Assessment of FAA HMA Overlay Procedures
Volume I: Technical Report

Airfield Asphalt Pavement Technology Program
Project 06-07
Final Report
June 2010

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ABSTRACT

The latest Federal Aviation Administration (FAA) Advisory Circular on airfield pavement design, Advisory Circular 150/5320-6E, is based on a mechanistic-empirical approach toward the analysis of pavement structures. The approach to hot-mix asphalt (HMA) overlay design is still based on a structural deficiency approach to limit subgrade rutting (and HMA fatigue but not by default), but takes a different approach in modeling the pavement for analysis. With layered elastic analysis for flexible pavements and finite element method for rigid pavements, it is now more important to appropriately characterize the contribution of existing pavement layers in the design.

One aspect of the FAA airport pavement HMA overlay design that is unusually vague in the new procedure is the characterization of the structural contribution of an existing HMA pavement in the overlay design process. Verbatim guidance on this topic consists of the following:

- The existing flexible pavement is characterized by assigning the appropriate thicknesses and moduli of the existing layers.
- Subgrade and subbase properties can be measured by conducting NDT (nondestructive testing).

However, there is little guidance on characterizing the condition, and therefore the structural contribution, of the existing stabilized layers. The incorporation of repair and distress mitigation techniques is also not clearly defined.

This research project reviews current HMA overlay design procedures, assesses the sensitivity of design inputs in the FAA HMA overlay design procedures, discusses the general characterization of an existing pavement and repair and mitigation techniques in the design of HMA overlays, and provides recommendations for potential clarifications in the design procedure. Recommendations are also discussed for aspects of the design procedure that require additional research. The project report is contained in two volumes: Volume I represents the *Technical Report*, with detailed summary and discussion of the activities performed and recommendations, and Volume II, *Guidelines for HMA Overlay Design*, provides a brief summary of these aspects and provides a proposed framework for assessing existing pavements and for designing HMA overlays.

SUMMARY OF FINDINGS

This research report, *Assessment of FAA HMA Overlay Procedures* (AAPTP Project 06-07), summarizes the research performed in association with the following aspects of hot-mix asphalt (HMA) overlay design for airfields:

- Available design procedures and performance models.
- Sensitivity of FAARFIELD design inputs within the HMA overlay design procedure.
- Assessment of existing pavement evaluation methods.
- Assessment of corrective actions (or pre-overlay repairs).

The report is contained in two volumes: Volume I represents the *Technical Report*, with detailed summary and discussion of the activities performed and recommendations, and Volume II, *Guidelines for HMA Overlay Design*, provides a brief summary of these aspects and provides a proposed framework for assessing existing pavements and for designing HMA overlays.

In this volume, Chapter 2 summarizes the more common overlay design procedures and performance models for airfield pavements obtained by conducting a literature review. In particular, Chapter 2 includes a discussion of the available HMA overlay design procedures used by the FAA, U.S. Army Corps of Engineers, and the Asphalt Institute, with an emphasis on identifying the inputs used for each procedure, the failure mechanisms analyzed, and the strengths and weaknesses of each procedure. Additionally, behavior of performance models and typical failure mechanisms of HMA pavements and overlays are reviewed. Suggestions are also made where their possible incorporation into the FAA design procedure and FAARFIELD may be appropriate.

The performance of an HMA overlay is often measured by criteria other than subgrade rutting and HMA fatigue, which is the basis of the current FAA pavement design procedure. Two failure mechanisms typically associated with HMA overlay performance are reflective cracking and in-layer rutting. Discussion is provided in the design procedure for methods of mitigating reflective cracking, but it is not directly accounted for in the design of an HMA overlay. Similarly, the FAA's material specifications are intended to address the potential of in-layer rutting, but it is not directly accounted for in design. A possible design check for reflective cracking is presented herein, which considers a simplified approach to determining an effective cracked section and then using FAARFIELD to determine HMA overlay requirements. Adjustments for treatments specifically for reflective cracking can be included in the approach. While there has been some aviation research on in-layer rutting, the developed models are generally mix specific. More of the research on this aspect has been for highways. Therefore, no specific recommendation is made for a simplified approach to address this issue. However, several approaches are presented for consideration.

Based on a review of the available design procedures and performance models, the following items should be considered for future improvements to the FAA's HMA overlay design procedure:

- In FAARFIELD, all layers are assumed to be bonded. To adequately model some existing pavement structures, FAARFIELD should allow unbonded layers to be modeled. Although layers will not (or at least should not) be constructed as unbonded, there are many cases where the layers become unbonded over time. This feature will allow the user to investigate the effect of the existing unbonded layer and will help determine the need for and depth of milling.

- The ability to model variable inputs due to changes in moisture conditions, temperature fluctuations, and seasonal changes should be considered in future versions of FAARFIELD.
- Other failure modes should be considered in future versions of FAARFIELD. In particular, rutting/deformation in layers other than the subgrade and reflective cracking are two common distresses mechanisms that deserve consideration in HMA overlay design.

A sensitivity analysis of the HMA overlay design was performed using FAARFIELD for flexible and composite pavement cross sections to evaluate the impact of select design inputs on the required overlay thickness, which are summarized in Chapter 3. Specifically, the modulus and thickness of existing HMA layers were assessed to determine the relative sensitivity of the overlay thickness requirement to a relative change in the design input. These analyses were used to assess recommendations on how to use available FAARFIELD inputs to characterize the existing pavement layers for HMA overlay design. Recommendations based on the sensitivity analyses include the following:

- The subgrade CBR (or elastic moduli) determined using field evaluation techniques or other analytical methods should be used as the design value as opposed to using default values in FAARFIELD. Care should be taken in choosing the subgrade input because of the influence on HMA overlay thickness results.
- The use of the “undefined” layer is likely appropriate for the HMA surface layer when the existing modulus is greater than 20 percent different than the default P-401 modulus.
- The fatigue performance should be checked if the stiffness of the anticipated HMA overlay mixture is greater than 300,000 psi.
- The flexural strength of the underlying PCC in composite pavements needs to be determined accurately either by laboratory testing of retrieved samples or through correlation with backcalculated results.
- An effective thickness of the existing HMA overlay in composite pavements should be considered.
- Assuming a CDFU of 100 percent and establishing an appropriate SCI appears to be a reasonable approach for determining HMA overlay thickness of composite pavements.

Additionally, the condition of the existing pavement structure and the level of effort to correct deterioration have a significant influence on the performance of an HMA overlay. Therefore, it is important to accurately characterize the existing pavement structure in the design process and incorporate the influence of repairs or mitigation techniques incorporated into the HMA overlay construction.

Traditional pavement evaluation techniques to characterize the existing pavement structure, including visual, destructive, and non-destructive techniques, are reviewed in Chapter 4 to assess the types of data that can be collected and their potential use in HMA overlay design. A visual assessment is one of the easiest and most effective means of collecting pavement condition data. However, the data are not directly used in flexible pavement HMA overlay thickness design. SCI is used for characterizing the PCC layer of a composite pavement, but SCI is likely not a suitable means for flexible pavements. While visual assessment data are only used on a limited basis, they can be used to determine what other evaluation techniques are needed to adequately assess the distress mechanism(s), to assess potential adjustments to other design inputs, and to establish the need for corrective action or selection of a rehabilitation method other than HMA overlay.

Non-destructive testing methods often allow a greater amount of data to be collected and analyzed (compared with destructive methods), which can be an advantage over a destructive testing method such as coring. However, it is often beneficial to perform both destructive and non-destructive testing, as the results from these two approaches offer the greatest coverage over a project area and a means to validate results. To develop the pavement design inputs for the existing pavement layers, a combination of available evaluation techniques, such as a PCI inspection, limited coring, and FWD testing, for example, typically provides the best results. The combination of techniques provides pavement layer thicknesses, material properties, general condition ratings, and estimates of pre-overlay repair needs.

Chapter 5 presents various corrective and mitigative techniques and methods that can be used to repair existing distress and improve HMA overlay performance. The discussion includes a description of the technique, its function or purpose, the expected impact of each activity on the performance of the overlay, and an assessment of how the technique should be accounted for in the HMA overlay design process. The following techniques are discussed:

- Localized Maintenance and Repair.
  - Crack sealing.
  - Partial-depth patching.
  - Full-depth patching.
- Modification of Existing HMA Pavement.
  - Cold milling.
  - Surface recycling.
  - Surface leveling course.
- Stress/Strain Relieving Interlayers.
  - Stress-absorbing membrane interlayer (SAMI).
  - Proprietary stress-absorbing interlayers (ISAC, STRATA®, etc.).
- HMA Overlay Reinforcement.
  - Steel or wire fabric.
  - Geogrids.
  - Geosynthetic fabric.
- HMA Overlay Mixture Modification.
  - Rubber-modified asphalt binder.
  - Polymer-modified asphalt binder.

