturally occur in CRCP offer more locations for infiltration of water into the PCC. [16] Another possible explanation is that CRCP sections in Illinois are constructed on dense-graded cement-stabilized or asphalt-stabilized base courses, which may have worse drainage characteristics than the granular base courses on which JRCP sections are constructed.

A thorough investigation of "D" cracking in AC/PCC pavements in Illinois found that AC overlays do not halt the progression of "D" cracking, and in some cases accelerate it. [29] Although the AC layer insulates the PCC to some extent, thereby raising the minimum pavement temperature and decreasing the number of freeze-thaw cycles in the PCC, it also decreases the rate of freezing, which is detrimental. In the laboratory, PCC samples subjected to 400 freeze-thaw cycles (equivalent to, for example, about 4 years of exposure near St. Louis), experienced reductions in split tensile strength of 20 percent. Samples with 2-inch and 4-inch AC overlays experienced greater strength losses than samples with no overlays. [29] "D" cracking may also contribute to loss of bond between the PCC and an AC overlay [29]. Loss of bond can result in increased rutting in the AC layer.
In that laboratory study, autogenous healing apparently occurred in "D"-cracked PCC samples when the samples were exposed to warmer temperatures. If autogenous healing occurs in the field, it may recover some of the strength lost due to "D" cracking. In some areas in Illinois, for example, warming during the summer could achieve strength gains that would offset strength losses due to freeze-thaw damage in the winter. This recovery process could only continue until the unhydrated cement in the PCC was consumed, after which one would expect the strength to steadily decline with successive winters. Although only a hypothesis, this may explain why some pavements do not manifest significant "D" cracking until they are ten or more years old.

An AC overlay of a "D"-cracked PCC pavement may not exhibit visible distress until the "D" cracking deterioration reaches an advanced state. On a jointed pavement, the overlay may deteriorate only in the vicinity of transverse joints or cracks which are affected by "D" cracking. However, on either a jointed or continuously reinforced pavement with an AC overlay, severe "D" cracking may eventually be manifested by concrete fines pumping through cracks in the AC overlay, pothole-like localized failures in the wheelpaths, along the centerline, or along the outer edge of the pavement, and even cracking in the wheelpaths which resembles alligator cracking.

"D" cracking distress in AC/CRCP is illustrated in Figure 11 and Figure 12. The distress shown in the first photograph would probably be rated low severity and therefore not in need of repair, despite the fact that white fines have pumped through the cracks in the AC overlay. The second photograph was taken at another location on the same project, and shows "D" cracking distress which has progressed to high severity.

Figure 13 is a photograph taken at a location where the pavement was being repaired. This photograph of the pavement cross section clearly illustrates the severe deterioration of the CRCP. The pavement shown in these three photographs is I-74 at Mansfield.

The strength loss caused by "D" cracking is also evident on a 9-mile AC overlay section on Interstate 70 near Marshall, Illinois. Severe "D" cracking of the 8-inch CRCP existed before the AC overlay was placed in 1980. The "D" cracking has continued, and in combination with a high volume of repeated heavy loads (10 million ESALs since 1980), has caused the concrete to completely disintegrate in some locations. Pumping of concrete fines is evident at several locations in the wheelpaths and at the edge joint. Nondestructive deflection testing was performed along the project using a Falling Weight Deflectometer (FWD). Sixteen cores were also taken at various cracks, localized failures, and apparently sound sections. Nearly all of the cores showed severe disintegration of the CRCP slab. Higher FWD deflections and lower backcalculated moduli of the CRCP slab corresponded to higher-severity cracks or failures and to more deteriorated areas in the underlying CRCP. Large areas along this project are showing major localized slab disintegration.
Figure 11. Low-severity "D" cracking in AC/CRCP.

Figure 12. High-severity "D" cracking in AC/CRCP.
Figure 13. Severe "D" cracking deterioration evident in cross-section of AC/CRCP.

3.1.3 Rutting

Rutting is another major distress in AC overlays of PCC pavements. Wheelpath ruts greater than one third to one half inch are considered by many highway agencies to pose a safety hazard, due to the potential for hydroplaning, wheel spray, and vehicle handling difficulties. [30]

The four mechanisms which cause rutting in AC are deformation of supporting layers, consolidation due to insufficient compaction during construction, surface wear due to studded tires and tire chains, and plastic deformation of the AC mix. Rutting due to permanent deformation of supporting layers cannot occur with a sound PCC slab. Rutting due to insufficient compaction generally does not contribute significantly to rutting in adequately constructed AC overlays of PCC. Rutting due to surface wear occurs on both AC and PCC pavements in some Western states where studded tires are still used, but is not seen in Illinois.

The primary mechanism of rutting in AC/PCC pavements is plastic flow of the mix laterally away from the wheelpaths, due to shear stress produced by applied loads. This plastic deformation normally occurs relatively slowly and develops into ruts of significant depth after several years. Mix deficiencies which increase an AC overlay's tendency to rut are rounded aggregates, excessive fines, improper aggregate gradation, stripping-susceptible aggregates, low air void content, low asphalt cement viscosity, and high asphalt content. [30]
The magnitude of shear stress experienced by the overlay under loading also depends on the AC/PCC interface bonding condition and the AC thickness, as illustrated in Figure 14. Shear stress in the AC is lowest when the AC and PCC are fully bonded, and increases when bond is lost. When the AC and PCC are fully bonded, AC shear stress increases with overlay thickness. This has been cited as a possible reason that very thin overlays do not fail primarily due to rutting. When the AC and PCC are not bonded, AC shear stress is highest for overlays between 4 and 6 inches thick. [31]

![Octahedral Shear Stress (psi)](chart)

**Figure 14. Shear stress in AC overlay of PCC pavement.** [31]

"Premature" rutting, which develops unusually rapidly and reaches a critical level within a year or two, occurs sometimes in an inadequately designed AC mix as a result of shear failure. Premature rutting has occurred in some AC overlays on Illinois Interstates, particularly in overlays placed prior to about 1984, as a result of inadequate AC mix design or field control.

For example, several miles of AC overlays placed on I-55 and I-70 prior to 1984 rutted prematurely. These overlays were either removed by cold milling and replaced with new overlays or were cold milled and not replaced. An Illinois DOT Task Force investigated the occurrences of premature rutting and concluded that the AC mix used for overlays at that time had gradation problems and low air voids. Changes in the mix design were made to correct these and other problems and were implemented during the 1984 construction season. [32]
3.1.4 AC Patches and Expansion Joints

Full-depth AC patches and "expansion" joints in many CRCP and JRCP pavements placed prior to overlay often cause significant distress in the AC overlay. In addition to creating a hump in the overlay surface, these discontinuities produce transverse cracks which reflect through the overlay and typically deteriorate very quickly. This increases roughness and maintenance requirements. [11, 33] Reflection cracking and roughness caused by AC patches are illustrated in Figure 15 and Figure 16.

Figure 15. AC patch reflection cracking in AC/JRCP.
3.1.5 Ravelling and Weathering

Ravelling and weathering are progressive deterioration of an AC surface as a result of loss of aggregate particles (ravelling) and asphalt binder (weathering) from the surface downward. [14, 34] Ravelling and weathering occur as a result of loss of bond between aggregates and the asphalt binder. This may be due to hardening of the asphalt cement, dust on the aggregate which interferes with asphalt adhesion, localized areas of segregation in the AC where fine aggregate particles are lacking, or low in-place density of the AC due to inadequate compaction. Ravelling and weathering may pose a safety hazard if deteriorated areas of the surface collect enough water to cause hydroplaning or wheel spray. Loose debris on the highway which may be picked up by vehicle tires is also a potential safety hazard. [34]
3.1.6 Stripping

Stripping is a loss of bond between aggregates and asphalt binder induced by moisture, which typically progresses upward from the bottom of an AC layer. Stripping may be manifested in several different types of visible distress, including premature rutting, shoving, ravelling, and cracking. [34] It is often necessary to examine a sample from the AC in order to determine whether stripping is occurring in the layer. Stripping may not be evident from visual examination of the exterior of an AC core, since the circumference of the core may be significantly damaged by the coring drill. It may be necessary to split the core apart to examine its interior. If stripping has occurred, partially coated or uncoated aggregates will be visible.

Factors related to the likelihood of stripping include mineral and chemical composition of the aggregate, exposure history of aggregate (freshly crushed versus weathered stone), physical and chemical properties of the asphalt cement, water content of the AC mix, climate, and construction methods. [34] The likelihood of stripping may be reduced by using compatible aggregates and asphalt cements, drying the aggregate to a minimal water content prior to mixing with asphalt cement, achieving adequate compaction, providing adequate surface and subsurface drainage, and using an effective antistripping additive. Several antistripping agents are available; hydrated lime has been shown effective in research studies. [35, 36]

AC overlays of PCC pavements become particularly susceptible to stripping when the bond between the AC and PCC layers is lost and water is able to collect at this interface. Severe stripping represents a loss of structural integrity of the overlay, since the effective thickness of the AC layer is reduced as the stripping progresses.

3.2 MAJOR DETERIORATION MODES BY PAVEMENT TYPE

The Illinois Interstate AC/PCC pavements may be divided into four categories, depending on whether the PCC slab is JRCP or CRCP, and whether or not the PCC contains "D"-cracking aggregate. The relative performance of these four categories of AC/PCC pavement may be generalized in the following descending order.

1. AC/Non-"D"-cracked CRCP: When adequate preoverlay repair is done, good to excellent performance has been observed for pavements in this category. This type of pavement exhibits few reflective cracks. The experimental project on I-57 at Manteno is a good example. Ten years after placement of the overlay, few cracks had occurred in the overlay except at preoverlay working cracks which had not been repaired prior to the overlay. The AC overlay dramatically reduced deflections and slowed the development of punchouts and steel ruptures on this pavement, which had been occurring rapidly prior to the overlay. [11] The performance life of AC over non-"D"-cracked CRCP is likely to be controlled by rutting rather than reflection cracking or structural distress.
2. **AC / Non-"D"-Cracked JRCP**: Fair to good performance has been observed for pavements in this category. Reflection cracking at joints, including full-depth repair joints, is often the predominant mode of failure, unless premature rutting occurs. Deteriorated transverse reflection cracks also contribute to increased roughness, reduced serviceability, and increased maintenance requirements.

3. **AC / "D"-cracked JRCP**: Fair to poor performance has been observed for pavements in this category. Extensive preoverlay repair (i.e., full-depth PCC repair at nearly every joint and midslab cracks in many slabs) is typically required, and even so, the "D" cracking may continue to progress in the PCC slab after being overlaid. AC patches are often used instead of PCC, and cause deterioration of reflection cracks and increased roughness.

4. **AC / "D"-cracked CRCP**: Fair to very poor performance has been observed for pavements in this category. "D" cracking has been shown to shorten the life of bare CRCP in Illinois much more than it does the life of bare JRCP. [37] The same trend exists after overlay. Field investigations of AC / "D"-cracked CRCP sections (e.g., I-70 at Marshall) suggest that the deterioration of the PCC slab may be much more extensive than surface distress observations indicate.

### 3.3 DISTRESS SURVEY GUIDELINES

The following distress types and quantities should be recorded during a distress survey of AC/PCC pavement:

1. Deteriorated reflection cracks per mile;
2. Full-depth AC patches and expansion joints per mile (except at bridges);
3. Localized failures per mile, especially those caused by "D" cracking of PCC;
4. Punchouts per mile, for AC/CRCP;
5. Mean rut depth,
6. Alligator cracking in wheelpaths and/or severe shoving of AC, and
7. Evidence of pumping of fines or water at cracks and at pavement edge.

A map of the locations and severities of distresses with respect to stations along the project length is useful for several reasons:

1. Interpreting the deflection data,
2. Selecting locations for coring,
3. Identifying locations where photographs were taken, and
4. Identifying locations where full-depth repairs are needed.
Depending on the amount of distress present, two or three people working together should be able to map and photograph about 2.5 miles of pavement in an 8-hour work day. If 100 percent mapping seems unreasonable due to the length of the project, an alternative is to select sections of the project for sampling. If the project's condition seems to vary considerably along its length or by direction, representative test sections should be selected in these different areas. Deflection testing should then be done in the test sections surveyed.

The 10 percent distress sampling which is typically done for network condition monitoring is probably insufficient to obtain an accurate assessment of distress for the purpose of project-level evaluation purposes. A sample of about 20 percent or more of the project's length (e.g., 1000 to 1500 ft at each milepost, or 500 to 750 ft at each mile and half mile) is more appropriate for a project-level survey. A 100 percent survey is usually not done until relatively late in the rehabilitation design process, when plans, specifications, and bid estimates are being prepared.

The objective of sampling a smaller percentage of the project is to obtain a reliable estimate of the distress quantities and repair needs well in advance of the Resident Engineer's inspection. These quantities may be significantly underestimated, and result in significant cost overruns, if an insufficient sample size is used, or if the project is delayed for one or more years after the project-level distress survey is done, and deteriorates significantly in the interim.

The importance of distress surveying of AC/PCC pavement prior to rehabilitation cannot be overemphasized. Lack of data on PCC pavement condition prior to rehabilitation is the major hindrance to estimating the performance life of an AC overlay of PCC pavement and to predicting distress and serviceability in AC/PCC pavements. Without distress surveying of AC/PCC pavements prior to second rehabilitation, it will be just as difficult to predict the performance and expected lives of second rehabilitation alternatives.
4.0 DEFLECTION TESTING GUIDELINES FOR AC/PCC PAVEMENT

Deflection testing on the AC/PCC pavements in this study was conducted with a Dynatest Falling Weight Deflectometer (FWD), owned and operated by IDOT's Bureau of Materials and Physical Research. The FWD is an impulse-loading deflection testing device, capable of applying loads between 1,000 pounds and 25,000 pounds by varying the number and drop heights of a set of weights. The dynamic load impulse has a duration of about 25 to 30 milliseconds, simulating a moving truck wheel load.

For highway pavement testing, the load is applied by a circular load plate with a radius of 5.9055 inches; a larger load plate is available for airport pavement testing. Deflections are measured by sensors at the center of the load plate and at as many as six other locations, up to seven feet from the load, as selected by the operator. The FWD operations are controlled, and applied loads and measured deflections are recorded, using a Hewlett-Packard computer in the tow vehicle. IDOT's FWD tow vehicle is equipped with a distance monitor which records the station at which each deflection test is done. FWD deflection testing is described further in ASTM Standard Test Method D 4694.

The procedures used for deflection testing of AC/PCC pavements in this study are described in this section. The available guidelines for deflection testing on bare PCC and AC pavements were modified as needed to establish procedures suitable for AC/PCC pavements.

4.1 TRAFFIC CONTROL

Deflection testing on highway pavements is usually done in the outer traffic lane only, since the outer traffic lane carries most of the truck traffic and thus typically exhibits much more load-associated distress than the inner lane. In addition, closing the outer lane for deflection testing is considered safer than closing the inner lane, due to the perception that motorists are more accustomed to and better able to respond to outer lane closures than inner lane closures. Deflection testing in the inner lane is usually only done when the inner lane is already closed for other reasons (e.g., for repair or bridge work) and the outer lane is open for traffic.

IDOT has established procedures for conducting deflection testing in conjunction with a moving traffic control operation. This requires at least one arrow board, one heavy truck equipped with a rear-end impact attenuator, and one flagman near the FWD, but does not require placing and moving cones. The arrow board and impact attenuator remain far enough behind the FWD that motorists are adequately alerted to move to the inner lane, but not so far back that motorists might misunderstand which section of the highway is closed and reenter the outer lane behind the FWD.
Particular caution must be exercised when testing in the vicinity of ramps, so that the FWD and traffic control vehicles do not interfere with traffic entering or leaving the highway. Caution should also be exercised when testing on the downward grade of a hill or around a horizontal curve, since approaching traffic must be able to see the traffic control vehicles in time to safely move to the inner lane. The difficulty of testing in ramp, hill, and curve areas depends on the specific conditions of the site and on the traffic volume, and is best left to the judgment of the FWD operator and traffic control personnel. It is far better to forego deflection testing in these areas than to risk an accident.