The activities can be used by themselves or in combination with other techniques, as is often done. Chapter 5 also discusses possible approaches to account for these corrective or mitigative activities in the FAA's HMA overlay design procedure, such as design input modification, effective thickness, or years of performance.

Chapter 6 summarizes the overall findings of the study and presents the recommended approaches for designing HMA overlays, which then serve as the basis for the guidelines presented in Volume II. Chapter 6 summarizes the recommendations for clarification in the Advisory Circular based on the research effort as well as the economic analysis. Recommendations for additional features to consider in future versions of FAARFIELD include the following:

- The ability to adjust the HMA layer modulus without the need to use the "undefined" layer. In-place properties can be considerably different than the assumed new material property, and using the "undefined" layer type has limitations, particularly if the HMA overlay properties need to be evaluated.
- The ability to account for variation in the HMA modulus based on temperature. Climatic regions will have much different temperature spectrums, and it can be anticipated that HMA material will perform differently in different climates. Including the ability to consider seasonal variation in the modeling would simplify the assessment of temperature differences. The more refined temperature modeling becomes (such as daily fluctuations), the more pressing the need for an automated process. A simplification would be to use a modulus established similar to using MAAT, which at least accounts for differences in climatic regions.
- Characterization of existing HMA overlays in composite pavement sections. Directly modeling the existing layer in the FAARFIELD analysis may resolve the arbitrary assessment of its contribution in the required HMA overlay thickness.

Based on the literature review and analyses performed as part of this study, recommendations for future research are also made, including in the following areas:

- Reflective cracking performance predictions.
- Assessing in-layer HMA rutting performance.
- Age-hardening effects on HMA modulus.
- Influence of levels of repair effort on long-term pavement performance.
- Validation of overlay designs with adjusted inputs compared to default inputs.
- Feasibility of using a structural index to characterize continued deterioration of the HMA.
- Assessment of top-down cracking significance in airfield HMA overlays.
- Frequency/time of loading effects on HMA modulus.

As this research is performed, further refinements and enhancements to the FAA's HMA overlay design procedure can be made.

This volume, *Volume I: Technical Report*, provides detailed discussion of the tasks performed as part of this project and recommendations for the Advisory Circular and FAARFIELD based on the accumulated data. Volume II of this report, *Guidelines for Design of HMA Overlays*, provides a brief summary of the design principles, existing pavement evaluation, and repair and mitigation techniques. Volume II also provides suggested steps for the design of HMA overlays within the FAA pavement design procedure.

CHAPTER 1. INTRODUCTION

BACKGROUND

The airfield pavement design procedure described in the Federal Aviation Administration (FAA) Advisory Circular 150/5320-6D (FAA 1995) was in use for over a decade, although its origins lie in research and empirical observations on the bearing capacity of roadways and airfields conducted over more than a 60-year period. That Advisory Circular also covers the structural design of hot mix asphalt (HMA) overlays of airfield pavements. The approach to overlay design is based on a thickness deficiency approach, in which the design thickness of an HMA overlay is the difference between the required thickness of a new HMA pavement designed to carry the projected aircraft traffic and the existing pavement structure. Depending on the nature of the existing pavement (HMA, portland cement concrete [PCC], fractured PCC, or HMA over PCC, for example), there are procedures to characterize the structural contribution of the existing pavement. Essentially, these are procedures to reduce the contribution of the existing pavement to the overall load-carrying capacity of the pavement based on its actual structural condition.

The 1995 pavement design Advisory Circular 150/5320-16, *Airport Pavement Design for the Boeing 777 Airplane*, was FAA's first attempt at incorporating mechanistic-empirical (M-E) concepts into the design and analysis of airfield pavement structures. That procedure was only for the design of pavements having the Boeing 777 in the traffic mix, and it used layered elastic analysis for both flexible and rigid thickness design. Specifically, that Advisory Circular introduced the FAA's Layer Elastic Design FAA (LEDFAA) software, in which M-E analysis is used to calculate critical load-induced strains, which are then used to develop pavement designs. The M-E concepts were later incorporated into Advisory Circular 150/5320-6D in 2004 with change 3 to the document.

Since 1995, the FAA has continued research and development activities in order to improve the mechanistic aspects of their pavement design procedure. In 2009, FAA released a new version of the pavement design Advisory Circular (Advisory Circular 150/5320-6E), and this version and the associated software design tool, FAARFIELD, continues FAA's movement toward a more mechanistic design approach for both new design and for overlay design (FAA 2009).

The Advisory Circular 150/5320-6E design procedure also uses the thickness deficiency approach to determine the required thickness of HMA overlays. For the design of an HMA overlay of PCC pavements, the procedure uses the Structural Condition Index (SCI) to provide an objective measure of the existing PCC pavement's condition. However, a similar method is not available to assess the condition of an existing HMA pavement to be overlaid. In fact, the Advisory Circular offers only the following verbatim guidance on characterizing the existing HMA pavement:

- The existing flexible pavement is characterized by assigning the appropriate thicknesses and moduli of the existing layers.

- Subgrade and subbase properties can be measured by conducting NDT [nondestructive testing].

There is no guidance on characterizing the condition, and therefore the structural contribution, of the existing pavement layers. Because determination of the overlay thickness is based on the existing structural capacity, this is an important omission and the impetus for this research study.

PROJECT OBJECTIVES

The research described in this report is intended to address the general characterization of an existing pavement in the design of HMA overlays. This research considers the available tools to evaluate a pavement's existing condition, including visual condition surveys, falling weight deflectometer (FWD) testing, ground penetrating radar (GPR), portable seismic pavement analyzer (PSPA), material sampling and testing, and other means.

The specific objectives of this project are summarized as follows:

- Review and provide comments and recommendations on the FAA's new elastic layer overlay design procedure.
- Develop guidelines for characterizing an existing HMA surface for airfield pavement HMA overlay designs that consider all realistic HMA failure modes. Consider reductions in assigned modulus values, layer thicknesses, and modifications to the cumulative damage factor.

An additional project objective is to develop recommendations for incorporating any proposed changes into both Advisory Circular 150/5320-6E and the accompanying FAARFIELD computer program.

RESEARCH APPROACH

To accomplish the project objectives, the project team performed the following seven tasks during the conduct of this research:

1. Review overlay design procedures and practices for airfields.
2. Conduct a sensitivity analysis on the FAARFIELD overlay design methodology.
3. Identify methods for assessing existing HMA pavement conditions, limiting values, and possible alternative corrective actions, including recommendations for how best to handle the condition of existing or modified pavements within the overlay design procedure.
4. Provide an interim report, documenting the efforts of the first three tasks and the future course of action being considered.
5. Revise the work plan based on the Technical Panel's comments and concerns, complete a draft report and guidelines, and submit to the Technical Panel for review.
6. Prepare a presentation detailing research completed and project products.

7. Revise report and guidelines and submit final products.

The team's approach to performing this research, and the results and recommendations from this research, are documented in this report.

REPORT ORGANIZATION

The report consists of seven chapters and one appendix, which roughly follows the work outlined by the task descriptions:

- Chapter 1. Introduction.
- Chapter 2. Review of HMA Overlay Design Procedures for Airfield Pavements.
- Chapter 3. Sensitivity Analysis of FAARFIELD HMA Overlay Design Procedure.
- Chapter 4. Identification and Assessment of Existing Pavement Evaluation Methods.
- Chapter 5. Identification and Assessment of Corrective Actions.
- Chapter 6. Recommendations for Revisions to FAA's HMA Overlay Design Procedure and for Future Research.
- Appendix A. Annotated Bibliography.

In addition, a stand-alone guide has been prepared to assist practitioners in the process of performing HMA overlay design using FAARFIELD and with HMA overlay design issues in general. These guidelines are presented under separate cover.

CHAPTER 2. REVIEW OF HMA OVERLAY PROCEDURES FOR AIRFIELD PAVEMENTS

INTRODUCTION

As part of this project, an extensive literature review was performed to identify publications regarding HMA overlay design procedures and practices for airfield pavements. The references identified are summarized in an Annotated Bibliography included as Appendix A of this report. This chapter summarizes the available literature pertaining to the overlay design procedures and practices for airfield pavements.

In particular, this chapter includes a discussion of the behavior, performance, and typical failure mechanisms of HMA overlays. Available HMA overlay design procedures—including those used by the FAA, U.S. Army Corps of Engineers, and the Asphalt Institute—are described in this chapter, with an emphasis on identifying the inputs used for each procedure, the failure mechanisms analyzed, the strengths and weaknesses of each procedure, and most importantly their possible incorporation into the FAA design procedure and FAARFIELD.

HMA OVERLAY PERFORMANCE AND FAILURE MECHANISMS

HMA Overlay Pavement Behavior

HMA overlays serve a wide variety of purposes ranging from addressing a surface problem such as poor skid resistance to providing additional structural capacity on a pavement that receives heavy traffic loads and/or high traffic volumes. In addition, with proper attention to traffic and environmental effects during structural and mix design, HMA overlays can be applied successfully over a wide range of climate and support conditions. In fact, HMA overlays are the most prescribed feasible alternative in most airports' pavement management systems.

An HMA overlay is generally assumed to be completely bonded to the underlying HMA pavement structure. If constructed properly, the overlay should behave similarly to the HMA surface layer of a new pavement. There are three primary factors that affect the behavior of an HMA pavement and an HMA overlay: aircraft gear loadings, temperature, and moisture. This section summarizes how an HMA pavement responds to these factors. This, in turn, is used as a basis to describe the various distress mechanisms.

Aircraft Gear Loading

Figure 1 shows a wheel load being applied to a pavement surface (through the tire) at a uniform vertical pressure. As demonstrated by the arrows, the pavement structure distributes the applied load through the structure and to the natural soil so that the maximum pressure on the subgrade soil is considerably less than the vertical pressure at the surface. In the process of distributing the load, various states of stress are built up within the various layers that can lead to the overall deterioration of the pavement.

In the HMA surface layer, tensile strains develop at the bottom of the layer (beneath the load) while vertical shear strains develop near the surface (at the edge of the load). The tensile strain is significant because it can cause cracks to develop and propagate through to the surface.

The shear strain is significant because it can lead to permanent deformation (rutting) in the traffic areas. The rate at which the distress develops depends on the number and magnitude of the wheel loads, pavement temperature, and the properties of the HMA layer(s), among other factors.

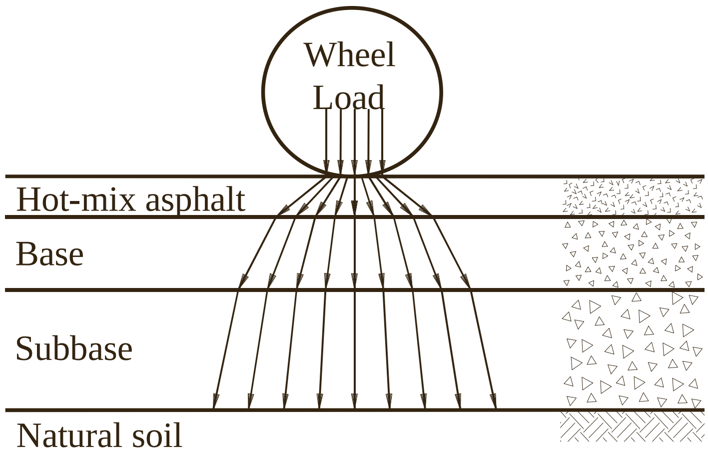


Figure 1. Distribution of wheel load in an HMA pavement.

In the base and subbase layers, compressive stresses and (some) tensile stresses will also develop. The compressive stresses can cause permanent deformation to develop in these layers. If the layer(s) consist of unbound granular materials, they will not be able to carry any tension resulting from "bending" of the structure. If an applied load produces excessive bending, the base and/or subbase layers are likely to de-compact and lose some of their load-carrying capacity.

Temperature

As ambient temperatures change throughout the day and throughout different seasons, the temperature of the HMA layer also changes. Because HMA is subject to the expansion and contraction associated with these temperature changes and because the HMA pavement is constructed in a situation where these temperature-related movements are restrained, compressive and tensile stresses will develop. The compressive stresses rarely create problems with pavement distress, but the tensile stresses do. If the tensile stress exceeds the tensile strength of the HMA layer, a crack will develop. Usually, these cracks appear first in the transverse direction. As the pavement ages, longitudinal cracks will also develop and eventually will form block cracking.

Temperature also has an effect on the properties of certain materials, principally the HMA surface and any other asphalt-stabilized layer. The properties of HMA change with changes in temperature; consequently, increases in HMA temperature can result in significant decreases in stiffness and resistance to permanent deformation. In hot environments, this problem should be handled through the selection of an asphalt binder and HMA aggregate gradation that is rut resistant at high temperatures.

Moisture

Moisture affects materials and, therefore, pavement behavior in two ways:

- In unbound (and some bound) layers, moisture acts as a lubricant. By permitting more freedom of movement between aggregate particles, it effectively weakens the material and reduces the pavement's load-carrying capacity.
- In certain asphalt-stabilized layers (including the HMA surface) with hydrophilic aggregates, moisture can cause a separation of the asphalt from the aggregate (i.e., stripping). This will also weaken the material and reduce the overall load-carrying capacity of the pavement.

Failure Mechanisms and Distress Manifestations

HMA material properties are important, but the condition of the existing pavement, including the extent of pre-overlay repairs performed, may be the single most important factor in determining how well an HMA overlay will perform (this is discussed in Chapter 5). Two of the most common failure mechanisms for HMA overlays are reflective cracking and in-layer permanent deformation. These factors are discussed below.

Reflective Cracking

Reflective cracks appear on the surface of an overlay above joints or cracks in the underlying pavement layer. Reflective cracking occurs in nearly all types of overlays, but is most often a problem in HMA overlays of HMA pavements with existing thermal cracks, and in HMA overlays of jointed PCC pavements. Although attempts have been made to develop a comprehensive design procedure that addresses the problem of reflective cracking (Seeds, McCullough, and Carmichael 1985; Majidzadeh et al. 1987); to date no procedure has received widespread acceptance.

Reflective cracking is a result of horizontal and vertical movements at the joints and cracks in the underlying pavement that create high stress concentrations in the overlay. These movements at joints and cracks are caused by a combination of low temperature and traffic loads (Smith et al. 1984).

Low temperatures cause the underlying pavement to contract, increasing joint and crack openings. This horizontal movement in the base pavement creates tensile stress in an overlay, as illustrated in figure 2. In addition, the overlay is subjected to further tensile stress as the overlay material itself also contracts in response to low temperatures, shown in figure 3.

Traffic loadings produce a completely different type of movement, shown in figure 3. Differential vertical deflection created by traffic passing over a joint or working crack creates shearing and bending stress in the overlay. Three distinct load pulses are produced by a moving wheel load (Jayawickrama and Lytton 1987):

- As the wheel load approaches the crack, the shear stress in the overlay above the crack will reach a maximum, illustrated as point A.
- When the wheel is directly above the crack, the maximum bending stress will occur, as illustrated by point B.
- As the wheel load crosses the crack, a second maximum shear stress in the reverse direction will occur, as illustrated by point C.

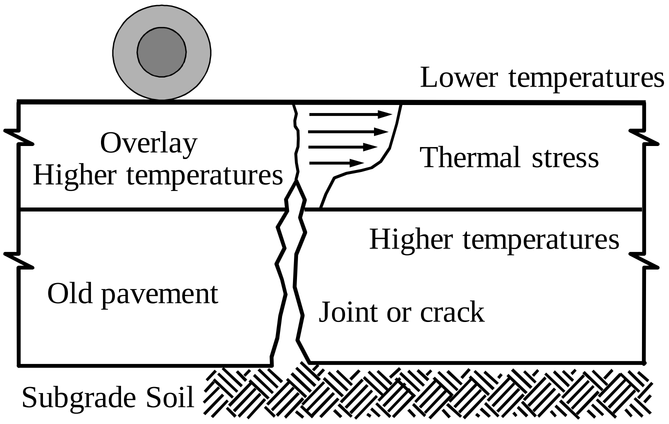


Figure 2. Stresses in HMA overlay caused by low temperatures.