4.2 NUMBER OF TEST STATIONS ALONG PROJECT LENGTH

Deflection testing on highway pavements is typically done at intervals of between 200 and 500 feet along a project. This corresponds to between 10 and 25 stations per mile. A test interval of 500 feet was used for most of the AC/PCC pavements tested for this study. Testing at closer intervals was sometimes done for short sections, e.g., every 10 feet for 50 or 100 feet. Intensive testing is useful for assessing the pavement condition in the vicinity of some visible distress or for assessing the variability of deflections.

In the field work conducted for this study, the FWD was typically able to test about 65 stations per hour, which corresponds to between 2.5 and 6 miles per hour depending on the testing interval selected. Therefore, at a normal pace and within the normal working hours for which traffic control is available, the FWD is fully capable of testing at least 20 miles of pavement in a single day, depending on the testing interval selected, and barring any equipment breakdown or other delays. The longest pavement sections actually tested were 9 to 10 miles in length, and were tested in both directions in one day unless a substantial amount of intensive testing was done. It is of no particular advantage to rush deflection testing on one project if it does not leave enough time in the same day to travel to and test another project, or if traffic control for testing another project has not already been arranged.

If testing in both directions is planned, a half hour should also be allotted for moving the traffic control operation to the other side of the highway. It is most convenient if testing in one direction can be complete by the middle of the day so that the traffic control adjustments can be made in conjunction with the lunch break.

4.3 LOAD PLATE POSITION AND SENSOR LOCATIONS

Deflection basin measurement in the outer wheelpath of the outer traffic lane is recommended, unless the wheelpath has significant rutting, in which case it may be necessary to move the load plate to the middle of the lane, between the wheelpaths. Inner wheelpath testing should be avoided since this will position the FWD and tow vehicle dangerously close to traffic in the inner lane.
The backcalculation procedure described in this report requires deflections measured at the center of the load plate and at 12, 24, and 36 inches from the center of the load plate. The deflections measured at these four sensors are referred to here as $d_{12}$, $d_{24}$, and $d_{36}$. If deflections are not measured at these locations, this and some other backcalculation procedures (e.g., COMDEF) cannot be used. Other backcalculation procedures are available (e.g., BISDEF) which accept deflections at any locations.

For measurement of load transfer, the FWD should be positioned with one edge of the load plate next to the joint or crack, with the joint or crack visible between the load plate and the nearest (front or back) sensor. This positioning is typically done by the FWD operator with an assistant standing on the shoulder. Some FWD models are equipped with a camera which permits the FWD operator to see the position of the load plate on the pavement.

For measurement of load transfer across joints and cracks, it is desirable to have a fifth sensor positioned on the opposite side of the load plate from the other sensors, 12 inches from the center of the load plate. The fifth sensor is referred to here as the back sensor and its deflection is referred to as "$d_{12}$." Using a fifth sensor permits two measurements of load transfer: with the load plate on the approach side of the crack, and the deflection measured on the unloaded (leave) side by the fifth sensor, and with the load plate on the leave side of the crack, and the deflection measured on the unloaded (approach) side by the sensor 12 inches from the center of the load.

Load transfer measurement on AC/PCC pavements may not be possible at some reflection crack locations if the crack is so badly deteriorated that the load plate and adjacent sensor cannot be properly seated or if deterioration or maintenance of the crack makes it difficult to judge the location of the joint or crack in the PCC slab.

4.4 LOAD LEVEL

For purposes of backcalculating pavement layer moduli and measuring load transfer across cracks and joints, a target load level of 9000 pounds is typical for highway pavements. Greater load levels are typically used for airport pavement testing. The actual load applied by the FWD may vary from the target load by as much as 1000 pounds.

Deflections measured at the applied load levels may be linearly scaled to 9000-pound deflections. This is useful for assessing the variation in maximum deflections along the length of a project or for assessing the magnitude of differential deflection across joints and cracks. However, it is not necessary to normalize deflections to a single load level in order to backcalculate the moduli of the pavement layers. A profile of PCC elastic moduli or foundation moduli along the length of a project may thus be developed with actual measured deflections and applied loads, if desired.
Detection of voids beneath bare PCC slabs is typically done using a "load sweep," i.e., measuring deflections for at least three load levels, such as 4000, 8000, and 12000 pounds. [38] The applicability or adaptation of these procedures to void detection in AC/PCC pavements was not addressed in this study.

4.5 NUMBER OF LOAD DROPS PER TEST STATION

After the FWD is positioned at a station, a small amount of weight is dropped to insure that the load plate is properly seated on the pavement. If it is not (because, for example, a rock is under the plate), an error message from the computer will alert the FWD operator. This seating drop is not recorded with the load and deflection data.

In addition to this seating drop, it is common practice when testing AC pavements to apply multiple load drops for each load level at each station tested. Indeed, ASTM D 4694 recommends at least two load drops per load level, and making additional drops (up to five total) "until deflections vary less than 5 percent." ASTM D 4694 further recommends that the first drop always be excluded from the deflection analysis. Multiple drops are considered necessary for flexible pavement testing because deflections typically decrease significantly (e.g., between 5 and 10 percent) between the first and second drop, due to compression of the pavement layers.

Multiple load drops are often used when testing bare PCC pavements as well, for the purpose of averaging results from several drops and assessing the variability in deflections from drop to drop. If this is not necessary, it would be desirable to make only one load drop per station. Multiple load drops do not significantly increase the time required for deflection testing, but they do require storage and manipulation of a much larger quantity of deflection data.

Maximum deflection (d₀) and deflection basin AREA (computed from deflections at 0, 12, 24, and 36 inches) are the two deflection basin parameters used to backcalculate pavement layer moduli for PCC and AC/PCC pavements according to the procedure presented in this report. If a significant difference exists in either d₀ or AREA between load drops, it must be interpreted as an indication of a significant difference in backcalculated pavement layer or foundation moduli.

To determine whether multiple load drops were necessary for testing AC/PCC pavements, two load drops were done at each station for two of the case study projects, I-74 at Mansfield and I-70 at Marshall. The deflections measured at 186 stations at Mansfield and 211 stations at Marshall were then subjected to paired t-tests at a confidence level of 95 percent to determine whether any statistically significant difference existed between the maximum deflection d₀ or deflection basin AREA for the first and second drops. The analyses did not show any significant difference in d₀ and AREA between the first and second drops. Based on these results, a single load drop was used for all of the remaining AC/PCC projects tested.
4.6 AC TEMPERATURE MEASUREMENT

The resilient modulus of AC varies substantially with temperature. Therefore, in order to analyze deflections measured on AC pavement or AC/PCC pavement, it is essential to adjust the deflections to account for the variation in temperature of the AC mix which occurs during the duration of the deflection testing. It is not uncommon for the AC mix temperature to vary by as much as 30 or 40 °F during a typical day of FWD testing. This magnitude of temperature variation could easily correspond to a variation of 600,000 psi in AC modulus. Failure to account for this variation may result in considerable errors in backcalculation of the PCC slab and foundation moduli.

For the AC/PCC pavements tested for this study, the temperature of the AC layer was monitored during deflection testing by drilling a hole to the middepth of the AC overlay, inserting liquid and a temperature probe into the hole, and reading the AC mix temperature when it had stabilized. This process takes only a few minutes. This was done three or four times during each day of testing (before testing began in the morning, before and/or after lunch, and when testing was completed in the afternoon).

The AC mix temperatures were entered as comments into the FWD data file, and later used to establish a curve of AC mix temperature versus time for each day of testing. Since the FWD records the exact time at which each station is tested, an AC mix temperature could then be assigned to each deflection basin. As described in Chapter Four, an established relationship for AC modulus as a function of temperature was then used to assign an AC modulus to each deflection basin. This method of fixing the AC modulus is a common practice in analyses of deflection data from AC pavements, particularly when the AC pavement has a high-modulus (e.g., stabilized base) layer beneath the AC surface.
5.0 MATERIALS SAMPLING AND TESTING GUIDELINES

Cores are obtained from AC/PCC pavements for the following purposes:

1. Resilient modulus testing of AC surface, for use in backcalculation,
2. Split tensile testing of PCC, for comparison with backcalculated moduli,
3. Visual examination of PCC for evidence of "D" cracking,
4. Examination of AC/PCC bonding condition (bonded or unbonded),
5. Confirmation of AC and PCC layer thicknesses,
6. Recovery of stabilized subbase, if possible, for visual examination and resilient modulus testing.

Most of the cores used in this study were taken with a 6-inch core drill, since split tensile testing of PCC should be done on 6-inch cores in order to obtain a good estimate of the flexural strength. Photographs were taken of the core locations. For the AC/CRCR sections with BAM (bituminous aggregate mixture) bases, the coring crew attempted to include the BAM in the core, but sound samples of BAM were rarely recovered. No attempt was made to recover granular base material from AC/JRCR projects. For one project, samples of subgrade material were obtained with an auger and were subjected to gradation testing.

In the laboratory, the cores were labelled and photographed, the AC and PCC layer thicknesses were measured, and the AC/PCC interface bonding condition was noted. If the AC and PCC were bonded, they were sawed apart. Since the diametral resilient modulus testing apparatus requires 4-inch-diameter AC cores, the 6-inch AC cores were cast in concrete and 4-inch-diameter cores were sawed out of them. The AC cores were also trimmed at each end to obtain test samples between 2 and 2.5 inches in length which would fit in the resilient modulus testing apparatus.

5.1 AC RESILIENT MODULUS TESTING

Repeated-load indirect tension testing of the AC cores was conducted in accordance with ASTM Standard Test Method D 4123. The cores were tested at 70° and 90°F in a temperature-controlled room. Prior to testing at each temperature, the cores were left in the temperature room overnight to stabilize at that temperature. A 185-pound load was applied at a frequency of approximately 1.25 Hz. The AC resilient modulus is computed from Equation 12.

\[ E_{ac} = \frac{P \times (0.2734 + \mu)}{(t \times \Delta h)} \]  

(12)

where \( E_{ac} \) = resilient modulus of AC, psi
P = load, pounds
\( \mu \) = Poisson's ratio of AC (0.35 assumed)
t = thickness of test sample, inches
\( \Delta h \) = diametral deflection, inches
Each core was tested twice, along perpendicular diameters, at each temperature. Therefore, the modulus value reported for a core at a particular temperature represents the average of two tests. An analysis of the variability of the AC resilient modulus testing method showed that the results were highly repeatable: multiple tests of cores yielded modulus values with an average coefficient of variation (standard deviation as a percentage of the mean) of about 3 percent. [39]

The variability in AC modulus among different cores may be much greater, however. Between two and eight AC cores were obtained for most of the AC/PCC pavement sections studied. The standard deviation of AC modulus values ranged from 48,000 to 262,000 psi (with an average of 119,000 psi), and an average coefficient of variation of 17 percent. The average of the modulus values for all of the cores obtained from a given project was used, unless a test result was rejected for some particular reason. For example, the temperature-monitoring apparatus of the temperature-controlled room malfunctioned at one point during the study, and a few tests done while the equipment was not functioning properly were considered suspect.

The subject of AC modulus variability, including the number of cores required to estimate the modulus within a certain range with a desired level of confidence, was not addressed in this study. This subject deserves further study, particularly with respect to the sensitivity of backcalculation results.

Many of the AC cores were also split in indirect tension, in order to examine the relationship between indirect tensile strength and resilient modulus. These data did not indicate a clear relationship between the two parameters.

5.2 PCC STRENGTH TESTING

Indirect tension testing of PCC cores was conducted in accordance with ASTM Standard Test Method D 496. Six-inch-diameter cores were requested to comply with the ASTM Standard and because most correlations between split tensile strength and flexural strength are derived from tests on 6-inch cores. The split tensile strength is obtained from the following equation.

\[ \sigma_t = \frac{2P}{\pi ld} \]  

where \( \sigma_t \) = split tensile strength of PCC core, psi  
\( P \) = maximum load, pounds  
\( l \) = core length, inches  
\( d \) = core diameter, inches

The standard deviation of PCC split tensile strengths ranged from 60 to 148 psi with an average of 103 psi, and an average coefficient of variation of 20 percent. The subject of PCC strength variability, including the number of cores required to estimate
the strength within a certain range with a desired level of confidence, was not addressed in this study.

5.3 VISUAL EXAMINATION OF AC AND PCC CORES

All of the cores were photographed and examined for evidence of deterioration, such as stripping in the AC or "D" cracking in the PCC. Several cores contained reinforcing steel, and several cores from AC/CRC core sections were broken at the depth of the steel. In general, little or no corrosion of reinforcing steel was noted.

For two of the case studies, I-70 at Marshall and I-74 at Mansfield, several cores were taken and photographed to examine the extent of deterioration caused by "D" cracking, and to relate the severity of distress at the pavement surface to deflection measurements and the condition of cores. At locations with unusually high deflections, coring invariably confirmed that the PCC was deteriorated.

5.4 INTERFACE BONDING CONDITION

The interface bonding condition was noted for each core obtained, but no attempts were made to conduct shear testing of the AC/PCC bond. For most projects, the AC and PCC were either bonded or unbonded for all of the cores obtained, but for some projects, both bonding conditions were noted. Older overlays were not necessarily more likely to be unbonded than younger overlays. The oldest AC/PCC pavement section in the study was I-74 from Urbana to St. Joseph, a 10-mile section of JRCP which was overlaid in 1979. All four cores obtained from this project were fully bonded at the AC/PCC interface. Cores from AC/CRC core sections were more often unbonded than AC/JRCP cores, particularly if the CRC was badly "D" cracked.

5.5 AC AND PCC LAYER THICKNESSES

The AC and PCC layer thicknesses were measured for the cores obtained, primarily to confirm the thickness of AC overlay given by construction records. Unless an obvious discrepancy was discovered, the AC overlay thickness indicated by the construction records was used in the backcalculation of the pavement layer moduli, rather than the average of the core thicknesses. The number of cores taken was not considered sufficiently large to reliably estimate an in-place AC overlay thickness other than the design thickness. The design PCC slab thickness was also used in backcalculation, for the same reason. One exception to this was a fairly short section of AC/PCC pavement from which several cores were taken. The PCC thickness of every core obtained was one quarter to one half inch greater than the design 7-inch thickness. In this case the average thickness of the PCC cores (7.3 inches) was used in the backcalculation.
5.6 BAM BASE FOR AC/CRCP

Sound specimens of BAM were recovered from only two of the AC/CRCP projects in the study, and the BAM cores were only suitable for testing for one of these projects. The two BAM cores had an average modulus of 162,500 psi at 70°F and 85,000 psi at 90°F.

The one BAM core obtained from the second project was easily broken down and subjected to gradation testing. The fines content (material passing the #200 sieve) of the BAM sample was 9.7 percent, indicating fairly low permeability of the BAM base. No granular base samples were obtained from AC/JRCP pavements for gradation testing. However, both the granular and BAM bases used for Illinois Interstate PCC pavements are dense-graded materials which are generally considered to drain poorly. It seems reasonable to speculate that the untreated granular base has somewhat better drainage characteristics than the BAM base, but this has not been thoroughly studied.

For the purpose of backcalculation, the base and subgrade were treated as one layer. This was considered very reasonable for the granular bases, and also reasonable for the BAM bases since in nearly all cases the BAM was moderately to severely deteriorated. Furthermore, the primary objectives of the backcalculation analyses done were to estimate the PCC slab modulus and to estimate the support given to the slab by the base/subgrade foundation. Backcalculation of the base moduli separate from the subgrade was not an objective, though this could be done with the deflection data collected.
6.0 BACKCALCULATION OF AC/PCC PAVEMENT LAYER MODULI

Much of the distress seen in AC/PCC pavements is reflected from deterioration in the underlying PCC slab. The PCC distresses which are most responsible for AC overlay deterioration are slab cracking, punchouts, joint deterioration, localized deterioration resulting from poor durability ("D" cracking and reactive aggregate distress), and deterioration of PCC and AC patches. This deterioration will also reflect through a second AC overlay unless it is identified and repaired. This requires a coordinated effort of distress surveying, nondestructive deflection testing (NDT), and coring for materials samples. The information obtained is valuable in establishing a profile of condition along the length of the project, which may be used to identify areas requiring specific repair and to determine second rehabilitation options.