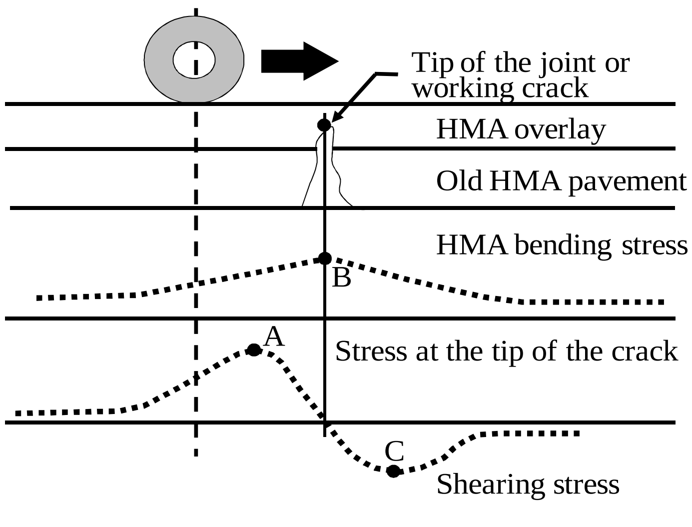


Figure 3. Shearing and bending stress in an HMA overlay created by a moving load (Jayawickrama and Lytton 1987).

Stresses from traffic loadings occur far more often than temperature stresses, but are typically much smaller in magnitude. The poorer the load transfer across the crack or the support under it, the more rapidly the crack will reflect through the overlay, and the more severe the deterioration will become. It is critical that the expected deflections before and after an overlay be evaluated so that the most cost-effective rehabilitation or reconstruction method may be selected.

For both temperature and load-induced reflective cracking, crack initiation usually begins at the bottom of the overlay. Each temperature cycle and traffic load causes damage that contributes to the propagation of the reflective crack further up into the overlay. The different types of horizontal and vertical movements do not propagate the crack equally, and the number of movements is not in itself a criterion for predicting the rate of reflective cracking. While further studies into the influence of each type of load damage are needed, much is already known today that can be applied to this problem.

The key to eliminating reflective cracking is to eliminate the deformations and stresses produced in the HMA overlay at existing joints and cracks. However, it is highly unlikely that these deformations and stresses can ever be completely eliminated; the most that can be achieved is a reduction in the rate of appearance and the severity of the reflective cracking. This is the primary reason that pre-overlay repairs and treatments are used.

AAPTP Project 05-04, *Techniques for Mitigation of Reflective Cracking*, provides guidance and recommendations to manage and design rehabilitation strategies of airside pavements for mitigating reflection cracks (ARA 2008). The guidance includes selection and use of materials and treatment methods to increase the time to the occurrence of reflective cracks in HMA overlays of rigid, flexible, and composite pavements. It also provides guidance on the cost-effectiveness of different treatments for minimizing reflection cracking from experience gained of various organizations (both airfield and highway uses) and an evaluation of numerous airfields that have used the different strategies.

Reflective cracking leads to increased infiltration of surface water into the pavement system, which in turn weakens the supporting layers. Over time, reflective cracks will also deteriorate and spall, decreasing the pavement’s serviceability. Design issues regarding reflective cracking consider the following factors:

1. The rate of reflective cracking through the overlay.
2. The amount and rate of deterioration of the cracks after cracking occurs.
3. The amount of water that can infiltrate through the cracks.

While each is important, the second factor is the most cost-effective to address. If the crack severity can be limited, sealing the crack is easier and the contribution of crack deterioration to pavement condition and roughness is greatly reduced.

Permanent Deformation

Permanent deformation (rutting) may occur in HMA overlays; sometimes within the first few years of overlay construction. This has become a growing concern at many airports, due to the continued increase in aircraft weights, gear weights, and tire pressures. Rutting observed at the surface can be the accumulation of deformation in one or all of the pavement layers. Rutting of the subgrade is generally considered in airport design and assumes no rutting in other pavement layers. Rutting of the subgrade is typically caused by shear deformation of the upper portion of the subgrade.

However, rutting also commonly occurs in the HMA and unbound granular layers as well. Surface rutting is especially problematic on airfields where static aircraft loads are applied, such as on aprons, at other parking areas, at hold lines, and on other areas where aircraft stacking may occur. Surface rutting occurs when the HMA mixture exhibits low stiffness under loading. Critical factors to consider in minimizing rutting include the angularity of the coarse and fine aggregate, aggregate gradation, the rut-resistance of the mix design, the proposed field compaction efforts, selection of asphalt binder, asphalt content, and volumetric mix properties.

Testing at the FAA's full scale testing facility has revealed significant contribution to total rutting from the P-209 and P-154 unbound materials (Kim and Tutumluer 2006). Under heavy loading, additional densification or shear failures can occur within the aggregate layers, particularly thick aggregate layers.

Critical Pavement Responses

The two pavement responses generally considered in the design of flexible pavements and HMA overlays are the vertical compressive strain at the top of the subgrade and the horizontal tensile strain at the bottom of the HMA layer(s). These are discussed below.

HMA Fatigue

The basic concept of most mechanistic approaches to overlay design is that the design overlay thickness will limit fatigue damage in the existing pavement and/or overlay to an acceptable level over the design period. The existing pavement and overlay are modeled using elastic layer theory or finite element analysis to estimate the critical fatigue responses associated with the design load. For flexible pavements with original HMA surface layers or HMA overlays, the critical response is the maximum tensile strain at the bottom of the original HMA surface layer or HMA overlay, as depicted in figure 4.

The mechanism of fatigue refers to a progressive process whereby a bound layer in the HMA pavement structure undergoes so many repeated applications of stress (or strain) that it eventually cracks. No one application of stress is enough to exceed the strength of the bound material, yet the accumulation over time is enough to eventually wear out or fatigue the material. In figure 4, response 2 represents the maximum tensile strain at the bottom of the HMA surface layer resulting from the application of a single wheel load. The repeated generation of this strain first causes the initiation of a crack at the bottom of the layer. Continued load repetitions ultimately cause the crack to propagate from the bottom to the surface. The presence of fatigue cracking is an indication of the loss of structural (load-carrying) capacity in the pavement. Once it initiates, fatigue cracking can progress at a very rapid rate as the pavement becomes weaker and weaker. Thus, if otherwise uniform areas of pavement begin to develop fatigue cracking, the rest of the pavement is likely not far behind. This should have some bearing on the selection of an appropriate rehabilitation alternative.

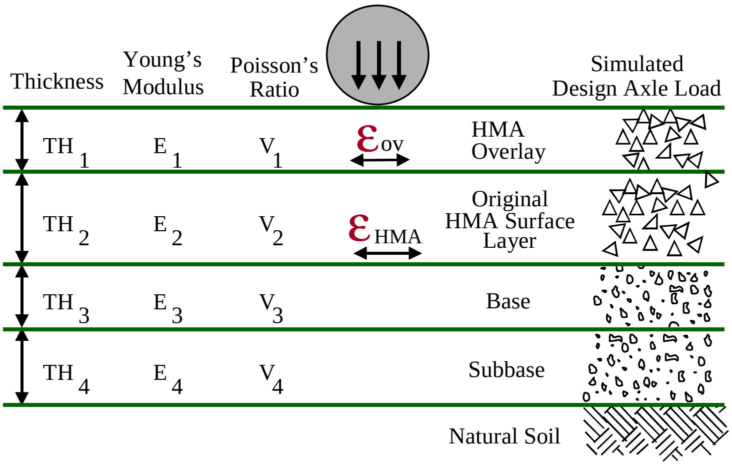


Figure 4. Illustration of elastic layer theory used to estimate critical tensile strain in HMA overlay (ε_{OV}) and/or original HMA surface layer (ε_{HMA}).

It should also be noted that fatigue cracking does not always initiate in the HMA surface layer for new flexible pavements. If a stabilized base course exists, its stiffness may be great enough such that tensile strains can be generated at the bottom of this layer and fatigue cracking may initiate there. Likewise, fatigue cracking of an HMA-overlaid pavement can initiate in the original surface or base course.

Permanent Deformation

As discussed in the previous section, permanent deformation (or rutting) refers to a progressive process whereby the accumulation of small amounts of wheel load-related permanent deformation in one or more layers ultimately leads to a depression of the pavement surface in its wheel paths (i.e., ruts). However, current FAA design procedures generally only consider rutting at the top of the subgrade.

In figure 5, responses 1, 3, and 4 represent the vertical compressive stresses on the HMA surface, base and subgrade soil, respectively. Each of these vertical stresses can contribute to the permanent deformation of the layer(s) beneath them. If deformation exists only in the HMA surface layer, then any one or combination of the following could be the culprit:

1. The HMA surface layer was overloaded.
2. Loading was exerted during a hot period (when the HMA layer was "soft").
3. There was a problem with the stability of mix.
4. There was a problem with the temperature susceptibility of the asphalt.

In some instances, ruts will develop in which some material will be shoved down, to the side, and upwards along the edge of the rut, creating a "lip" at the pavement surface. This type of rutting, along with double-ruts left by dual-wheel aircraft, often indicates a problem with the HMA surface layer (either mix design or construction).

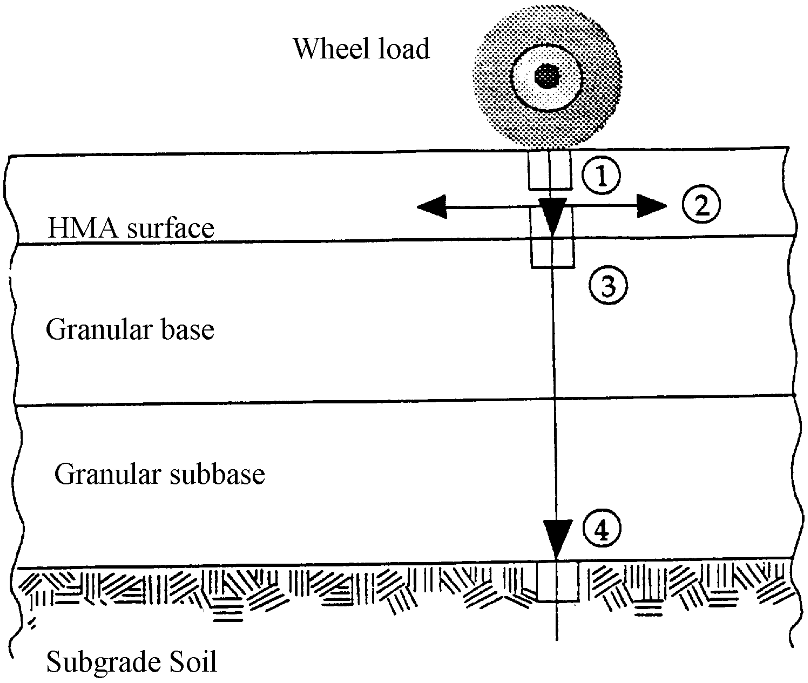


Figure 5. Critical stress and strain locations in HMA pavement with granular base.

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

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

If the total rutting at the surface exists in all layers down to the subgrade, then the problem is likely attributed to one of the following reasons:

- The pavement structure is too thin for the applied loads.
- The subgrade is naturally weak (or becomes weaker due to periods of saturation).

AVAILABLE OVERLAY PAVEMENT DESIGN METHODS

Available HMA overlay pavement design methods from the FAA, military, and the Asphalt Institute are presented in this section. For each procedure, the approach to HMA overlay design, required inputs, and failure modes are discussed. The pros and cons of the current FAA pavement design procedure are also addressed. For the other design procedures, their potential applicability for incorporation into the FAA overlay design procedure is also discussed.

FAA Overlay Design Procedures

This section discusses the history of the FAA design procedures, from the first procedure (or policy as it was called at the time) in 1958 to the latest procedures documented in Advisory Circulars 150/5320-6D and 150/5320-6E, with particular emphasis on approaches used for the design of HMA overlays in the most recent Advisory Circulars.

History of FAA Design

In 1958, the FAA developed a policy to limit the amount of federal funding to those pavements designed to serve a 350,000-lb aircraft with a DC-8-50 landing gear configuration. The intent was to ensure that future aircraft were equipped with landing gears that would not stress the pavements more than the referenced 350,000-lb aircraft (FAA 1995). For the most part, aircraft manufacturers were able to comply with the policy, even though the aircraft gross weight often exceeded 350,000 lbs, by increasing the number and spacing of landing gear wheels. However, with the development of increasingly larger aircraft it was only a matter of time until this criterion was exceeded by the aircraft manufacturers.

The first procedure (Advisory Circular 150/5320-6C) bearing resemblance to the FAA procedure used recently (Advisory Circular 150/5320-6D) was introduced in 1978. These procedures are based on the California Bearing Ratio (CBR) method and Westergaard edge-stress analysis for flexible and rigid pavement design, respectively. These methods serve as the basis of the FAA's "nomograph" (or conventional) design procedure. For flexible pavement design, the performance criterion is based on controlling subgrade rutting, while rigid pavement performance is based on controlling slab cracking. The design inputs for the nomograph-based flexible design method include subgrade support and traffic data (anticipated loadings of the determined "design aircraft") for a given design period. For rigid pavement design, the inputs include subgrade support, PCC flexural strength, and traffic.

Mechanistic-empirical airport pavement design, as introduced in FAA's Layered Elastic Design, Federal Aviation Administration (LEDFAA) program (introduced as Advisory Circular 150/5320-16 in 1995 for design of pavements with the Boeing 777 in the traffic mix) and continued in their new FAA Rigid and Flexible Iterative Elastic Layered Design (FAARFIELD) program (released as part of Advisory Circular 150/5320-6E in 2009), correlates critical pavement stresses and strains to empirical performance models. The move to mechanistic principles also incorporates additional layer properties into the design procedure, including elastic modulus and Poisson's ratio. Traffic loadings are also handled differently. The procedure no longer uses the "design aircraft" concept, instead analyzing each specific aircraft loading to determine its contribution to pavement damage.

FAARFIELD uses layered elastic analysis to determine critical strains (compressive strain at top of subgrade and tensile strain at bottom of the top HMA surface layer) in flexible pavements and three-dimensional finite element analysis to determine critical stresses (tensile strain at the bottom of the slab) in rigid pavements. Although it is the stresses and strains that are being calculated, the performance criteria are essentially the same as the nomograph procedure: subgrade rutting for flexible pavement design and slab cracking for rigid pavement design. HMA fatigue is also determined in FAARFIELD, but is not the default criterion (i.e., the user can view the results, but it does not affect the resulting design thicknesses).

FAA Advisory Circular 150/5320-6D

Following this procedure, the design of a structural HMA overlay on an existing flexible pavement is based on a thickness deficiency approach. The designer must first design a new flexible pavement and then the structural contribution of the existing pavement is subtracted from this requirement, leaving the deficiency to be made up with the HMA overlay.

The flexible pavement design procedure outlined in FAA Advisory Circular 150/5320-6D is based on the CBR method, and the performance criterion is based on controlling rutting in the subgrade (LEDFAA was first released with Advisory Circular 150/5320-16 in 1995 for design of pavements with the Boeing 777 in the traffic mix and incorporated in 2004 in change 3 of this Advisory Circular). The design inputs include subgrade support (CBR) and anticipated traffic data (aircraft design weight and volume) over a given design period (typically 20 years for FAA design).

This procedure is commonly referred to as FAA's "conventional" or "nomograph" procedure, because it uses a design nomograph to determine the required thickness. It was traditionally a manual method, although the designs can now be accomplished using spreadsheets developed by the FAA. The procedure is based on the "design aircraft" concept in which the effects of each aircraft (or at least the most damaging aircraft) are represented by the equivalent departures of a single aircraft. The design aircraft is not necessarily the heaviest, but rather the single aircraft in the mix that by itself produces the greatest pavement thickness when using the combination of gross aircraft weight and estimated departures in the appropriate pavement thickness design nomograph.

The following steps are required to design a new flexible pavement:

1. Determine the design inputs:
    a. Design subgrade CBR (average minus one standard deviation or 85th percentile).
    b. Aircraft traffic mix (maximum take-off weight and annual departures).
    c. Design life (typically 20 years for FAA design).
2. Select the design aircraft by determining the required thickness of each aircraft using the pavement design curves (nomographs).
3. Convert all aircraft into equivalent departures of design:
    a. Convert to equivalent gear.
    b. Convert to equivalent weight.
4. Determine the total required pavement thickness using the appropriate curve for the design aircraft and the design subgrade CBR.
5. Determine the required pavement thickness on the subbase using the subbase CBR (typically 20) and the same design curve.
6. Determine the required HMA surface thickness, which is listed on the design curve for the appropriate design aircraft (typically 4 or 5 inches).
7. Calculate the required subbase thickness as the total required thickness (from step 4) minus the thickness required on the subgrade (from step 5).

8. Calculate the required base thickness as the total required thickness (from step 4) minus the required HMA surface thickness (from step 6) and the required subbase thickness (from step 7).