Analysis of deflections measured at locations where the underlying PCC is severely deteriorated, as in the case of "D" cracking, will produce low backcalculated in situ PCC modulus values. These low modulus values should not be interpreted as the true stress/strain response of the PCC as a homogeneous elastic layer, but rather as an indication of the extent to which its behavior departs from that of a sound slab, i.e., the extent of the PCC's deterioration. The ability to diagnose the condition of the PCC from analysis of deflection measurements is particularly valuable in evaluation of AC/PCC pavements, since the extent of the deterioration of the PCC is often not fully evident from visible distress. In some cases, the deterioration of the PCC may be so severe and so widespread that the only feasible rehabilitation alternatives are substantial structural improvements such as a very thick AC overlay, an unbonded PCC overlay, or reconstruction. A second AC overlay must be sufficiently thick to reduce stresses and deflections in the PCC slab to low levels.

Structural evaluation using NDT data is perhaps more difficult for AC/PCC pavements than for all other pavement types. The available computer programs for backcalculation of pavement layer moduli possess a variety of theoretical and practical limitations which hinder their usefulness in AC/PCC pavement analysis. Valid and repeatable results are typically only obtained from even the best of these tools by very knowledgeable pavement engineers with considerable experience in backcalculation.

6.1 LIMITATIONS OF AVAILABLE BACKCALCULATION TOOLS

Most of the tools currently used for backcalculation of pavement moduli are computer programs based on multilayer elastic theory. These programs determine the in situ elastic moduli of pavement layers by matching deflection basin measurements to deflections predicted by multilayer elastic theory, given the layer thicknesses and Poisson's ratios and the magnitude and area of the applied load. A few backcalculation programs exist which utilize the equivalent thickness concept, i.e., reduction of a multilayer elastic system to an equivalent system of fewer layers for which a solution is more easily obtainable. Backcalculation may also be done using plate theory,
i.e., two-layer elastic theory for the special case of an upper layer which exhibits pure bending (without transverse shear deformation) in response to load. The use of plate theory permits the characterization of the subgrade as either a dense liquid or as an elastic solid.

In backcalculation programs based on multilayer elastic theory, actual deflections are matched to predicted deflections in one of two ways: by iterative numerical integration of elastic layer equations, or by searching a database of deflection basins which have been generated for ranges of layer thicknesses and moduli. Backcalculation by the equivalent thickness method may also be done by iteration or by database search. Graphical procedures were used in the first backcalculation methods based on plate theory, but direct solutions may also be obtained from closed-form equations.

6.1.1 Iterative Backcalculation Programs

BISDEF [40], CHEVDEF [41], WESDEF [42], and ELSDEF are examples of iterative backcalculation programs which make repetitive calls to an elastic layer analysis subroutine (e.g., BISAR [43] for BISDEF) in order to match measured deflections to deflections predicted for program-selected layer moduli. The process stops when the measured and predicted deflections match within tolerance levels set by the user, or when the maximum number of iterations set by the user is reached. A detailed description of the solution algorithm used in these programs is given by Anderson [44].

One limitation of iterative elastic layer backcalculation programs is that they require the user to enter starting values and ranges for the layer moduli. Unless appropriate starting values are selected, the program may never converge to a solution within the selected ranges. Some researchers have noted that there is no unique solution to the set of moduli which will produce a given deflection basin. Rather, there are as many solutions as there are layers in the pavement structure. [45, 46, 47] As a result, the solution toward which the program converges depends on the initial or "seed" modulus values selected. The boundary values must also be selected judiciously. Limits which are too narrow may prevent the program from converging to the correct solution. Limits which are too broad may allow the program to converge to an incorrect solution, particularly if inappropriate seed moduli are selected.

Success with these programs thus requires a good knowledge of pavements and materials and experience in backcalculation for the specific pavement type in question. It has even been suggested that iterative elastic layer backcalculation can never be truly automated until an expert system is developed to guide decisions such as selection of seed moduli. [45, 48] Another limitation of iterative backcalculation is that it is time-consuming, increasingly so for increasing number of layers. Convergence to a solution may require several iterations for a pavement system of three or more layers.
In general, the iterative elastic layer backcalculation programs available do not perform well in analyzing AC/PCC pavements, for both of the reasons cited above. Frequently they are unable to match predicted and actual deflection basins well even when given carefully selected ranges of moduli and permitted to run several iterations. Their tendency is to underpredict the modulus of the AC surface, often going to the lower limit of the AC modulus range allowed by the user, and consequently overpredicting the modulus of the PCC slab. As a result, it is necessary to confine the AC modulus to a narrow range bracketing an appropriate value (determined by independent means, e.g., as a function of AC mix temperature), in order to obtain meaningful backcalculated modulus values for the PCC layer. The long execution time required for backcalculation of AC/PCC pavement layer moduli is also a significant limitation. Analysis of several dozen AC/PCC pavement deflection basins, such as might be measured on a highway section a few miles in length, may require several hours of program execution time even on a high-end personal computer. A considerable amount of additional time is required to code the input so that the program will execute successfully.

BOUSDEF [49] is an iterative backcalculation program similar to BISDEF, except that deflections for trial layer moduli combinations are computed not by an elastic layer subroutine but rather an equivalent thickness subroutine. This dramatically reduces execution time, which is BOUSDEF's major advantage over the BISDEF class of programs. However, the appropriateness of BOUSDEF for backcalculation of AC/PCC pavement layer moduli is questionable, owing to material behavior limitations which are inherent in the assumptions of the equivalent thickness method. These include the assumption that the pavement layers above the subgrade exhibit pure bending behavior, the assumption that all layers are fully bonded at their interfaces, the assumption that the layer moduli decrease with depth, and the assumption that the equivalent thickness of any layer with respect to the layer below is larger than the radius of the applied load.

6.1.2 Database Backcalculation Programs

Database backcalculation programs run much more quickly than iterative programs, but require a large amount of computer storage. Furthermore, a database backcalculation program can only be applied to situations comparable to those for which the database was generated, i.e., number of layers, material types, ranges of thicknesses and elastic moduli, interface bonding conditions, magnitude and geometry of loading, and number and spacing of sensors.

Of the backcalculation programs currently available, the database program COMDEF [50] is the only one developed specifically for AC/PCC pavements. COMDEF's database of deflection basins contains the results of more than 40,000 elastic layer program (BISAR) runs. As a result, the complete COMDEF database occupies more than 4 Megabytes of hard disk space on a personal computer. It is possible to load portions of the database corresponding to the specific cross-sections of interest to
conserve hard disk space. A second and more serious limitation of COMDEF is that it requires deflections for 7 sensors at 12-inch spacings; it cannot accommodate fewer sensors or other spacings. COMDEF does not permit the user to choose whether to model the AC/PCC interface condition as bonded or unbonded, and the program's documentation does not indicate which interface condition (presumably bonded) was used in the development of the database.

MODULUS [51] is a database backcalculation program in which the deflection basin database is produced by a factorial of elastic layer program (CHEVRON) runs. MODULUS was developed for analysis of flexible pavements, but may be used to analyze AC/PCC pavements if a database of deflection basins is generated for the specific AC/PCC pavement cross-section to be analyzed. This process may take between 15 minutes and an hour depending on the complexity of the pavement structure and the capabilities of the computer used, and must be repeated for every cross-section of interest. At least 1 Megabyte of hard disk space must be available to store the generated database. Once the database is generated, analysis of deflection data proceeds fairly quickly.

6.2 CLOSED-FORM BACKCALCULATION FOR BARE PCC PAVEMENTS

6.2.1 Deflection Basin AREA

A simple two-parameter approach to backcalculation of surface and foundation moduli for a two-layer pavement system was proposed by Hoffman and Thompson in 1981 for flexible pavements. [52] They proposed that the deflection basin could be characterized by its AREA as defined by the following equation:

\[
\text{AREA} = 6 \times \left[ 1 + 2 \left( \frac{d_{12}}{d_0} \right) + 2 \left( \frac{d_{24}}{d_0} \right) + \left( \frac{d_{36}}{d_0} \right) \right]
\]

(14)

where \( d_0 \) = maximum deflection at the center of the load plate, inches
\( d_i \) = deflections at 12, 24, and 36 inches from plate center, inches

AREA has units of length, rather than area, since each of the deflections is normalized with respect to \( d_0 \) in order to remove the effect of different load levels and to restrict the range of values obtained. AREA and \( d_0 \) are thus independent parameters, from which the surface and foundation moduli in a two-layer pavement system may be determined. Hoffman and Thompson developed a nomograph for backcalculation of flexible pavement surface and subgrade moduli from \( d_0 \) and AREA.
6.2.2 Radius of Relative Stiffness \( l_k \)

The AREA concept was subsequently applied to backcalculation of PCC slab elastic modulus values and subgrade k-values. [53, 54] Further investigation of this concept by Barenberg and Petros [55] and by Ioannides [56] has produced a forward solution procedure to replace the iterative and graphical procedures used previously. This solution is based on the fact that, for a given load radius and sensor arrangement, a unique relationship exists between AREA and the dense liquid radius of relative stiffness \( (\ell) \) of the pavement system, in which the subgrade is characterized by a k-value [57]:

\[
\ell = 4 \sqrt{\frac{Eh^3}{12(1 - \mu^2)k}}
\]  (15)

where \( \ell \) = dense liquid radius of relative stiffness, inches  
\( E \) = PCC elastic modulus, psi  
\( h \) = PCC thickness, inches  
\( \mu \) = PCC Poisson's ratio  
\( k \) = effective k-value, psi/inch

The following equation for \( \ell \) as a function of AREA was developed by Hall [58]:

\[
\ell = \left[ \ln \left( \frac{36 - \text{AREA}}{1812.279133} \right) \right]^{\frac{4.387009}{-2.559340}}
\]  (16)

Figure 17 illustrates the relationship between AREA and \( \ell \) for \( a = 5.9 \) in, the radius of an FWD load plate. With AREA calculated from measured deflections using Equation 14, \( \ell \) may be obtained from Equation 16 or Figure 17.

6.2.3 Slab Size Correction

This backcalculation procedure for the subgrade k-value and concrete slab E value employs Westergaard's equation for deflection of an infinite plate on a dense liquid foundation. Recent research has shown that an adjustment must be made to obtain appropriate k-values and concrete E values for slabs which are not sufficiently large to approximate infinite slab behavior. [59] If \( L/\ell \), the ratio of least slab dimension (length or width) to radius of relative stiffness, is less than about 8, incorrect k and E values will be backcalculated unless this adjustment is made. The necessary adjustment involves the following steps:
Figure 17. Relationship of AREA to $\ell$. [58]

2. Estimate $\ell$ from Equation 16.
3. Calculate $L/\ell$, where $L$ = least slab dimension (length or width) in inches.
4. Calculate adjustment factors for maximum deflection ($d_0$) and $\ell$ from the following equations:

$$ AF_{d_0} = 1 - 1.15085 \ e^{-0.71878 \left( \frac{L}{\ell} \right)^{0.00151}} $$ (17)

$$ AF_{\ell} = 1 - 0.89434 \ e^{-0.61662 \left( \frac{L}{\ell} \right)^{1.04631}} $$ (18)

5. Calculate adjusted $d_0 = \text{measured } d_0 * AF_{d_0}$
6. Calculate adjusted $\ell = \ell * AF_{\ell}$
7. Proceed with backcalculation of k-value and concrete E as described below, using adjusted $d_0$ and $\ell$. 

50
6.2.4 Backcalculated k-value

The backcalculated k-value may be obtained by rearranging Westergaard's [57] deflection equation to solve for k, using $d_0$ and $\ell$:

$$k = \left( \frac{P}{8 \ d_0 \ \ell^2} \right) \left[ 1 + \left( \frac{1}{2 \ \pi} \right) \ln \left( \frac{a}{2 \ \ell} \right) + \gamma - 1.25 \right] \tag{19}$$

where $d_0 =$ maximum deflection, inches

$P =$ load, pounds

$\gamma =$ Euler's constant, 0.57721566490

Figure 18 was developed from Equation 19 for a load $P = 9000$ pounds and a load radius $a = 5.9$ inches. For loads within about 2000 pounds of this value, the deflections $d_{d0}$, $d_{12}$, $d_{24}$, and $d_{36}$ may be scaled linearly to 9000-pound deflections.

Note that the backcalculated k-value is typically about twice the k-value which would be obtained if a static plate bearing test were conducted on the subgrade. Thus, the backcalculated k-value should be divided by two to obtain an estimate of the plate bearing k-value for use in any overlay design procedure which requires a plate bearing k input.

6.2.5 Concrete Elastic Modulus

With the k-value known, the slab $Eh^3$ may be computed from the definition of $\ell$ (Equation 15), and for a known or assumed slab thickness $h$, the concrete elastic modulus $E$ may be determined. Figure 19 was developed for determination of the concrete slab $E$, assuming a Poisson's ratio $\mu = 0.15$ for the PCC and a load radius $a = 5.9$ inches.

6.3 BACKCALCULATION FOR AC/PCC PAVEMENTS

In order to apply the backcalculation procedure described in the preceding section to an existing AC/PCC, deflections measured on the existing AC surface must be adjusted to account for the influence of the AC layer. The procedure for doing so is described in this section.

6.3.1 AC Elastic Modulus

An existing AC/PCC pavement cannot properly be modelled as a slab on grade, since the AC overlay exhibits not only bending but also compression. To determine the amount of compression that occurs in the AC overlay, the elastic modulus of the AC
Figure 18. Effective dynamic k-value determined from $d_0$ and AREA.

Figure 19. PCC elastic modulus determined from k-value, AREA, and slab thickness.
layer must be determined. The recommended method for determining $E_{ac}$ is to monitor the temperature of the AC mix during deflection testing, and to use a relationship between $E_{ac}$ and temperature to assign a modulus value to each deflection basin.

The AC mix temperature may be measured directly during deflection testing by drilling a hole to the middepth of the overlay, inserting a liquid and a temperature probe into the hole, and reading the AC mix temperature when it has stabilized. This should be done at least three times during each day's testing, so that a curve of AC mix temperature versus time may be developed and used to assign a mix temperature to each basin.

If measured AC mix temperatures are unavailable, they may be estimated from pavement surface and air temperatures using procedures developed by Southgate [60], Shell [61], the Asphalt Institute [62], or Hoffman and Thompson [52]. Pavement surface temperature may be monitored during deflection testing using a hand-held infrared sensing device which is aimed at the pavement. The mean air temperature for the five days prior to deflection testing, which is an input to some of the referenced methods for estimating mix temperature, may be obtained from a local weather station or other local sources.

Two methods for determining the AC elastic modulus as a function of mix temperature are presented here. The first method uses the Asphalt Institute's equation for AC modulus as a function of mix parameters, mix temperature, and loading frequency. This equation, developed by Witczak for use in the Asphalt Institute's Design Manual (MS-1) [62], is a refinement of work originally done for the Asphalt Institute by Kallas and Shook. [63] It is considered highly reliable for dense-graded AC mixes with gravel or crushed stone aggregates. [64]

\[
\log E_{ac} = 5.553833 + 0.028829 \left( \frac{P_{200}}{F_{0.17033}} \right) - 0.03476 \ V_v \\
+ 0.070377 \ \eta_{70^\circ F, 10^6} + 0.000005 \ t_p^{(1.3 + 0.49825 \log F)} P_{ac}^{0.5} \\
- \frac{0.00189}{F_{1.1}} t_p^{(1.3 + 0.49825 \log F)} P_{ac}^{0.5} + 0.931757 \left( \frac{1}{F_{0.02774}} \right)
\]  

(20)
where $E_{\text{ac}}$ = elastic modulus of AC, psi  
$P_{200}$ = percent aggregate passing the No. 200 sieve  
$F$ = loading frequency, Hz  
$V_v$ = air voids, percent  
$\eta_{70^\circ F, 10^5}$ = absolute viscosity at $70^\circ F$, 10$^5$ poise (e.g., 1 for AC-10, 2 for AC-20)  
$P_{\text{ac}}$ = asphalt content, percent by weight of mix  
$t_p$ = AC mix temperature, $^\circ F$

This can be reduced to a relationship between AC modulus and AC mix temperature for a particular loading frequency by assuming typical values for the AC mix parameters $P_{\text{ac}}, V_v, P_{200},$ and $\eta$.

Prior to 1984, these parameters had the following typical values in IDOT's AC mix design:

- $P_{200} = 5$ percent  
- $V_v = 2$ percent  
- $\eta_{70^\circ F, 10^5} = 1$ for AC-10, 2 for AC-20  
- $P_{\text{ac}} = 5$ percent

IDOT changed its AC overlay mix design in 1984 to address rutting problems attributed to gradation and low air voids. AC-20 was also required, whereas AC-10 had often been used in the past. These changes were implemented during the 1984 construction season. Therefore, the above mix parameters are appropriate for AC overlays placed prior to 1984, while for overlays placed in or after 1984, the following mix parameters are typical:

- $P_{200} = 4$ percent  
- $V_v = 5$ percent  
- $\eta_{70^\circ F, 10^5} = 2$ for AC-20  
- $P_{\text{ac}} = 5$ percent

For overlays placed before 1984, information on the asphalt viscosity grade used (AC-10 or AC-20) may be obtained from the MISTIC materials on-line database maintained by IDOT's Bureau of Materials and Physical Research in Springfield. In the absence of this information, it is safe to assume that AC-10 was more likely to have been used than AC-20. Asphalt content, void content, and fines content data may also be obtained from the MISTIC database, or the typical values given above may be used.

IDOT currently uses a Dynatest Falling Weight Deflectometer (FWD) for deflection testing. The FWD is an impulse loading device with a load duration of about 25 to 30 milliseconds. [65] This corresponds to a loading frequency of approximately 18 Hz. Figure 20 illustrates the modulus-temperature relationship for IDOT mixes before and after 1984 at this loading frequency.
Figure 20. AC modulus vs temperature for IDOT overlay mixes.

The following equations for these curves were obtained from the Asphalt Institute equation for the mix parameters given above.

**IDOT mix, AC–20, pre 1984:**

\[
\log E_{ac} = 6.712176 - 0.000164671 \; t_p^{1.92544}
\]

**IDOT mix, AC–10, pre 1984:**

\[
\log E_{ac} = 6.641799 - 0.000164671 \; t_p^{1.92544}
\]

**IDOT mix, AC–20, 1984 and after:**

\[
\log E_{ac} = 6.451235 - 0.000164671 \; t_p^{1.92544}
\]
The Asphalt Institute's equation for AC modulus applies to new mixes. AC which has been in service for some years may have either a higher modulus (due to hardening of the asphalt) or lower modulus (due to deterioration of the AC, from stripping or other causes) at any given temperature.

The second method for establishing a relationship between $E_{ac}$ and mix temperature involves repeated-load indirect tension testing (ASTM D 4123) of AC cores taken from the in-service AC/PCC pavement. Testing at two or more temperatures (e.g., 40, 70, and 90°F) is recommended to establish points for a curve of $\log E_{ac}$ versus temperature. AC modulus values at any temperature may be interpolated from the laboratory values obtained at any two temperatures, as shown below:

$$\log E_{ac} t = \left( \frac{\log E_{ac} t_1 - \log E_{ac} t_2}{t_1 - t_2} \right) * (t - t_1) + \log E_{ac} t_1$$

(22)

This straight-line interpolation method is suitable for assignment of moduli at temperatures which are not outside the range of $t_1$ to $t_2$ by more than about 10°F. Because the relationship of $\log$ AC modulus to temperature is not linear but rather S-shaped, extrapolation to much lower or much higher temperatures may produce unreasonably high or unreasonably low moduli, respectively. Laboratory testing at 70° and 90°F should be adequate for analysis of pavement deflections measured at temperatures between about 60°F and 100°F.

For the purpose of interpreting NDT data, AC modulus values obtained from laboratory testing of cores must be adjusted to account for the difference between the loading frequency of the test apparatus (typically 1 to 2 Hz) and the loading frequency of the deflection testing device (18 Hz for the FWD). This adjustment is made by multiplying the laboratory-determined $E_{ac}$ by a constant value which may be determined for each laboratory testing temperature using the Asphalt Institute's equation. Field-frequency $E_{ac}$ values will typically by 2 to 2.5 times higher than lab-frequency values. For this study, AC cores were tested in the laboratory with a load duration of approximately 0.4 second, which corresponds to a frequency of 1.25 Hz. The corrections given below were obtained for laboratory values of AC modulus at 70° and 90°F:

**IDOT mix, AC-10 and AC-20, pre 1984:**

$E_{ac \ FWD, \ 70^\circ F} = 2.0344 \ E_{ac \ lab, \ 70^\circ F}$

$E_{ac \ FWD, \ 90^\circ F} = 2.2446 \ E_{ac \ lab, \ 90^\circ F}$

(23)

**IDOT mix, AC-20, 1984 and after:**

$E_{ac \ FWD, \ 70^\circ F} = 2.0824 \ E_{ac \ lab, \ 70^\circ F}$

$E_{ac \ FWD, \ 90^\circ F} = 2.2976 \ E_{ac \ lab, \ 90^\circ F}$

56
Some researchers have been able to correlate AC resilient modulus to split tensile strength for specific AC mixes compacted in the laboratory. [66] However, AC resilient modulus and split tensile strength generally do not correlate as well for cores taken from in-service mixes. This is often true for samples taken from several different projects, and is sometimes true for samples taken from a single project.

Figure 21 illustrates the results of resilient modulus and split tensile testing done at 70°F on a large number of cores taken from a section of AC/PCC pavement on I-80 near Ottawa, Illinois. The 3-inch AC overlay was eight years old at the time the testing was done. Equally poor results were obtained for two other AC/PCC projects (IL 53 in Palatine and I-55/74 in Bloomington) from which several cores were taken. Because of the poor correlation observed, development of a model for AC resilient modulus versus split tensile strength was not pursued further in this study.

Figure 21. AC resilient modulus versus split tensile strength for cores from I-80.
6.3.2 Correction to \( d_0 \)

An elastic layer program (BISAR) was used to model AC/PCC pavement structures over a broad range of parameters:

AC thickness: 3, 5 and 7 inches  
AC modulus: 250, 500, 750, 1000, and 1250 ksi  
PCC thickness: 6, 9, and 12 in  
PCC modulus: 3, 5, and 7 million psi  
Subgrade modulus: 6, 24, and 42 ksi  
AC/PCC interface: bonded and unbonded

A load magnitude of 9,000 pounds and a load radius of 5.9 inches were used. Poisson's ratio values used for the AC, PCC and subgrade were 0.35, 0.15, and 0.50 respectively. The PCC/subgrade interface was modelled as unbonded.

Deflections were computed at the surface of the AC and the surface of the PCC at radial offsets of 0, 12, 24, and 36 inches. Compression in the AC layer, as indicated by the change in \( d_0 \) between the AC and PCC surfaces, often accounted for a significant portion of the total deflection, depending primarily on the thickness and modulus of the AC, and to a lesser extent on the AC/PCC interface condition. For example, in systems with a thick AC layer (7 inches) and a low AC modulus (250 ksi), more than 50 percent of the total deflection in the pavement occurred in the AC layer.

The change in \( d_0 \) is significantly greater when the AC is not bonded to the PCC than when it is bonded. For each interface bonding condition, it was found that the change in \( d_0 \) could be predicted very reliably as a function of the ratio of the AC thickness to AC modulus \( (h_\text{ac}/E_\text{ac}) \). These relationships were found to be very insensitive to the ranges of other parameters investigated. The following equations were obtained for these relationships:

**AC/PCC BONDED:**

\[
d_0 \text{ compress} = -0.0000328 + 121.5006 \left( \frac{h_\text{ac}}{E_\text{ac}} \right)^{1.0798}
\]

**AC/PCC UNBONDED:**

\[
d_0 \text{ compress} = -0.00002132 + 38.6872 \left( \frac{h_\text{ac}}{E_\text{ac}} \right)^{0.94551}
\] (24)
where $d_{o,\text{compress}}$ = AC compression at center of load, inches
$h_{ac}$ = AC thickness, inches
$E_{ac}$ = AC elastic modulus, psi

**AC/BONDED:**
- $R^2 = 99.97$ percent
- $\sigma_Y = 3.50 \times 10^5$ in
- $n = 180$

**AC/PCC UNBONDED:**
- $R^2 = 99.97$ percent
- $\sigma_Y = 4.34 \times 10^5$ in
- $n = 180$

Using these equations, the $d_0$ of the PCC slab in the AC/PCC pavement under a 9000-pound load may be determined by subtracting the compression which occurs in the AC surface from the $d_0$ measured at the AC surface.

The interface condition is a significant unknown in backcalculation. The AC/PCC interface is fully bonded when the AC layer is first placed, but how well that bond is retained is not known. Examination of cores taken at a later time may show that bond has been reduced or completely lost. This is particularly likely if stripping occurs at the AC/PCC interface. If the current interface bonding condition is not determined by coring, the bonding condition which is considered more representative of the project should be assumed.

In the elastic layer analyses conducted, only $d_0$ was found to change significantly between the AC and PCC layers. Changes in $d_{12}$, $d_{24}$, and $d_{36}$ were very small over the entire range of parameters. This is illustrated by Figures 22 and 23 for bonded and unbonded AC/PCC interface conditions respectively. The PCC $d_0$ values predicted by the above equation differ from the actual PCC $d_0$ values by at most -0.13 to +0.20 mils when the AC and PCC are bonded, and by at most -0.13 to +0.10 mils when the AC and PCC are unbonded.

### 6.3.3 Computed AREA of PCC

The AREA of the PCC slab may be computed from the following equation using the $d_0$ of the PCC slab determined as described above, and $d_{12}$, $d_{24}$, and $d_{36}$ measured at the AC surface. The PCC $d_0$ and $\text{AREA}_{pcc}$ may then be used to determine the PCC elastic modulus and effective dynamic k-value.

\[
\text{AREA}_{pcc} = 6 \times \left[ 1 + 2 \left( \frac{d_{12}}{d_{0,pcc}} \right) + 2 \left( \frac{d_{24}}{d_{0,pcc}} \right) + \left( \frac{d_{36}}{d_{0,pcc}} \right) \right]
\]
Figure 22. AC and PCC deflections computed by BISAR, with compression correction applied to $d_t$, AC/PCC bonded.
Figure 23. AC and PCC deflections computed by BISAR, with compression correction applied to \( d_0 \). AC/PCC unbonded.
Examination of Equation 25 shows that even very small errors in the predicted PCC $d_0$ can produce larger errors in the computed PCC AREA. Figures 24 and 25 illustrate the correlations between PCC AREA values predicted from Equation 25 (i.e., using AC surface deflections computed by BISAR and correcting $d_0$ using Equation 25) and AREA values calculated from the actual PCC deflections computed by BISAR, for bonded and unbonded AC/PCC respectively. The predicted PCC AREA values differ from the actual PCC AREA values by at most -0.75 to +0.45 inches when the AC and PCC are bonded, and by at most -0.57 to +0.26 inches when the AC and PCC are unbonded. However, the largest errors in the predicted PCC AREA correspond to actual PCC AREA values greater than 33.5 inches, which occur very rarely in the field.

6.3.4 Effect on AC Compression of Rigid Layer Beneath Foundation

This backcalculation method is based on an assumption of an infinite subgrade depth. Other researchers [67, 68] have noted the sensitivity of various backcalculation procedures to the depth and stiffness of a rigid foundation layer. The sensitivity of this procedure to a rigid foundation layer was investigated by taking a weak cross-section (2-inch AC, $E_{ac} = 100,000$ psi, 6-inch PCC, $E_{pcc} = 3$ million psi, and subgrade $E_s = 6,000$ psi), and determining the effect of a rigid layer (modulus 250,000 psi), at depths of 5 to 20 feet, on BISAR-computed deflections at the AC and PCC surfaces. The rigid layer had no significant effect on either the change in $d_0$ or the change in AREA between the AC and PCC. The depth to a rigid layer was therefore judged to be not sufficiently significant to require an additional correction.

6.4 COMPARISON WITH OTHER BACKCALCULATION RESULTS

The backcalculated PCC modulus values obtained from the procedure described in this chapter were compared with results obtained using the backcalculation program BISDEF using 100 deflection basins measured on I-74 near Mansfield, Illinois. The pavement is a 7-inch CRCP with a 3-inch AC overlay. Based on observations that the AC and PCC layers were unbonded in the cores recovered, the AC/PCC interface was treated as unbonded in both the AC/PCC backcalculation procedures and in BISDEF.

The dense liquid $E_{pcc}$ values backcalculated by the procedure described here are compared to the $E_{pcc}$ values obtained from BISDEF in Figure 26. In this comparison, the subgrade characterizations differ (dense liquid versus elastic foundation) as well as the slab characterizations (plate versus elastic layer). The net result, however, is that the dense liquid $E_{pcc}$ values and BISDEF $E_{pcc}$ values actually correlate fairly well.

A second factor in the observed scatter is the fact that the outputs of the BISDEF backcalculation are less repeatable than those of the plate theory backcalculation method. In fact, the PCC moduli obtained from BISDEF may be varied by several hundred thousand psi by manipulating the seed moduli, moduli limits, and tolerance limits.
Figure 24. Predicted versus actual PCC AREA values for bonded AC/PCC.

Figure 25. Predicted versus actual PCC AREA values for unbonded AC/PCC.
Figure 26. Comparison of backcalculated \( E_{pcc} \) values for AC/PCC pavement.

The BISDEF results shown here were obtained by carefully limiting the AC modulus for each basin to a narrow range bracketing the AC modulus assigned to the basin for the other backcalculation methods (i.e., using resilient modulus tests on cores, adjusting for the FWD frequency, and interpolating the modulus as a function of the mix temperature).

Another key difference between this method and iterative elastic layer backcalculation methods is the time required to obtain the results. Using this method, foundation k-values and concrete E values may be computed using a Lotus spreadsheet, which permits nearly instantaneous analysis of hundreds of deflection basins. The BISDEF analysis of the one hundred basins used in this analysis required a total of 63 minutes of execution time, an average of about 40 seconds per basin, on a very fast (33 Mhz, 80386 processor) personal computer. However, the greatest consumption of time associated with the BISDEF analysis was not the time required for the program to execute, but the time required to create and modify the inputs so that BISDEF would execute correctly. Preparation of the input files for the hundred basins used in this BISDEF analysis required twenty to twenty-five hours.

It should not be at all surprising that the results of the two different methods do not agree more closely, considering their very different theoretical bases, and the margin of error associated with each of the methods. From a practical standpoint, however, it is important not to place undue emphasis on the precision and accuracy of backcalculated moduli. Any reliable and convenient backcalculation method may be
used to obtain moduli, but the practicing engineer must keep in mind that other methods may yield different values. Backcalculation results should be viewed as "effective" moduli, as if the pavement layers and foundation did indeed behave in the ideal manner that the theoretical models suggest. In fact, the theories are only approximations of the real behavior of the pavement layers and foundation.

The conclusions drawn from the comparisons between this backcalculation procedure and BISDEF apply to a portion of the deflection data from this one project only. The deflection basins used for this comparison were the first 100 basins measured in the eastbound outer lane of the project, measured over a distance of about one and a half miles. The full length of the project is about three miles in each direction. The I-74 Mansfield section is a complete case study which is described in full in report IHR 532-5.

6.5 EXAMPLE OF AC/PCC BACKCALCULATION PROCEDURE

To illustrate the application of the backcalculation procedure, an example is given in this section using deflection and coring data from a section of I-70 near Marshall, Illinois. Analysis of the I-70 Marshall section as a case study is described in full in report IHR 532-5.

6.5.1 Project Description

Deflection testing was conducted in September 1989 on a 9-mile section of I-70 near Marshall, Illinois. The original pavement was an 8-inch CRCP on a 4-inch BAM (bituminous-aggregate mixture) base. The CRCP was constructed in 1968, and carried more than twice its design traffic by the time it was overlaid in 1980. Due to the heavy traffic and "D"-cracking aggregate used in the PCC, the pavement was severely deteriorated at the time that it was rehabilitated. The AC overlay placed was 4.5 inches thick.

The first four basins were measured ten feet apart in an eastbound section of the project which was rated in good condition, based on ride quality and visible distress. The second four, also ten feet apart, were measured westbound at the same milepost, in a section of the project which was rated in fair to poor condition.