A sample nomograph is presented in figure 6 for a dual tandem gear. Similar nomographs are also presented in the Advisory Circular for single wheel gear, dual wheel gear, and specific aircraft. The appropriate chart must be selected to correspond with the design aircraft.

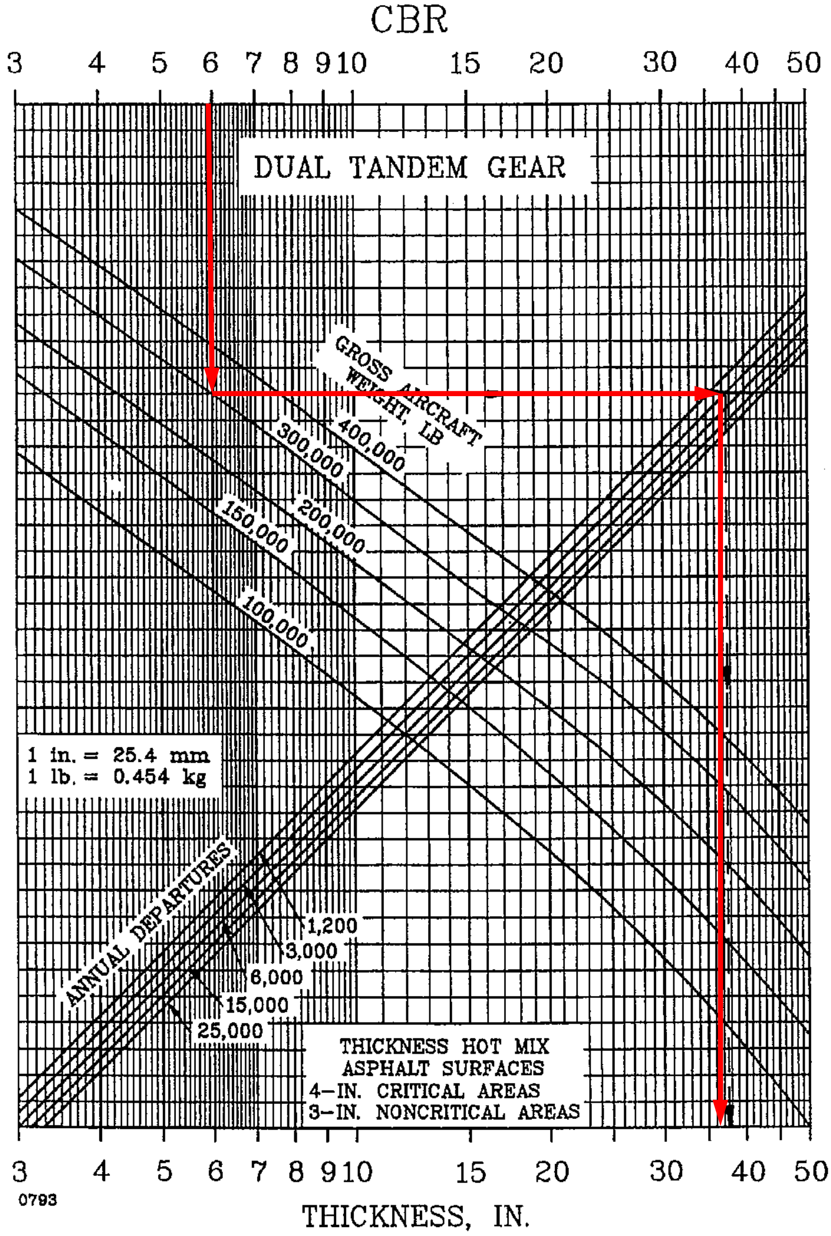


Figure 6. Flexible pavement design curve for dual tandem aircraft (from Figure 3-4, FAA 1995).

The cross section developed from these steps is referred to as the “standard cross section.” For new flexible pavement design, additional steps need to be taken to determine if a stabilized base is required (if there are aircraft greater than 100,000 lbs in the mix) and to check the minimum base and subbase thicknesses. However, these additional steps are not required when designing an HMA overlay, although they should be considered when determining the ability of an HMA overlay to provide the required level of performance.

Once the required new flexible pavement cross section is established, the existing pavement layers must be converted to match the required cross section. HMA surface can be converted to base, and base may be converted to subbase. However, a lower quality material cannot be converted to a higher quality material. This conversion is done using the equivalency factors provided in Tables 3-6 through 3-9 in Advisory Circular 150/5320-6D, although it must be recognized that these factors are based on new materials and not in-place materials. The Advisory Circular does not provide much additional guidance other than to say the selection should be “based on experience and judgment” and “surface cracking, high degree of oxidation, evidence of low stability, etc. would tend to reduce the equivalency factor.”

A sample table is presented in table 1 for converting layers. In this case, the conversion is from a granular subbase to a stabilized subbase. As an example, 1 inch of P-401 would be equivalent to 1.7 to 2.3 inches of a granular subbase (P-154). Again, the selection of the specific factor to use for design is left to the discretion of the engineer.

Table 1. Recommended equivalency factor ranges for stabilized base (from Table 3-7, FAA 1995).

Material	Equivalency Factor Range
P-301, Soil Cement Base Course	1.0 – 1.5
P-304, Cement Treated Base Course	1.6 – 2.3
P-306, Econocrete Subbase Course	1.6 – 2.3
P-401, Plant Mix Bituminous Pavement	1.7 – 2.3

As noted, the design method for HMA overlays on existing HMA pavement follows a thickness deficiency approach. This method consists of the following steps:

- The thickness required for a new pavement for the anticipated load and the projected traffic is determined using the appropriate design nomograph. The thickness of each pavement layer is determined in the same manner as if a new HMA pavement were being designed, as previously discussed. However, the properties of the subgrade and pavement layers may be different than used for new design.
- The required thickness of the new pavement is compared to the existing pavement structure and the thickness of the overlay is determined. In this step, the existing pavement must be converted to the “standard cross section” consisting of P-154 subbase, P-209 base, and P-401 surface. The existing base layer may need to be converted to subbase, and the existing surface may need to be converted to base. These conversions

are performed using the factors provided in Tables 3-6 through 3-9 of Advisory Circular 150/5320-6D. A higher quality material layer may be converted to a lower quality material layer, but a lesser quality material may not be converted to a higher quality material.

Considerable variability and subjectivity is associated with the estimation of the structural condition of the existing pavement to be overlaid. Hence, while using the layer equivalency factors, proper consideration should be given to the condition of the existing pavement, as defects such as surface cracking, a high degree of oxidation, and evidence of low stability would affect the selection of the equivalency factor. The Advisory Circular also notes that any HMA layer located between granular courses in the existing pavement should be evaluated inch for inch as granular base or subbase course (i.e., no conversion is allowed).

FAA Advisory Circular 150/5320-6E

The design procedure outlined in Advisory Circular 150/5320-6E is radically different from the nomograph procedure and for flexible pavement design advances the use of layered elastic design initiated with LEDFAA. This design procedure adopts a much more mechanistic approach towards the analysis of pavement structures. Mechanistic-empirical design correlates critical pavement stresses and strains to empirical performance models. The move to mechanistic principles also incorporates additional layer properties into the design procedure, including elastic modulus and Poisson’s ratio. Traffic loadings are also handled differently. And whereas in Advisory Circular 150/5320-6D a solution could be developed manually with a nomograph, in this procedure a program must be used.

As with the explanation of the Advisory Circular 150/5320-6D procedure, it is first important to understand the concept for developing designs for new flexible pavements before describing the overlay design approach. First, the procedure is no longer based on the CBR methodology, although the models are calibrated to the CBR design procedure. It still evaluates rutting in the subgrade as one of the potential failure modes, but the process is based on determining the vertical compressive strain on the subgrade using the layered elastic subprogram, LEAF, and correlating that response to failure through a performance model (note that the performance models are calibrated to CBR-based design data). FAARFILED also allows the option of evaluating the horizontal tensile strain at the bottom of the HMA surface layer to investigate the potential for fatigue cracking of the HMA surface. This failure mode is turned off by default, but the user can enable this failure mode by deselecting the “No AC CDF” checkbox within the FAARFILED Options screen, as shown in figure 7.

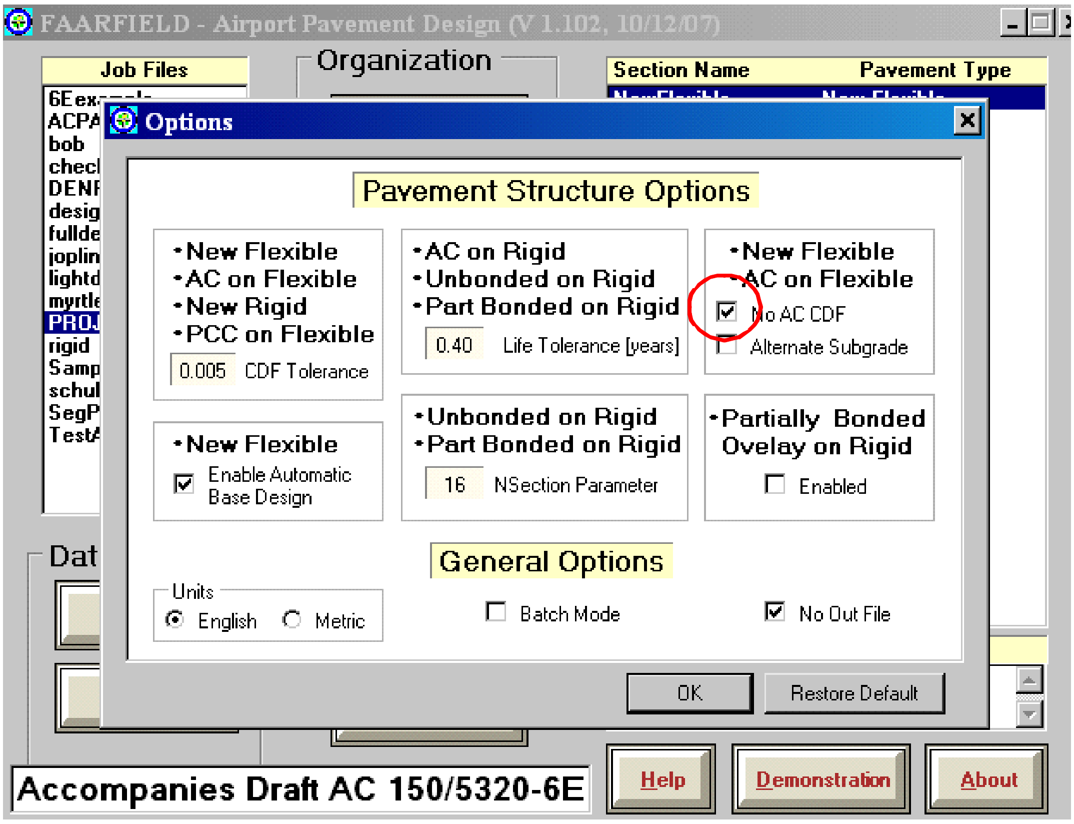


Figure 7. Option for incorporating HMA fatigue as failure criterion.

However, worth noting is that in FAARFIELD only the strain of the HMA surface layer is evaluated for fatigue failure and all bituminous layers are assumed to be bonded to each other. This is different than other design procedures that evaluate fatigue failure, such as the Asphalt Institute, which evaluate the strain at the bottom of the lowest stabilized layer (not necessarily the bottom of the surface layer). In other words, most procedures evaluate the strain at the bottom of the stabilized base. Because the HMA surface and stabilized base layers are assumed to be bonded in FAARFIELD and the surface modulus is set at 200,000 psi, this failure criterion seldom, if ever, controls predicted performance (note: this is further illustrated by the results of the sensitivity analysis described in Chapter 3 of this report).

Another major change is the way in which the procedure characterizes traffic. The procedure in Advisory Circular 150/5320-6E (previously Advisory Circular 150/5320-6D, Change 3) no longer uses the “design aircraft” concept. Instead, it analyzes each specific aircraft loading to determine its contribution to pavement damage, using a cumulative damage factor (CDF) to represent the life of the pavement. For a single airplane and constant departures, the CDF is expressed as the ratio of the applied load repetitions to the allowable load repetitions or coverages to failure (as determined from the applicable performance model). Multiple airplane types are accounted for by applying Miner’s rule. Table 2 shows the relationship between the CDF value and the remaining life of the pavement. Essentially, a pavement has used up its life when the CDF equals 1.0 (or 100 percent).

Table 2. Pavement remaining life based on CDF value (FAA 2009).

CDF Value	Pavement Remaining Life
1	The pavement has used up all of its fatigue life
< 1	The pavement has some life remaining, and the value of CDF gives the fraction of the life used.
> 1	The pavement has exceeded its fatigue life

Within the program, CDF is calculated for each 10-inch strip along the pavement over a total width of 820 inches (82 strips). Using the pass-to-coverage ratio for each aircraft and a normal distribution of aircraft wander with a standard deviation of 30.435 inches, CDF is computed for all 82 strips. FAARFIELD iterates by changing the layer thickness (the subbase thickness for new design and the HMA overlay thickness for overlay design) until the maximum CDF of any strip is equal to 1.0. Figure 8 provides an illustration of this concept, in which the cumulative CDF is computed as the sum of the CDF from each individual aircraft over each 10-inch strip.

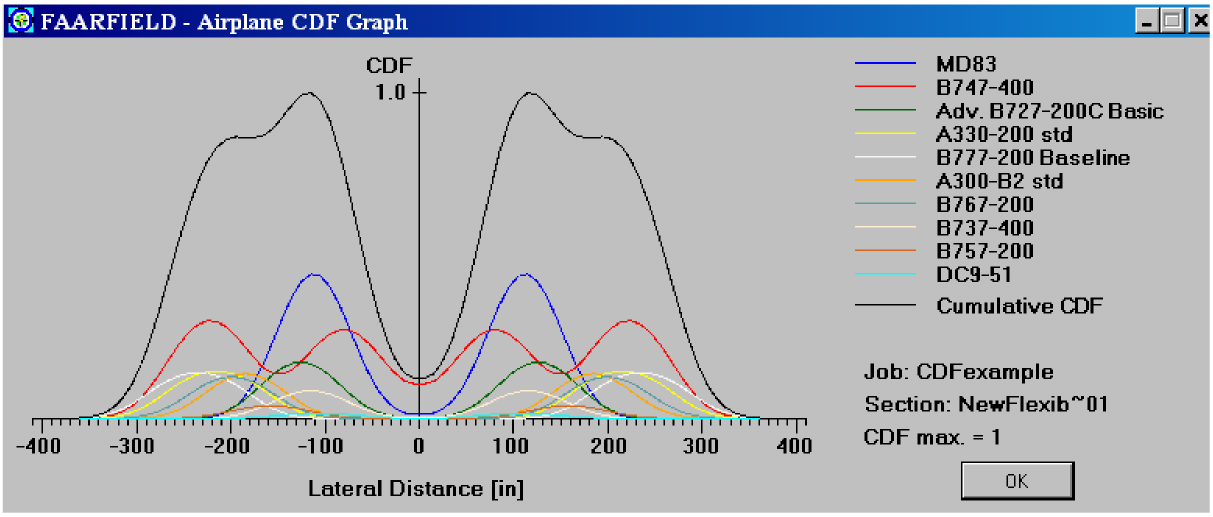


Figure 8. FAARFIELD screen illustrating cumulative CDF calculation.

The following inputs are required for new flexible pavement design using FAARFIELD:

- Design life – Typically 20 years for FAA standard design.
- Traffic – Aircraft type, weight, and volume for each aircraft (note: FAARFIELD has an aircraft library that includes other properties, such as gear type, wheel spacing, and tire pressure).
- HMA surface – A minimum surface thickness of 4 inches is required. The HMA surface modulus is fixed at 200,000 psi (this value was conservatively selected by FAA to represent a pavement temperature of 90 °F)..

- Stabilized base course – A stabilized base course is required for pavements being designed to handle aircraft exceeding 100,000 lbs. The available stabilized bases in FAAFIELD are presented in table 3, along with the default modulus (or range of allowable values) and Poisson's Ratio. The materials conforming to FAA's construction specification have a fixed modulus in FAARFIELD. If another material or modulus is desired, a "variable" layer must be used.
- Unstabilized base course – The standard aggregate base course for FAA flexible pavement design is a P-209, crushed aggregate base course. The modulus of this layer is automatically calculated in FAARFIELD using the "Modulus" procedure developed by the U.S. Army COE (UFC 3-260-02, Army 2001).
- Subbase course – The standard subbase material is a P-154, aggregate subbase course. The modulus of this layer is also internally calculated similar to the aggregate base layer. For pavements required to handle aircraft exceeding 100,000 lbs (in which case a stabilized base course is required), the Advisory Circular recommends a higher quality subbase (such as P-208 or P-209) be used for the subbase course.
- Subgrade support condition – Although the program uses the elastic modulus (E) of the subgrade in its calculations, either CBR or E can be entered, and FAARFIELD will automatically calculate the other parameter using the following correlation:

$$E = 1500xCBR \qquad \text{(Eq. 1)}$$

In a new HMA design, the required thickness of the base course (unbound and stabilized) can be automatically determined (note: the base thickness can be set by the user by deselecting the "Enable Automatic Base Design" checkbox in the Options window). The thickness of P-209 required to protect a subbase with CBR of 20 (assumed to be P-154 material) is automatically determined in FAARFIELD, and then the P-209 base is converted to a stabilized base using an equivalency factor of 1.6 when aircraft weighing more than 100,000 lbs are included in the traffic mix.