6.5.2 AC Elastic Modulus

The AC mix temperature was monitored during deflection testing by drilling holes to the middepth of the overlay, inserting liquid and a temperature probe, and allowing the temperature to stabilize before reading. The mix temperature varied from 66°F at 9 AM to 90°F at 3 PM, as shown in Figure 27.
Figure 27. Temperatures during deflection testing for I-70 example.

Resilient modulus testing done later on cores from the AC surface showed modulus values of 1.141 million psi at 70°F and 392,000 psi at 90°F. These lab values, tested at 1.25 Hz, correspond to field values of 2.322 million psi and 879,000 psi respectively at the FWD frequency of 18 Hz. These values were used to assign an AC modulus of 1.176 million psi (at 84°F) to the four eastbound basins and 1.067 million psi (at 86°F) to the four westbound basins.

6.5.3 Backcalculation of PCC and Foundation Moduli

The backcalculation results for the eight deflection basins are shown in Table 1. The dense liquid $E_{pcc}$ values corresponding to the unbonded AC/PCC interface assumption average 4.7 million psi for the four eastbound basins, but only 1.1 million for the four westbound basins. Since the AC and PCC were observed to be unbonded in 15 of the 16 cores taken from this project, the backcalculated values corresponding to the unbonded interface assumption are considered, for this particular example, to be the more realistic values.
<table>
<thead>
<tr>
<th>AC layer thickness = 4.50 inches</th>
<th>PCC slab thickness = 8.00 inches</th>
<th>EASTBOUND (GOOD CONDITION):</th>
<th>AC/PCP interface = BONDED</th>
<th>AC/PCP interface = UNBONDED</th>
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Table 1. Backcalculation results for L-70 Marshall example, without slab size correction.
Table 1. Backcalculation results for I-70 Marshall example, without slab size correction (continued).

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AC/PCC interface = BONDED

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<td>23.67</td>
<td>18.71</td>
<td>424</td>
<td>1.19</td>
</tr>
<tr>
<td>E2</td>
<td>9000</td>
<td>7.51</td>
<td>4.38</td>
<td>3.90</td>
<td>3.20</td>
<td>7.21</td>
<td>22.44</td>
<td>17.20</td>
<td>503</td>
<td>1.01</td>
</tr>
<tr>
<td>E3</td>
<td>9000</td>
<td>8.78</td>
<td>5.51</td>
<td>4.80</td>
<td>3.75</td>
<td>8.48</td>
<td>23.24</td>
<td>18.16</td>
<td>385</td>
<td>0.99</td>
</tr>
<tr>
<td>E4</td>
<td>9000</td>
<td>7.39</td>
<td>4.94</td>
<td>4.30</td>
<td>3.62</td>
<td>7.09</td>
<td>24.70</td>
<td>20.19</td>
<td>375</td>
<td>1.43</td>
</tr>
</tbody>
</table>

AC/PCC interface = UNBONDED
It is evident that the PCC at the four eastbound basin locations is much stiffer than the PCC at the four westbound basin locations. Obviously, such low modulus values for the westbound basins are unreasonable for sound PCC, and suggest that the PCC in this area is severely deteriorated by "D" cracking. It is worth noting that although this entire mile of the project was rated in fair condition based on ride quality and distress observations, the deflections shown in Table 1 were measured at locations where the AC overlay was uncracked. This is consistent with the results of the subsequent coring operation. At locations where exceptionally high deflections were measured, the underlying PCC was invariably deteriorated, as shown in Table 2. This was true even at locations with little or no distress visible at the AC surface.

Table 2. PCC deterioration and backcalculated PCC modulus, I-70 Marshall.

<table>
<thead>
<tr>
<th>Core</th>
<th>Dense Liquid $E_pcc$ (psi)*</th>
<th>Recovered PCC Core Thickness, in</th>
<th>Severity of Surface Distress</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>2,011,000</td>
<td>2.5</td>
<td>Low</td>
</tr>
<tr>
<td>A2</td>
<td>918,000</td>
<td>3.5</td>
<td>None</td>
</tr>
<tr>
<td>A3</td>
<td>2,074,000</td>
<td>6.0</td>
<td>Low</td>
</tr>
<tr>
<td>B1</td>
<td>388,000</td>
<td>4.5</td>
<td>Medium</td>
</tr>
<tr>
<td>B2</td>
<td>646,000</td>
<td>AC patch</td>
<td>None</td>
</tr>
<tr>
<td>B3</td>
<td>57,000</td>
<td>PCC rubble</td>
<td>High</td>
</tr>
<tr>
<td>C1</td>
<td>808,000</td>
<td>9.75</td>
<td>Medium</td>
</tr>
<tr>
<td>C2</td>
<td>657,000</td>
<td>5.0</td>
<td>Medium</td>
</tr>
<tr>
<td>C3</td>
<td>8,485,000</td>
<td>9.5</td>
<td>None</td>
</tr>
<tr>
<td>C4</td>
<td>626,000</td>
<td>8.5</td>
<td>Low</td>
</tr>
<tr>
<td>C5</td>
<td>906,000</td>
<td>1.5</td>
<td>None</td>
</tr>
<tr>
<td>D1, D2</td>
<td>6,440,000</td>
<td>13.5</td>
<td>None</td>
</tr>
<tr>
<td>D3</td>
<td>187,000</td>
<td>1.5</td>
<td>High</td>
</tr>
<tr>
<td>D4, D5</td>
<td>113,000</td>
<td>PCC rubble</td>
<td>High</td>
</tr>
</tbody>
</table>

* Note: PCC thickness of 8 inches assumed in backcalculation.
6.6 ASSUMPTIONS IN BACKCALCULATION

The results of backcalculation by any method should be viewed in light of the inherent assumptions concerning the pavement layers and loading conditions. For the AC/PCC backcalculation procedure described here, these assumptions include impulse loading by an FWD, and characterization of the AC as an elastic layer, the PCC as a plate (an elastic layer exhibiting pure bending without transverse shear deformation), and the foundation as a dense liquid.

These characterizations are certainly simplifications of the true nature of the layer properties. The most obvious violation of these assumptions is the attribution of plate bending behavior to severely "D" cracked PCC, which may have more in common with a granular base than with a sound PCC slab. The low backcalculated values which result should not be interpreted as the true stress/strain response of the PCC as a homogeneous elastic layer, but rather as an indication of the extent to which its behavior departs from that of a sound slab, i.e., the extent of the PCC's deterioration. The ability to diagnose the condition of the PCC from deflection measurements is particularly valuable in evaluation of AC/PCC pavements, since the extent of the deterioration of the PCC is often not fully evident from visible distress.
7.0 FUNCTIONAL EVALUATION OF AC/PCC PAVEMENTS

The objective of a functional evaluation is to assess whether or not a pavement provides a safe and comfortable ride. The major causes of functional deficiency in AC/PCC pavement are described in this section.

7.1 RUTTING AND SURFACE FRICTION

Poor surface friction of an AC surface is usually attributable to asphalt bleeding or polished aggregates. Bleeding and polishing are primarily surficial problems which would be corrected when a second overlay was placed, but would need to be addressed in some other way if the pavement is not resurfaced.

Rutting which exceeds about 0.25 inch is a significant functional problem due to the potential for hydroplaning, wheel spray, and vehicle handling deficiencies. Rutting in an AC overlay of a PCC pavement typically develops gradually over several years. Significant rutting may warrant partial milling of the existing AC surface prior to placement of a second overlay, since it may otherwise be difficult to achieve good compaction of the new AC in the wheelpaths.

If premature rutting has occurred due to mix deficiencies, complete removal of the AC surface may be warranted. Mix problems which may contribute to premature rutting are described in Chapter Three. Mix data for AC overlays (air void content, asphalt cement content, etc.) may be obtained from IDOT's MISTIC materials database, or may be investigated by laboratory examination of cores.

7.2 ROUGHNESS

The major contributors to roughness in AC/PCC pavements are deteriorated reflection cracks, deteriorated patches, localized failures, and full-depth AC patches. Roughness may also be caused by ravelling of the AC surface. As described in Chapter Three, the development of a significant quantity of these distress types is generally accompanied by a noticeable reduction in ride quality.
8.0 STRUCTURAL EVALUATION OF AC/PCC PAVEMENTS

A thorough structural evaluation requires investigation of the condition and load-carrying contribution of each of the layers of the pavement system: the AC surface, the PCC slab, the base, and the subgrade. The results of the structural evaluation are crucial to decisions which must be made for rehabilitation programming and design, including:

1. Division of the project into uniform sections,
2. Identification of areas requiring repair,
3. Determination of structural improvement needs,
4. Selection of an appropriate overlay type, and
5. Second overlay design.

The PCC elastic modulus and foundation k value or elastic modulus may be backcalculated from slab deflection measurements. Backcalculation may be done using the procedure described in this report, or a variety of other methods available. Backcalculation can yield the following useful results:

1. Average PCC slab and foundation moduli of project,
2. Variability in slab and foundation moduli,
3. Percentage of the values below a selected critical level,
4. Trends in slab and foundation moduli over the length of the project,
5. Significant differences in slab and foundation moduli by direction,
6. Identification of specific areas with unusually high or low moduli, and
7. Relationship of PCC modulus to quantity and severity of distress.

8.1 PCC MODULUS AND STRENGTH

The average backcalculated PCC modulus over the length of a project is an important indicator of the pavement's structural capacity. Typical elastic modulus values for concrete slabs of various types and conditions are summarized in Table 3, along with general estimates of the corresponding remaining life of the concrete.

For jointed plain or jointed reinforced pavement, the PCC modulus of rupture may be estimated from the PCC modulus (backcalculated from deflection basins in uncracked areas assuming a k-value foundation), using Equation 26 developed by Foxworthy [69]:

$$ S'_{c} = 43.5 \left( \frac{E}{10^6} \right) + 488.5 \quad (26) $$
where \( S'_{c} \) = third-point modulus of rupture, psi

\( E \) = backcalculated PCC slab modulus, psi

\( R^2 \) = 71 percent

\( \sigma_t \) = 38.5 psi

\( n \) = 13

Table 3. Relationships between concrete condition, modulus, and remaining life.

<table>
<thead>
<tr>
<th>Concrete Slab Condition</th>
<th>Typical Modulus</th>
<th>Remaining Structural Life of Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sound JRCP or JPCP</td>
<td>3 to 8 million psi</td>
<td>More than five years</td>
</tr>
<tr>
<td>Sound CRCP</td>
<td>2 to 8 million psi</td>
<td>More than five years</td>
</tr>
<tr>
<td>Concrete with significant &quot;D&quot; cracking</td>
<td>500,000 to 3 million psi</td>
<td>Three to five years</td>
</tr>
<tr>
<td>Concrete with severe &quot;D&quot; cracking</td>
<td>50,000 to 500,000 psi</td>
<td>Less than two years</td>
</tr>
</tbody>
</table>

The modulus of rupture values used to develop Equation 26 were estimated from the indirect tensile strength \( (\sigma_t) \) of PCC cores using Equation 27, developed by Hammitt [70]:

\[
S'_{c} = 1.02 \sigma_t + 210
\]  

(27)

The backcalculated PCC elastic modulus may also be used to estimate the PCC strength in an AC-overlaid jointed pavement. However, unusually low values may be obtained at some basins if the underlying slab is cracked within the deflection basin, even if a reflection crack is not visible at the AC surface.

For CRC pavement, it is not advisable to estimate the PCC modulus of rupture from the backcalculated elastic modulus, because of the likelihood of shrinkage cracks within the deflection basin. Cores taken from CRCP may have much higher indirect tensile strengths, and thus higher flexural strengths, than the backcalculated \( E_{pc} \) would suggest. This is even more true for AC/CRC pavements, since the AC obscures viewing of cracks in the CRCP.
The backcalculated PCC modulus may also be inconsistent with the strength of PCC cores if the pavement has "D" cracking. Relatively good concrete modulus values (e.g., 2 to 3 million) may be backcalculated on "D"-cracked pavement from which it is difficult to obtain complete cores for testing. Conversely, strong cores may be obtained in some areas of a pavement which is severely "D"-cracked in other areas.

These inconsistencies raise the question of which parameter, the elastic modulus or the modulus of rupture, is the better measure of the pavement’s structural capacity. It is better to think of the two as different measures, neither of which is better. The backcalculated elastic modulus represents of the slab stiffness within a radius of several feet of an applied load. This stiffness depends not only on the strength of the concrete material but also on the homogeneity of the concrete, the contribution of the reinforcing steel, and the presence of visible cracks and microcracks, caused by shrinkage, fatigue, and durability (freeze-thaw or reactive aggregate) deterioration. The modulus of rupture represents the concrete strength at the location of the core, and is not dependent on these other factors, with the exception of microcracking. It should not be surprising, therefore, that in some cases the two measures correlate well, while in other cases they do not.

8.2 VARIABILITY IN PCC AND FOUNDATION MODULI

Backcalculated PCC and foundation moduli are not adequately described by their mean values alone, since these may vary considerably over the length of a project. It is important to have a sense of how much variability in backcalculated moduli is typical in order to recognize when a pavement exhibits unusually high variability. The magnitude of variability is expressed by the coefficient of variation, which is the standard deviation of the values obtained as a percentage of the mean.

Caution should be exercised in ascribing significance to differences in PCC moduli or foundation moduli observed for different sections of a project, when in fact, due to the magnitude of variation, such differences may not be statistically significant. The topic of variability in backcalculation results for different pavement types and conditions deserves further study.

Several examples are presented here to illustrate typical results for new and old jointed and continuously reinforced pavements.

The first example is a new 10-inch JPCP airport pavement section, which was deflection tested before it was opened to traffic. [71] The dense liquid and elastic solid closed-form backcalculation method described in this report was used to determine the foundation k-value and dense liquid $E_{pcr}$. The results shown in Table 4 were obtained from analysis of some 150 deflection basins.
Table 4. Example backcalculation results for new airport JPCP.

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Subgrade k (psi/in)</th>
<th>Concrete Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>284</td>
<td>4,225,000</td>
</tr>
<tr>
<td>St deviation</td>
<td>24</td>
<td>617,000</td>
</tr>
<tr>
<td>COV, percent</td>
<td>9</td>
<td>15</td>
</tr>
</tbody>
</table>

The second example is a new 10-inch CRCP section on I-57 near Effingham, Illinois. The original 10-inch JRCP constructed in 1964 was recycled and reconstructed as CRCP in 1987. Analysis of deflection data collected in 1990 yielded the results shown in Table 5.

Table 5. Example backcalculation results for new CRCP.

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Subgrade k (psi/in)</th>
<th>Concrete Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>317</td>
<td>4,711,000</td>
</tr>
<tr>
<td>St deviation</td>
<td>77</td>
<td>1,470,000</td>
</tr>
<tr>
<td>COV, percent</td>
<td>24</td>
<td>31</td>
</tr>
</tbody>
</table>

While the mean foundation and slab moduli for this CRCP are similar to those of the new JPCP section used in the first example, the coefficient of variation of the backcalculated PCC modulus values are higher. The pavement has no "D" cracking or other significant distress.

The third example is an old and "D"-cracked 8-inch CRCP section on I-74 near Farmer City, Illinois. The pavement was built in 1971 and overlaid in 1985. Table 6 shows the backcalculation results obtained from deflections measured on the bare CRCP prior to overlay.
Table 6. Example backcalculation results for "D"-cracked CRCP.

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Subgrade k (psi/in)</th>
<th>Concrete Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>247</td>
<td>5,905,000</td>
</tr>
<tr>
<td>St deviation</td>
<td>107</td>
<td>3,167,000</td>
</tr>
<tr>
<td>COV, percent</td>
<td>44</td>
<td>54</td>
</tr>
</tbody>
</table>

This pavement's backcalculated moduli are even more variable than those of the new CRCP described in the second example. This magnitude of variability, while somewhat alarming, is not unusual for old and deteriorated CRCP. Roman and Darter [72] reported similar results in an analysis of a deteriorated section of I-77 in South Carolina. The backcalculated $E_{pc}$ values in several test sections analyzed had coefficients of variation ranging from 43 to 89 percent, and the test section k-values had coefficients of variation ranging from 36 to 58 percent. Interestingly, the coefficients of variation of k and $E_{pc}$ were not correlated to the amount of distress observed on the I-77 project. Test sections in very good condition, with relatively few failures, were just as highly variable in moduli as test sections in very poor condition. In contrast, a report on the performance of the I-57 Manteno CRCP project [73] found that the coefficient of variation of deflections was strongly correlated to the occurrence of medium- and high-severity distress in different sections of the project. This subject deserves further study.

The last example is the AC/CRCP case study on I-74 at Mansfield, Illinois. The backcalculation results are shown in Table 7. The CRCP is severely deteriorated due to "D" cracking. The coefficients of variation in backcalculated values are at least as high as those of the previous examples of old CRCP.

Table 7. Example backcalculation results for AC/CRCP with severe "D" cracking.

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Subgrade k (psi/in)</th>
<th>Concrete Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>373</td>
<td>3,868,000</td>
</tr>
<tr>
<td>St deviation</td>
<td>138</td>
<td>2,947,000</td>
</tr>
<tr>
<td>COV, percent</td>
<td>37</td>
<td>76</td>
</tr>
</tbody>
</table>
These few examples are not intended to define acceptable levels of variability for different types or ages of pavement, but rather to illustrate typical variability in backcalculation results. The topic of variability in backcalculation results for different pavement types and conditions deserves further study.

8.3 FREQUENCY DISTRIBUTION OF MODULI

A cumulative frequency distribution of concrete modulus values is a useful means of determining the median value, which may be different than the mean value, particularly if some values are unusually high. The cumulative frequency distribution also illustrates the percentage of deflection basins with moduli below a critical low level.

Figure 28 shows the cumulative frequency distribution of concrete moduli for some 170 deflection basins tested in the eastbound outer lane of I-74 near Mansfield. Modulus values greater than 9 million (most likely corresponding to locations of PCC patches greater than 7 inches thick) are not shown. The AC-overlaid 7-inch CRCP has extensive "D" cracking. The frequency graph shows that the median $E_{\text{pcc}}$ value is about 3.15 million psi, which is lower than the mean value of 3.87 million psi.

![Cumulative frequency, percent](image)

**Figure 28.** Comparison of PCC modulus frequency distributions.
The frequency distribution also shows that $E_{pc}$ was below 2 million psi for approximately 28 percent of the basins tested, below 1.5 million psi for 15 percent of the basins tested, and below 1 million psi for 7 percent of the basins tested. The percentage of the project area with $E_{pc}$ below each of these levels is certainly greater, since deflection testing was only done at locations with little or no surface distress.

The cumulative frequency distribution for concrete modulus values for the recently constructed 10-inch CRCP section on I-57 at Effingham, Illinois is also shown in Figure 28. The mean $E_{pc}$ value for this pavement was 4.71 million psi. This frequency distribution differs from the I-74 Mansfield distribution in several ways. The slope of the cumulative frequency line is steeper, indicating less variation in concrete modulus values. The median value is much higher: 4.85 million versus 3.15 million. Less than 5 percent of the values are less than 2 million psi, and none are less than 1.5 million psi.

8.4 TRENDS IN PCC AND FOUNDATION MODULI

A plot of PCC slab moduli and foundation moduli over the length of a project may reveal one or more significant shifts in the average modulus. As stated before, what constitutes a significant shift must be judged in light of the variability in moduli. Using the mean and standard deviation of modulus values for two apparently dissimilar sections of the project, a statistical t-test may be applied to determine whether the mean values are truly different. If they are, the two sections may need to be considered separately in designing rehabilitation for the project. Appendix JJ of the AASHTO Guide [74] also provides a statistical method for dividing a project into uniform sections using deflections or backcalculated moduli.

If deflections or backcalculated moduli differ markedly in one area from those in the rest of the project, the difference may be due to a change in the AC overlay thickness or PCC slab thickness. This occurred on at least one of the AC/PCC pavements studied. In a previous study of the I-57 Manteno project a dramatic drop in preoverlay deflections was observed for one 1000-foot section of the pavement. Subsequent conversations with IDOT District personnel revealed that the pavement in that section was actually 12-inch CRCP, rather than 8-inch CRCP with a 4-inch cement-aggregate mixture (CAM) subbase. [73]

A dramatic change in deflections or backcalculated moduli in a particular area of a project will usually coincide with a noticeable difference in the quantity and severity of distress in that area. Coring in the area may be warranted to determine whether a change in layer thicknesses or other cause is responsible for the difference.
8.5 DIFFERENCES IN MODULI BY DIRECTION

Nearly all of the AC/PCC pavement sections used as case studies showed differences in PCC moduli by direction which were sometimes minor but sometimes quite dramatic. Significant differences in slab moduli by direction usually coincided with significant differences in distress quantities and severities by direction, and differences in other condition measures (serviceability, roughness, etc.) as well. Foundation moduli were generally much more consistent by direction.

Figure 29 shows the PCC modulus frequency distributions for the I-74 Mansfield eastbound and westbound outer lanes. The eastbound distribution has a much lower median modulus and a larger percentage of low moduli. Patching records shed some light on the possible reason for the discrepancy seen for this project.

![Cumulative frequency, percent](image)

Figure 29. PCC moduli distributions for one project, two directions.
Before the first AC overlay was placed in 1983, the eastbound lanes had more distress (although the reason for this is unknown). About 2 percent of the eastbound traffic lane area was patched with AC prior to the 1983 overlay, while slightly less than 1 percent of the westbound area was patched. These percentages correspond to one AC patch about every 200 ft eastbound and every 412 ft westbound. The eastbound lanes continued to deteriorate more rapidly than the westbound lanes after the overlay, no doubt due in no small part to the continuity of the CRCP being disrupted by the closer spacing of AC patches. When the pavement was patched again in 1992 prior to a second AC overlay, the eastbound lanes again received more extensive patching. The total area patched in 1983 and 1992 was about 6 percent eastbound (an AC patch about every 67 ft), but only 2 percent westbound (an AC patch every 200 ft). Clearly, the two directions are in very different condition now, and even with more extensive patching, one must question whether the same overlay thickness is adequate for both directions. One must even question, for this particular example, whether the existing concrete, especially in the eastbound direction, can properly be considered a CRC slab anymore for purposes of overlay design.

It is generally inadvisable to base estimation of repair quantities and design of rehabilitation on deflection data collected for only one direction of a project, particularly when distress or other condition factors (e.g., serviceability or roughness) indicate a clear difference in pavement condition by direction. Only when all condition indicators show that the two directions are very similar and the time available for deflection testing is very limited should only one direction be tested.

8.6 UNUSUALLY HIGH OR LOW MODULUS VALUES

Sometimes individual deflections are measured which are much higher or lower than the average values in the area tested. Unusually high deflections and low PCC modulus values usually indicate that the PCC slab is severely deteriorated. This may be true even at locations with little or no distress visible at the surface of the AC. High deflections also occur at locations where the underlying PCC slab has a full-depth AC patch, although this is usually evident from reflection cracking at the surface.

Unusually low deflections and unreasonably high PCC modulus values are usually due to the presence of a PCC patch which is thicker than the original slab. Occasionally a maintenance crew replaces both the PCC thickness and the underlying base thickness with full-depth PCC when patching a pavement. For example, a 7-inch pavement may have 11-inch-thick PCC patches in some locations. These repair locations are often not known in advance of deflection testing. These repairs do represent an improvement in the pavement condition, and thus should not be ignored. However, it may be unwise to include the backcalculated moduli at repairs in the calculation of the mean PCC modulus for the project, since a few very high values can change the mean enough to give a misleading impression of pavement condition in unrepaired areas.
8.7 RELATIONSHIP OF PCC MODULUS TO CONDITION

The most difficult aspect of AC/PCC pavement structural evaluation, the one which requires the most experience and expert judgement, is the assessment of the overall "condition" of the PCC slab. This requires consideration of the backcalculated PCC moduli along with the type, quantity, and severity of visible distress. This is made particularly difficult by the fact that the PCC modulus results obtained depend on the way in which deflection testing is conducted.

One possible option for relating PCC moduli to distress is to conduct deflection testing in both cracked and uncracked areas. An uncracked area is defined for the purpose of this discussion as an area in the interior of a PCC slab (away from slab joints or edges) without linear cracks or localized failures within the deflection basin. If deflection basins are measured in both cracked and uncracked areas, PCC moduli backcalculated from these deflections should be considered "effective" moduli, which represent not the true stress-strain behavior of the PCC, but the condition of the slab in its current state of cracking.

An example of this approach is the work done by Rollings at the Waterways Experiment Station, in which a relationship was established between "E-ratio" (initial slab modulus versus cracked slab effective modulus) and Structural Condition Index (SCI, determined from cracking data). [75] Falling Weight Deflectometer deflections were measured on full-size slabs when intact, and at several subsequent stages of cracking. Although the actual moduli obtained by Rollings are dependent on the backcalculation method used, the deflection data and trends in backcalculated moduli demonstrate that a decrease in SCI is accompanied by a decrease in effective slab modulus.

Testing PCC or AC/PCC pavements in cracked areas poses several practical difficulties, however. No detailed guidance is available for how to conduct deflection testing in cracked areas, i.e., what testing interval should be used, how many locations should be tested, where to position the load plate and sensors with respect to cracks, how close to joints should testing be done, etc. The deflection and backcalculation results are likely to be highly variable depending on exactly how the testing is done. For example, three very different deflection basins will be obtained at one transverse crack, depending on whether the crack is positioned between \( d_{12} \) and \( d_0 \), between \( d_0 \) and \( d_{12} \), or at \( d_0 \) (directly beneath the load plate). The engineer is then faced with the difficulty of deciding which of the three PCC moduli backcalculated from these three basins best represents the condition of slab in that area.

The second approach, which is the approach taken in this study, is to measure deflections in uncracked areas only. The PCC moduli backcalculated from these deflection basins represent the condition of the slab in uncracked areas, separate from the
type, quantity, and severity of visible distress. In the field studies conducted for this research, deflections were only measured across cracks for the purpose of load transfer measurement, and these deflections were not used in backcalculation.

A disadvantage of this approach is that the engineer must consider the backcalculation results and distress survey results separately in assessing the "condition" of the slab. Testing in uncracked areas has some significant advantages, however, over testing in cracked and uncracked areas. It has the practical advantage that it is much easier for an engineer or FWD operator to identify and avoid cracked areas during testing than to make the many decisions described above concerning how to test in the cracked areas. Its other major advantage is that it is an excellent way to distinguish "D"-cracked PCC from sound PCC. This is particularly important in AC/PCC pavement structural evaluation, since the extent of "D" cracking deterioration in the PCC slab is very difficult to assess from visible distress alone.

The I-57 Manteno case study is an example of an AC/PCC pavement which does not have "D" cracking, but does have considerable load-related distress. The AC overlay has reflection cracks caused by unrepaired cracks in the CRCP slab and full-depth repair joints. In uncracked areas, however, the PCC slab modulus is high. The distress quantities and the median and mean dense liquid PCC moduli for the two test sections of the project in the worst condition are shown in Table 8. "Failures/mile" refers to the number of deteriorated reflection cracks, PCC repairs, and AC patches per mile.

Table 8. Distress versus $E_{pcc}$ for AC/CRCP without "D" cracking (I-57 Manteno).

<table>
<thead>
<tr>
<th>Test Section</th>
<th>Failures/mi</th>
<th>Median $E_{pcc}$ psi</th>
<th>Mean $E_{pcc}$ psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>58.2</td>
<td>5,500,000</td>
<td>6,700,000</td>
</tr>
<tr>
<td>D</td>
<td>50.3</td>
<td>5,000,000</td>
<td>6,200,000</td>
</tr>
</tbody>
</table>

If the PCC slab has "D" cracking or other severe deterioration (e.g., a cracked and seated or rubblized slab), low backcalculated PCC modulus values will be obtained for deflection basins even in uncracked areas. These low modulus values will be reflected in the low mean and median values obtained, and also in the cumulative frequency distribution of modulus values. One example of this is the I-74 Mansfield case study, a severely "D"-cracked CRCP slab with an AC overlay. The backcalculation results for this project were given in Table 7 and Figure 29. The project is characterized by low mean and median dense liquid PCC moduli (3.87 million psi and 3.15 million psi respectively), and large percentages of low PCC moduli (e.g., 28 percent below 2 million psi) for deflection basins in uncracked areas.
Another example of the relationship between backcalculated PCC modulus and "D" cracking is the I-70 Marshall case study. This pavement is also a severely "D"-cracked CRCP slab with an AC overlay. It is also characterized by low median and mean PCC moduli, and large percentages of low PCC moduli for deflection basins in uncracked areas. For example, in the westbound direction, the mean and median PCC moduli are 3.6 million psi and 2.4 million psi respectively, and 40 percent of the basins have PCC moduli below 2 million psi. At specific locations where high deflections were measured (and for which low PCC moduli were backcalculated assuming a slab thickness of 8 inches), coring confirmed that the PCC was moderately to severely deteriorated. At locations where very low PCC moduli were backcalculated (e.g., less than 200,000 psi), as little as 1.5 in of sound concrete was recovered by coring.

These examples demonstrate that extensive "D" cracking in an overlaid PCC pavement may be diagnosed from backcalculation results. Indicators that the PCC slab is deteriorated are low mean or median PCC moduli (e.g., less than about 5 million psi) or a high percentage of low moduli (e.g., 15 percent or more less than 2 million psi) obtained for deflection basins in uncracked areas. Further analysis should be done using data for additional projects to better establish critical levels for PCC modulus.

Deflection testing in both cracked and uncracked areas may become a more useful structural analysis technique in the future, if additional field studies are conducted and detailed guidelines are developed for conducting the testing and data analysis. For the present, deflection testing in uncracked areas is recommended since the testing and analysis do not require the level of expertise that testing in cracked areas requires. Interpretation of backcalculated PCC moduli is an important subject which certainly deserves further study.

8.8 INTERPRETATION OF LOAD TRANSFER RESULTS

The deflection load transfer across joints and cracks is often expressed by the deflection of the unloaded side of the joint or crack as a percentage of the deflection of the loaded side. The following equation for deflection load transfer has been proposed by Darter [76]:

\[
\Delta LT = 100 \times \left( \frac{\Delta u}{\Delta l} \right) \times B
\]  

(28)

where \( \Delta LT \) = deflection load transfer, percent
\( \Delta u \) = unloaded side deflection, inch
\( \Delta l \) = loaded side deflection, inch
\( B \) = slab bending correction factor
The correction factor, $B$, is necessary because the deflections $d_0$ and $d_{12}$, measured 12 inches apart, would not be equal even if measured in the interior of a slab. Use of a slab bending correction factor is typically used for load transfer calculations for bare PCC pavements (for example, in Reference 69). It is proposed that a comparable correction factor could be employed in load transfer calculations for AC/PCC pavements. This factor would account for both bending in the PCC slab and compression in the AC overlay. An appropriate value for the correction factor may be determined from the ratio of $d_0$ to $d_{12}$ for typical center slab deflection basin measurements, as shown by the following equation:

$$B = \frac{d_{0\; center}}{d_{12\; center}} \quad \text{(29)}$$

Deflection load transfer, expressed as a percentage, is considered important in evaluation of bare PCC pavement because of its relationship to stress load transfer across joints and cracks. However, for the purpose of predicting reflection crack occurrence and deterioration in AC overlays of PCC (and second AC overlays of AC/PCC), the actual magnitude of differential deflection is probably a more descriptive parameter than the percent load transfer.

Consider a joint in a PCC pavement which deflects 5 mils on the loaded side and 2.5 mils on the unloaded side. This joint has only 50 percent deflection load transfer. Then consider another joint which deflects 10 mils on the loaded side and 5 mils on the unloaded side. This joint also has 50 percent deflection load transfer. However, the differential deflection at is only 2.5 mils at the first joint and 5 mils at the second joint. It is reasonable to expect that a reflection crack in an AC overlay will propagate and deteriorate more quickly at the second joint than the first joint.

Analytical investigations of reflection cracking in AC overlays have to date focused almost entirely on the use of finite element analysis, fracture mechanics, or other methods to model reflection crack propagation. [77, 78, 79, 80, 81, 82, 83] Some of these studies have advocated the use of field measurements of load transfer at joints and cracks in PCC pavement to determine the shear strain which will be induced in an AC overlay.

These previous research efforts have concentrated on predicting reflection crack occurrence. The deterioration of reflection cracks from low to medium and high severities has hardly been addressed at all. As described previously in this chapter, it is reflection crack deterioration that is primarily responsible for loss of serviceability in AC/PCC pavement. Factors which are believed to be important in reflection crack deterioration include the magnitude of differential deflections at joints and cracks after overlay and the number and magnitude of applied loads. This is an important topic which deserves further study.
9.0 DRAINAGE EVALUATION FOR AC/PCC PAVEMENTS

Most Interstate PCC pavements in Illinois were originally constructed without drains, on poorly draining foundations, i.e., silty clay subgrades and dense-graded granular and dense-graded stabilized bases. As a result of these design factors, in combination with Illinois' wet climate, these pavements typically exhibit distress caused or accelerated by excess water in the pavement structures. Moisture-related distresses commonly observed on JRPC and CRCP in Illinois include "D" cracking, faulting at joints and cracks, punchouts, and pumping of water and fines at transverse and longitudinal joints and cracks. Poor subdrainage conditions are also a major contributor to deterioration of BAM bases.

IDOT's practice has been to install longitudinal subdrains in Interstate PCC pavements in conjunction with restoration or resurfacing. However, some of these pavements continue to manifest moisture-related distress after overlay, which indicates that the subdrainage system is either inadequate or is not functioning properly. The most obvious visible indications of excess moisture are pumping of water and/or fines at transverse and longitudinal joints, blowholes along the lane/shoulder joint, and localized settlement of the AC shoulder near blowholes. Deterioration of the PCC pavement due to "D" cracking, though not always visible at the AC surface, may also indicate insufficient drainage. A third major moisture-related problem in AC/PCC pavement is stripping in the AC surface, which may be investigated by visual examination of cores after splitting.

When an AC/PCC pavement is being evaluated for second rehabilitation, the adequacy of the subdrainage system in place should be evaluated, and if the system is not adequate, further subdrainage improvements should be considered in conjunction with the second rehabilitation. Pumping is a clear indication that the pavement requires a drainage improvement, regardless of whether or not a subdrainage system is in place. Absence of any moisture-related distress suggests that the existing system is functioning adequately, and no additional improvement is warranted.

An existing subdrainage system may be deficient for a number of reasons, including insufficient capacity (pipe size, pipe grade, and outlet spacing), clogging of the drains by fines, poorly designed or maintained outlets, or ditches of inadequate depth or grade. The causes of the drainage deficiency will dictate the nature of the subdrainage improvements required with the second rehabilitation. These factors should be identified in the evaluation phase, so that the same deficiencies are not built into the second subdrainage improvement. References 32 and 84 provide additional guidance on pavement subdrainage evaluation and subdrainage improvement.
10.0 AC MATERIAL EVALUATION

AC/PCC pavement may exhibit some particular problems which are due to the quality of the AC surface layer, independent of the condition of the underlying PCC slab. The most significant of these problems are premature rutting, weathering, ravelling, and stripping. If any of these problems exist, they should be identified in the pavement evaluation, since they will have a major influence on the appropriate treatment of the existing overlay in the second rehabilitation effort.

10.1 PREMATURE RUTTING

Premature rutting may occur when an unstable AC mix is used to overlay a PCC pavement. It also may occur when bond is lost at the AC/PCC interface, which increases the shear stress experienced in the AC and therefore greatly increases the likelihood of premature rutting in the mix. This problem is not limited in Illinois to pre-1984 mixes, although the mix design changes made at that time have made it a less frequent occurrence. Visible indications of premature rutting are deep wheelpath ruts in an AC overlay less than ten years old, cracking in the wheelpaths which resembles alligator cracking, and pronounced lateral shoving of the AC.

Figure 30 illustrates premature rutting in an AC/PCC pavement on I-57 near Mt. Vernon (milepost 95). The original 8-inch CRCP was overlaid with 3.25 inches of AC in 1986. After six years in service and approximately 1.7 million accumulated ESALs, the overlay is already exhibiting severe rutting, as well as lateral shoving and alligator cracking in the wheelpaths. It was clearly evident from observations in areas of the project where milling had been done that no bond existed between the AC overlay and PCC slab, as illustrated by Figure 31.

Cores obtained from an AC overlay which appears to be developing premature rutting may be examined in the laboratory for mix deficiencies which may be responsible for the instability. A high fines content may be a factor, since excessive fines occupy void space in the mix which is then unavailable to the asphalt cement. Another factor may be an aggregate gradation which does not follow fairly close to the maximum density curve for bituminous mixes, i.e., a straight-line relationship between percent passing and sieve sizes raised to the 0.45 power.

10.2 WEATHERING/RAVELLING AND STRIPPING

Weathering and ravelling occur as a result of loss of bond between asphalt cement and aggregate. These problems may be limited to the surface of the AC, as a result of oxidation of the asphalt cement. They may, however, be indicative of loss of asphalt-aggregate bond (i.e., stripping) through the full depth of the AC. This may be determined by taking cores from the AC surface and splitting them apart. If uncoated aggregates are visible, stripping is occurring. It is emphasized that the interiors of split cores should be examined for stripping, rather than the circumference of the core, since the core's outer surface may be significantly damaged by the coring drill.
Figure 30  Premature rutting in AC/PCC pavement.

Figure 31  Loss of bond at AC/PCC interface evident after milling.
11.0 CONCLUSIONS

Two types of AC/PCC pavement evaluation are addressed in this report. The first type is selection of AC/PCC pavements for rehabilitation, which is a network-level pavement management activity. The second type is project-level evaluation of AC/PCC pavements, which involves distress surveying, nondestructive deflection testing, coring and materials testing. A thorough project-level evaluation includes a functional evaluation, a structural evaluation, a drainage evaluation, and an evaluation of the AC surface mix.

11.1 SELECTION FOR MULTI-YEAR REHABILITATION PROGRAM

The key condition indicators available (distress, roughness, serviceability, subjective ratings, and rutting) were examined to assess their usefulness to AC/PCC pavement rehabilitation project selection. Based on the results of the analyses, critical levels for each of the condition indicators examined were identified for use in selecting AC/PCC pavement sections for rehabilitation.

11.2 DISTRESS, ROUGHNESS, AND SERVICEABILITY

The dominant factor used by IDOT in targeting project for rehabilitation is the CRS. A predictive model is presented for AC/PCC pavements which may be used to determine the "remaining life" of the pavement in terms of years for CRS to reach a specified critical level.

CRS was found to correlate fairly well with ride quality, but have not yet been shown to correlate well with distress or measured roughness. A model was presented for estimation of Present Serviceability Rating (PSR) for AC/JRCP and AC/CRCP from International Roughness Index (IRI), which may be computed from pavement profile measurements. A critical IRI level of 110 inches per mile was identified as corresponding to a PSR level of 3.0. A model was also developed to predict PSR from distress (medium- and high-severity reflection cracks, patches, and failures) for AC/JRCP and AC/CRCP. A critical level of 25 medium-high reflection cracks, patches, and failures per mile was identified as corresponding to a PSR level of 3.0.

11.3 REFLECTION CRACK DETERIORATION

The progression of reflection cracks, patches, and failures from low to medium and high severities was shown to follow a fairly predictable progression for AC/JRCP, and in one case study for which detailed preoverlay condition and repair data were available, for AC/CRCP as well. In general, however, deterioration of reflected distress in AC/CRCP is more difficult to predict without preoverlay condition and repair data.
11.4 RUTTING

A rut depth between a third and a half of an inch is typically considered unsafe in terms of hydroplaning potential, vehicle handling difficulty, and wheel spray. The mean of two manual rut depth measurements per mile was shown to correlate poorly with the mean of more than two thousand rut depth measurements per mile obtained using the South Dakota Profiler. The high degree of variability in the manual rut depth measurements may be the primary cause of this poor correlation. This finding has less impact on the use of either method for rehabilitation project selection than on the use of rut measurements to develop rutting prediction models for AC/PCC pavements.

11.5 FUNCTIONAL EVALUATION

An AC/PCC pavement may be functionally deficient because of poor surface friction, excessive rutting, or excessive roughness. Excessive rutting may be due to lateral distortion of an AC surface which develops normally over several years, or it may be due to mix instability which manifests itself in just a few years. The major causes of excessive roughness in AC/PCC pavement are reflection cracking (including reflection of deteriorated patches and localized failures), premature rutting, or ravelling.

11.6 STRUCTURAL EVALUATION

Detailed procedures were developed in this study for conducting a structural evaluation of an AC/PCC pavement, including distress surveying, nondestructive deflection testing, coring, and materials testing. A thorough structural evaluation requires investigation of the condition and load-carrying contribution of each of the layers of the pavement system: the AC surface, the PCC slab, the base, and the subgrade. The results of the structural evaluation are crucial to decisions which must be made for second rehabilitation, including:

1. Division of the project into uniform sections,
2. Identification of areas requiring repair,
3. Determination of whether or not the pavement requires a structural improvement,
4. Selection of an appropriate overlay type, and
5. Second overlay design.

Many of the AC/PCC pavements evaluated in the field studies for this research exhibited significant and in some cases quite dramatic differences in distress, deflection, and backcalculated moduli by direction. A thorough project-level evaluation should include distress surveying and deflection testing in both directions of a project, at sufficient sampling frequencies to accurately assess the overall condition of the pavement.
11.7 BACKCALCULATION OF AC/PCC PAVEMENT LAYER MODULI

A simple and straightforward procedure for backcalculation of AC/PCC pavement layer moduli was developed in this study. The procedure relies on knowledge of the AC surface modulus based on AC mix temperature monitored during deflection testing. Resilient modulus testing of AC cores from the actual AC/PCC pavement being evaluated is recommended to establish a relationship between AC modulus and temperature. When resilient modulus testing is not feasible, the AC modulus may be estimated as a function of temperature and mix parameters. The relationship between AC modulus and temperature, adjusted for the difference between laboratory testing frequency and FWD testing frequency, is used to assign an AC modulus to each deflection basin. The AC modulus and AC thickness are used to determine the compression which occurs in the AC layer under the load plate, and thereby adjust the deflection basin measured at the AC surface in order to determine the deflection basin induced in the PCC layer.

Assignment of the AC modulus based on mix temperature at the time of deflection testing is necessary to avoid backcalculating incorrect values for the PCC modulus and thus misinterpreting the condition of the PCC. Other researchers have noted that the solutions to multilayer backcalculation problems are not unique. Thus, it is possible to obtain good matches between measured and predicted deflections for AC modulus values which are clearly inconsistent with the conditions existing at the time of deflection testing.

Available closed-form backcalculation solutions for bare PCC pavements were demonstrated to be applicable to AC/PCC pavement backcalculation, if the necessary adjustments were made to account for compression in the AC surface. In analysis of a large sample of actual FWD deflection data from an in-service AC/PCC pavement, this backcalculation approach produced results which correlated well with results obtained using a multilayer elastic backcalculation program believed to be highly reliable. In case studies, the AC/PCC backcalculation procedure demonstrated its ability to produce results consistent with the observed distress and the condition of cores. The only exceptions to this occurred when the PCC slab thickness differed significantly from the thickness assumed in backcalculation. The major advantages of the AC/PCC backcalculation procedure developed in this study over other backcalculation routines are its greater ease of use, its independence of seed moduli and other user-controlled inputs, and its ability to process large sets of deflection data much more quickly than the existing backcalculation routines.

Backcalculation, by this method or by any other reliable method, can yield the following useful results:

1. Average PCC slab and foundation moduli of project,
2. Variability in slab and foundation moduli,
3. Percentage of values below a selected critical level,
4. Trends in slab and foundation moduli over the length of the project,
5. Significant differences in slab and foundation moduli by direction,
6. Identification of specific areas with unusually high or low moduli, and
7. Relationship of PCC and foundation moduli to quantity and severity of distress.

11.8 LOAD TRANSFER EVALUATION

The magnitude of differential deflection across reflection cracks in AC/PCC pavements is proposed as a more influential factor in reflection crack deterioration than percent load transfer. Research efforts to study reflection cracking in AC/PCC pavements have to date focused almost exclusively on predicting the occurrence of low-severity reflection cracks, rather than predicting the deterioration of these cracks to medium and high severities. This capability is of more practical importance, since it is reflection crack deterioration that relates directly to loss of serviceability and increasing maintenance costs for AC/PCC pavements.

11.9 DRAINAGE EVALUATION

Most if not all of the AC/PCC pavements included in the field studies for this research were not originally constructed with subdrains, but had subdrains installed in conjunction with construction of the AC overlay. In some cases, these subdrainage systems have functioned well and the AC/PCC pavements have exhibited no pumping or other signs of drainage deficiency. This was true even for one case study in which the original PCC pavement had exhibited severe pumping prior to rehabilitation.

However, some other AC/PCC pavements exhibited significant pumping, which is a clear indication of a drainage deficiency. This may be due to a poorly designed or poorly functioning drainage system. A drainage evaluation is recommended for any AC/PCC pavement which exhibits pumping or other moisture-related distress, even if construction records indicate a drainage system is already in place.

11.10 AC MATERIAL EVALUATION

AC/PCC pavement distresses associated primarily with the quality of the AC mix are premature rutting, weathering/ravelling, and stripping. These problems may be difficult to diagnose these problems from visual observations alone. Examination of cores is recommended whenever visual observations suggest that one or more of these problems may exist. Complete removal of the AC surface may be warranted.
REFERENCES


70. Hammitt, G. M. II, "Concrete Strength Relationships," U. S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 1971.


APPENDIX

LITERATURE REVIEW OF NEW TECHNOLOGIES FOR NONDESTRUCTIVE PAVEMENT EVALUATION

INTRODUCTION

Nondestructive testing techniques are most useful in evaluation of pavement condition if they can be performed rapidly and the results could be used to obtain information on current pavement material properties, and interface conditions. The availability of techniques providing detailed and accurate results is important to many aspects of pavement analysis and rehabilitation design. These techniques should be efficient, repeatable, safe to use, and cost-effective.

Deflection testing has been used extensively. Other techniques include ground-penetrating radar (GPR) and infrared thermography which have been applied to the evaluation of pavements. They are used to investigate subsurface properties including pavement layer thicknesses, joint deterioration, voids, and base moisture contents. Likewise, delaminations in steel-reinforced concrete pavements and debonding at asphalt-concrete interfaces may be located using these techniques.

Illinois Highway Research (IHR) Project-532 is an investigation into the rehabilitation of asphalt-overlaid Interstate pavements in Illinois. This group of pavements presents specific problems to the application of nondestructive testing techniques such as GPR and infrared thermography. For instance, their ability to function over pavements with an existing asphalt overlay and pavements containing reinforcing steel are two priorities for this project. Also, determining the extent of durability cracking ("D" cracking), which is a severe problem for many Illinois pavements, is of major importance.

Descriptions of these devices, applications to pavement evaluation, advantages and disadvantages of each technique, and a discussion as they pertain to IHR-532 are presented.

GROUND-PENETRATING RADAR

Capabilities

Ground-penetrating radar’s (GPR) capabilities include prediction of pavement layer thicknesses and subsurface conditions such as joint deterioration and voids. GPR has been tested on asphalt, concrete, and composite pavements and bridge decks. In addition, GPR has been used to predict moisture contents in a variety of base layer materials.
Principles and Methods of Operation

Short-pulse, ground-penetrating radar functions according to the theory of wave propagation and dielectric properties in the pavement materials. A brief pulse of electromagnetic energy is directed into the pavement. Dielectric discontinuities in the pavement layers (such as asphalt-concrete or concrete-base interfaces) cause the reflected waves to vary in amplitude and frequency, as shown in Figure 1. This information is collected by recording devices at the surface, and later analyzed to determine pavement properties (layer thicknesses, voids, etc.). Several vendors have equipped test vans complete with radar systems, transducers, antennas, and on-board recording devices such as magnetic tapes, oscilloscopes, and computer hardware and software. These vans operate at speeds ranging from 5 to 40 mph and require only a single lane, moving closure. Usually, a single operator can perform all necessary vehicle and equipment-related activities needed for testing.

Case Studies

Layer Thicknesses and Base Moisture Content. The most actively tested capability of GPR is that of pavement layer thickness determination. A recent study in Kansas [1] investigated the use of GPR in predicting layer thicknesses over a variety of pavement types obtaining accuracy of +/- 5 to 10 percent. A previous study in Texas [2] obtaining similar results is described.

In the summer of 1990 GPR was tested on four, 1500-foot SHRP sections in Texas. These in-service sections were full-depth asphalt pavements, ranging in thickness from 1 to 9 inches. Full-depth AC pavements were chosen since they present the greatest variation in asphalt thicknesses. A test van covered the sections in speeds ranging from 5 to 40 mph. The radar data were recorded by an on-board computer and later analyzed using customized software. Following completion of test runs, coring and penetrometer tests were performed at specific locations to verify the recorded data (Figure 2).

The results of this testing predicted asphalt thicknesses to within +/- 0.32 inches of actual values when just the radar data alone were used. When one calibration core was used per site, accuracy was improved to +/- 0.11 inches. Furthermore, base thicknesses ranging from 6 to 10 inches were predicted to within +/- 0.71 inches. Under previous procedures based on core samples and prior records, asphalt thicknesses would have been predicted with up to +/- 1.5 inches of error. When combined with Falling Weight Deflectometer (FWD) data, these variations would have resulted in up to 100 percent errors in the prediction of base layer moduli.

A second property researched in this project was the moisture content of the base layer, and its variation with time and location. Previously, the only common method for predicting this property was through the use of dry coring. In addition to the initial radar tests, a second set of runs was made one month later.
Position Along Deck ------>
(one waveform/foot)

A  Top of Asphalt
C  Asphalt/Concrete Interface
   Top Rebar
   Bottom Rebar
   Bottom of Deck

time (ns.)

Figure 1. Interpretation of waveforms measured by GPR.
FIGURE 5
TYPICAL DATA for a 1500 ft. SURVEY

SHPD ID A83559
LTPP Project Data:
Asphalt Thickness 8"
Base Thickness 6"
Base Type Bit. Treated Soil
Location:
SH 30
Texas

- ground truth data points

Figure 2. Layer thickness measurement using GPR.
The predicted base moisture contents agreed with measurements of actual moisture contents within 2 percent by weight. Spatial and temporal variation in moisture content were profiled by superimposing the individual runs (Figure 3).

![Base Moisture Content](image)

Figure 3. Base moisture content measurement by GPR.

*Joint Deterioration and Voids.* A case study was done by Donohue and Associates of Milwaukee, Wisconsin, using GPR to detect subsurface joint deterioration and voids in concrete pavements [3]. Test runs of 11 miles each were made to investigate these distresses.

Testing for joint deterioration was carried out over two days using van-mounted equipment. A 2-inch loss in cross-section for an 8-inch pavement was set as the criteria for a joint to be defined as deteriorated. Radar waveforms were generated, recorded on magnetic tape and analyzed to determine which joints were deteriorated (Figure 4).

![Deteriorated Joints](image)

Figure 4. Deteriorated joints identified by GPR.
Following the collection of radar data, a series of cores were taken for the purposes of calibration of the test equipment and verification of test results. The cores showed the test results to be 80 percent accurate. Of all joints investigated, less than 2 percent showed any visual signs of deterioration. Since the original Donohue test run was performed, equipment has been developed which automatically analyzes data and calculates deterioration to within 0.5 inches. The improved equipment speeds up the survey process, and improves accuracy to about 90 percent.

The detection of voids in subgrade material beneath concrete pavements was carried out in much the same manner as for joint deterioration. A test van operated at speeds of 2 mph over sections of pavement characterized by faulting at joints, settlement of entire pavement sections, and pavement cracking. Voids ranging in size from 0.25 inches to 2.0 inches in depth were identified by the radar. Again, following the test run a series of cores were taken for calibration and verification purposes. Of a total twenty-seven locations identified by the GPR as void locations, cores showed all except eight as having voids. These eight showed serious honeycombing, another sign of future pavement distress.

**Freeze/Thaw Damage.** A difficult determination for GPR is the extent and severity of durability problems in a pavement such as those due to freeze/thaw damage (ie. D-cracking). A 1988 National Cooperative Highway Research Program (NCHRP) report listed a severely "D"-cracked section of Interstate 74 in Illinois as a field study for the testing of radar and video imaging on pavements [4]. Examples of the severity of D-cracking existing in this pavement are shown (Figure 5). The testing provided mixed results on the effectiveness of GPR on this type of pavement. Severe locations of D-cracking were easily detected, however the extent and location of less severe cracking was not distinguishable. Also, the presence of steel dowel bars at transverse joints (a common location of "D" cracking) interfered with the performance of the radar.

![Figure 5](image)

Figure 5. Freeze-thaw damage at concrete pavement joint.
More recent and extensive studies of GPR on freeze/thaw damaged bridge decks have been presented by INFRASENSE, Inc. of Cambridge, Massachusetts. One paper describes the use of radar to detect "punky concrete" in asphalt-overlaid bridge decks in five New England states [5]. A second paper provides details of an evaluation program developed for the State of New Hampshire using this technology at the "network" level [6]. "Punky concrete" is a common problem with bridge decks in New England states and is a result of wet freeze/thaw conditions and a high chloride content. This is different than "D" cracking; however, both distresses manifest themselves in similar fashion (disintegration of concrete, retained moisture and salt, etc.) and therefore are detected by GPR in a similar manner. The radar detects areas of concrete exhibiting higher than normal dielectric constants due to the presence of voids, moisture and salt (Figure 6), and generates a plan view showing the extent of deteriorated areas (Figure 7).

Figure 6. Concrete bridge deck deterioration measured by GPR.

Figure 7. Plan view of bridge deck deterioration measured by GPR.
Of 44 bridge decks surveyed in the New Hampshire study, 19 were evaluated during rehabilitation for comparison with the radar results. The results showed to be accurate to +/- 4.4 percent of the total surface area. When used at the network level to place decks into one of four condition categories, the radar resulted in appropriate classification 93 percent of the time.

Advantages and Disadvantages

The major advantages of GPR are its speed and accuracy in providing detailed pavement information. The moving nature of the test runs provide continuous, detailed information on subsurface pavement conditions. The radar is able to operate on a variety of pavement types and is very adept at distinguishing differing pavement materials. A disadvantage of GPR is the complexity of the radar output. Current data analysis methods are labor intensive and require expert knowledge for interpretation of raw data. Also, some coring is still required with GPR for calibration purposes.

INFRARED THERMOGRAPHY

Capabilities

Infrared thermography has been tested extensively in its ability to locate steel to concrete delaminations in reinforced concrete pavements. Operating on the same principles, infrared has also been used to detect debonding at asphalt/concrete interfaces. Developed initially for its application to evaluating bridge decks, infrared thermography has been used to survey pavements as well.

Principles and Methods of Operation

Infrared thermography is the process of detecting temperature differences associated with defective areas within a pavement. Various types of infrared scanners have been used to detect both delamination and debonding in a pavement. The scanning equipment can be van-mounted and operated at speeds of 15 mph. Temperature differences indicative of defects, such as a thin delamination heating faster than the thicker, sound pavement around it, are detected by the scanners and recorded on videotape. Often, real-image video recording equipment is mounted in conjunction with the scanner to record possible surface flaws such as patches and potholes which may be interpreted incorrectly when viewed on the infrared output. Recorded data are then processed and presented on a plan-type drawing.

Case Studies

Steel-to-Concrete Delaminations. In 1983, infrared thermography was used to survey 18.5 lane miles of the Dan Ryan Expressway in Chicago, Illinois [3]. This heavily trafficked expressway was surveyed in five days using a test van equipped with infrared scanners and video recorders. Both infrared and real-life images were recorded and later analyzed to detect delaminated areas in the pavement. This information was then entered into a CADD mapping system to be used as part of the rehabilitation process.
The sections of bridge deck identified in need of repair were cross-referenced with an earlier underside inspection of the decks. Based on these two evaluations, sections of deck were scheduled for either full-depth repair or partial-depth repair depending on the severity of distress. In this study the infrared devices were able to detect delaminated areas as small as 4 inches in diameter, and survey the expressway a lane at a time. Cores were taken for verification purposes, and the infrared results were shown to be accurate.

A second case study using infrared thermography was also performed by Donohue and Associates on 79 lane miles of concrete pavements [3]. These included continuously reinforced concrete pavement (CRCP), asphalt-overlaid jointed reinforced concrete pavement (AC/JRCP), and a section of CRCP that was cathodically protected. The survey lasted eight days and again, delamination of steel to concrete was the distress investigated.

As in the previous case, infrared and real-life images were recorded and analyzed. Delaminated areas in need of repair were mapped along with the locations of asphalt and concrete patches. Cores were taken for verification and infrared results were shown to be 99 percent accurate. Using this information, repair expenditures were estimated, and the effectiveness of the previous cathodic protection system was evaluated.

Advantages and Disadvantages

The major advantages of infrared thermography are its speed and accuracy for subsurface data collection. Case studies have shown large quantities of data to be collected in a comparatively short period of time. Output is continuous and over an entire area, not just point data. The testing method is completely nondestructive and is relatively unaffected by otherwise troubling conditions such as surrounding traffic, buried utilities and pavement material type. Disadvantages of the testing method include its sensitivity to non-pavement related conditions such as time of day and recent weather conditions. Also, since the infrared output is two-dimensional it is not possible to directly predict the depth of a distressed area. Perhaps the largest practical disadvantage of infrared, however, is the complexity of the infrared outputs and video images. Data interpretation is very tedious and requires considerable experience.

OTHER TECHNIQUES

Wave Propagation/Spectral Analysis

The current state-of-the-art use of wave propagation as a method of pavement evaluation has been described in a recent study performed for the U.S. Air Force [7]. In this technique, the dispersion of Rayleigh waves in a pavement are monitored to predict pavement condition. Rayleigh waves are also known as surface waves, and dispersion refers to the change in wave velocity with frequency or wavelength. In a layered system, the dispersion of Rayleigh waves is indicative of the relative conditions of distinct layers.
Three common methods exist for the production of Rayleigh waves in a pavement. They are drop weight devices, vibratory devices, and strike hammers. The third of these devices, strike hammers, has been employed in a technique known as spectral analysis of surface waves (SASW). In this technique a series of progressively larger hammers are used to produce waves of increasing wavelength, which tend to propagate through the deeper layers of a pavement. The majority of recent pavement evaluation performed using wave propagation has centered on the spectral analysis technique.

Spectral analysis is performed using a strike hammer, transducers, spectral analyzer, and microcomputer (Figure 8). Waves are generated in the pavement by the strike hammer and their dispersions are monitored by two transducers acting as receivers (Figure 9). The spectral analyzer acquires data which the microcomputer analyzes. The data are displayed as phase shift, known to be a function of frequency, and therefore, are a measure of Rayleigh wave velocity. Through an inversion process the data can be displayed as a plot of modulus versus depth.

![Diagram of equipment used in spectral analysis of surface waves.](image)

Figure 8. Equipment used in spectral analysis of surface waves.
Figure 9. Surface wave propagation in pavement layers.

The SASW technique was field-tested on a variety of airfield pavements in a study performed at Mississippi State University in 1988 [8]. Test sites included flexible, rigid and composite pavements over coarse-grained and fine-grained subgrades. Results of the SASW analysis were compared with results of deflection based backcalculations and laboratory tests. The conclusion of this report was that SASW provided a viable method for the determination of layer moduli in multi-layered systems. However, the report did state the need for further automation of the testing and analysis techniques in order to make this technology practical for use other than research.

The use of spectral analysis has the advantage of predicting pavement layer moduli without needing to know beforehand layer thicknesses or material type. This is different from programs which require inputs such as layer thickness in order to backcalculate moduli. A disadvantage of current SASW technology is the difficulty and time required for data reduction and interpretation. Automated methods for acquiring data and analyzing results are needed to make it a practical application.
Sonic/Ultrasonic/Seismic Wave Analysis

The use of sonic, ultrasonic, and seismic waves to evaluate internal conditions of concrete has also been applied to pavements. In this technique a source emits stress waves into the surface of a pavement while very precise sensors record direct or refracted wave characteristics. Compression and shear wave values are used to determine modulus and strength. Horizontal and vertical seismic waves are used to detect voids beneath a concrete layer. Furthermore, analysis of the reflected wave data can provide pavement layer thicknesses, delaminations and coupling characteristics.

Sonic/ultrasonic/seismic testing is performed by initiating stress waves in the pavement surface by means of a wave source such as a transducer or high-velocity/low-mass projectile. Sensors placed at selected positions along the pavement surface record sonic/seismic energy travelling through the concrete. The sensors in turn send an electrical signal to recording instrumentation where it is magnified and stored on tape or disk. The raw data are then reduced and analyzed to determine the pertinent pavement characteristics.

An NDT device utilizing sonic/ultrasonic/seismic wave technology has been developed and tested by Weston Geophysical, Westboro, Massachusetts [9]. This device shoots a steel pellet at the pavement at high velocity as the device is pushed along its path. A test van trails the operator and records the reflected data. One case study performed by Weston using this device was on the Broadway Viaduct, Boston, Massachusetts. The device was used to survey the entire 3000-foot section and determine which areas were in greatest need of repair. Properties investigated were concrete strength and steel to concrete delaminations. Maps containing spatial representations of these distresses were then produced and used by the agency to prepare a maintenance scheme (Figure 10).

The system developed by Weston has the advantage of providing continuous, detailed information collected over an area, not just a single point. Also, the system appears to be capable of detecting pavement properties and presenting them in such a manner that the information is very practical and useful for the contracting agency. However, little information is provided on the limitations of the device such as accuracy, precision, and sensitivity to material type and environmental conditions. More case studies are needed.

DISCUSSION OF NEW NDT TECHNOLOGIES AS PERTAINING TO IHR-532

Several new nondestructive testing techniques are currently being developed for the specific purpose of pavement evaluation. While it is clear a wide variety of pavement properties can now be reasonably determined using these techniques, it is important to keep in mind how well the specific needs of IHR-532 could be met through their use. Unfortunately, when searching for research performed on field conditions specific to this project (composite pavements, D-cracking, etc.) little documentation is found. However, several case studies in similar areas have shown the potential for existing technology to be adapted to the specific conditions of IHR-532 if a strong effort were made in this area. For example, the case study on bridge decks in the New
Figure 10. Example sonic/ultrasonic/seismic waveform measurement.

NOTE:  
* Resonant period of upper concrete = 44 microsec.  
** Thickness = 3.2"  
*** Energy affected by surface conditions  
I Good concrete  
II Decomposed concrete between asphalt and 1st rebar  
III Delamination at 1st rebar
England states showed GPR performed effectively on decks overlaid with asphalt and containing a high percentage of reinforcing steel. The GPR accurately located areas of freeze/thaw damaged concrete when surveyed at the project level. It stands to reason this same type of success could be accomplished if the time and money were spent to adapt the radar system to Interstate pavements in Illinois.

CONCLUSIONS

Of the new nondestructive testing technologies which are being developed for evaluation of subsurface pavement conditions, ground-penetrating radar and infrared thermography have been the most widely developed and field tested. They have been used to predict properties and distresses ranging from pavement layer thicknesses to joint deterioration to delaminations between steel and concrete. Full-depth asphalt, reinforced concrete, and asphalt-overlaid concrete pavements have all been tested using these methods. Both technologies have shown to be capable of collecting large amounts of continuous data in relatively short periods of time. When calibrated using core samples, the devices have also shown to give reasonable results. However, data analysis and reduction of nondestructive testing outputs has often proven to be difficult and time-consuming. Also, more research into the cost-effectiveness and availability of equipment and services for nondestructive testing is also desirable from a practicality standpoint.

Considerable research is currently going on in the area of nondestructive testing, and the new technologies are making its way into pavement testing and evaluation. In fact, currently there is an ASTM Specification covering the use of ground penetrating radar for the specific use of pavement evaluation [10]. While further investigation into the effectiveness of innovative nondestructive testing techniques and their application to pavements is needed, these devices have already shown to be a potentially valuable source of information to the engineer.
REFERENCES FOR LITERATURE REVIEW OF NEW TECHNOLOGIES FOR NONDESTRUCTIVE PAVEMENT EVALUATION