Table 3. Available stabilized base layers in FAARFIELD (from Table 3-8, FAA 2009).

Base Layer	**Modulus, psi**	**Poisson's Ratio**
Stabilized (Flexible) Variable P-401/P-403 Asphalt	 150,000 – 400,000 400,000	0.35
Stabilized (Rigid) Variable P-304 Cement Treated Base P-306 Econocrete Subbase	 250,000 – 700,000 500,000 700,000	0.20

Once these inputs are selected and entered into FAARFIELD, the design (at least from the user's standpoint) is simple. The user simply clicks the "Design Structure" button. Internally the program performs an iterative process of adjusting the layer thicknesses and computing the CDF (for all 82 strips) until a CDF of 1.0 is reached.

Figure 9 shows the FAARFIELD screen after the design has been run. A few items on this screen are worth noting:

- The base thickness is determined automatically within FAARFIELD as the minimum thickness required to protect the subbase.
- The subbase thickness is what is designed for flexible pavement design. This is noted by the arrow on the left side of the pavement layers.
- Once the design is complete, the CDF will be approximately equal to 1.00, as noted at the bottom of the window.

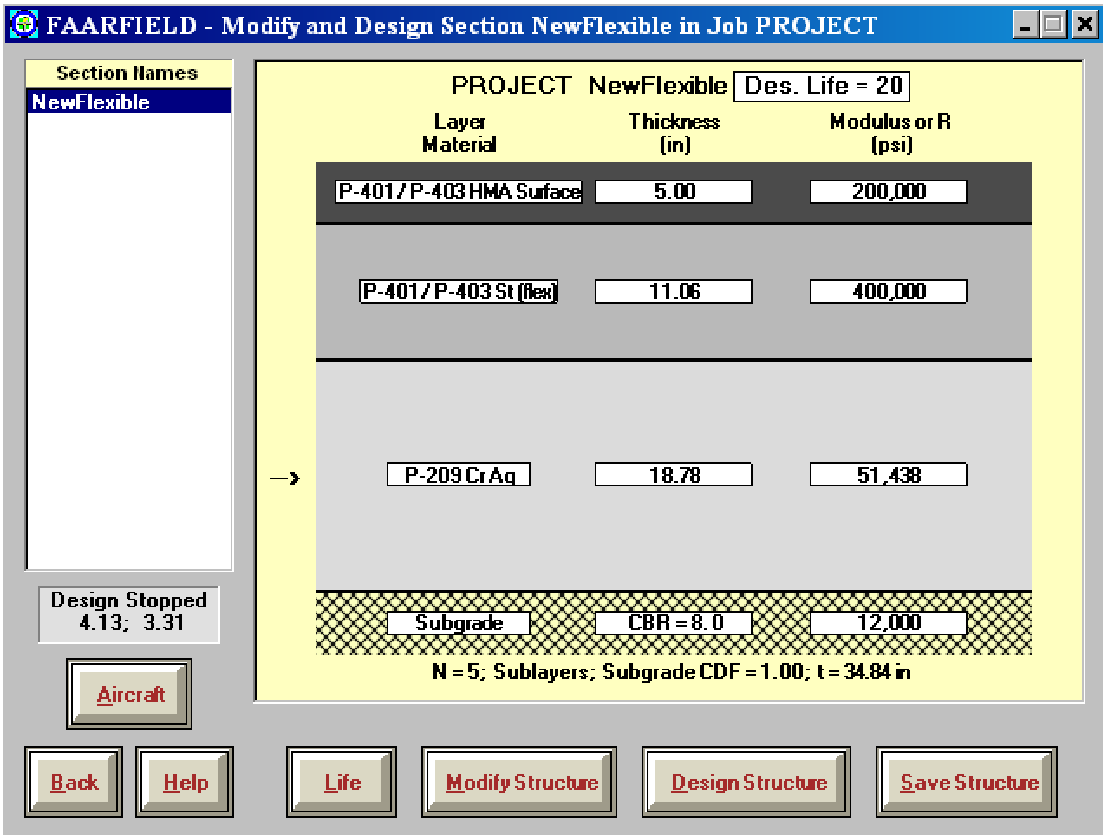


Figure 9. FAARFIELD flexible pavement design screen.

Understanding the new flexible pavement design, the determination of an overlay thickness using FAARFIELD is a fairly easy process in which the user adds the overlay layer to the pavement cross section. When this is done, the arrow indicating the layer being designed will move to the overlay layer, as shown in figure 10. The other layers (type, thickness, and modulus) are set to represent the in-place properties of those materials. Then the user simply clicks the "Design Structure" button, and FAARFIELD calculates the required overlay thickness. The minimum allowable HMA overlay thickness is 2 inches.

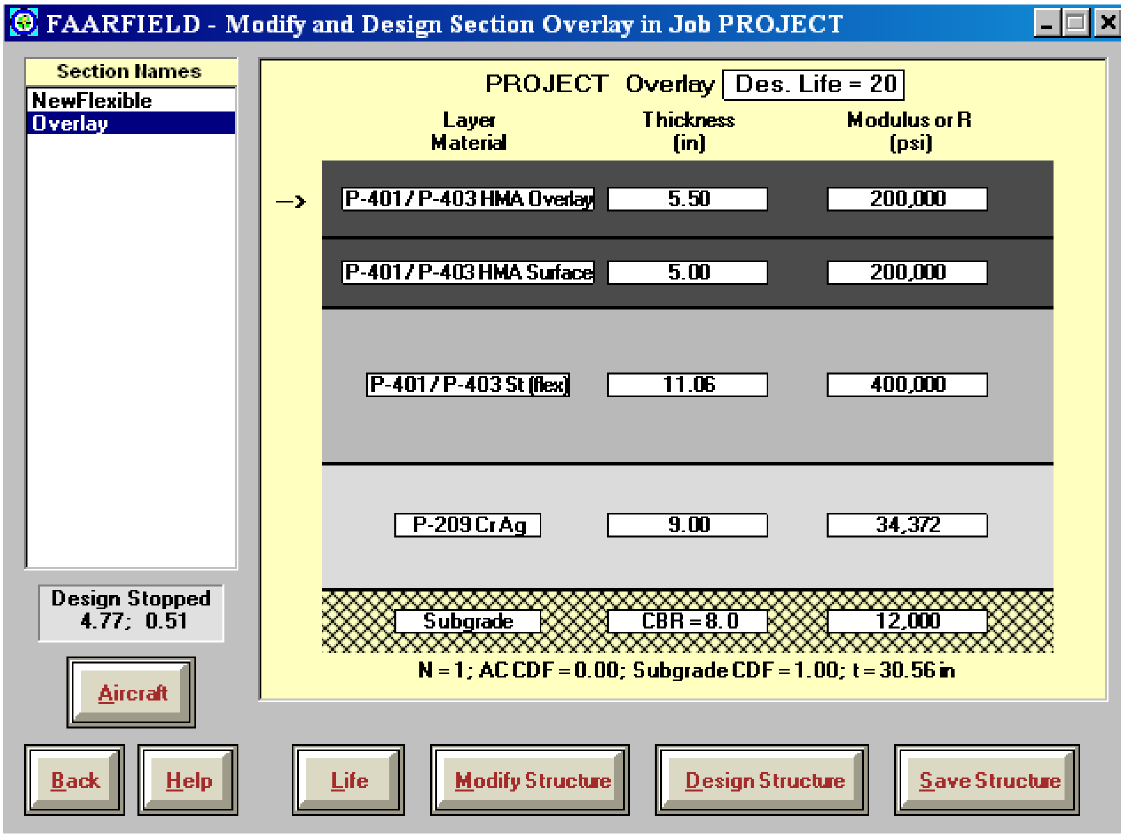


Figure 10. FAARFIELD flexible overlay design screen.

The difficulty in the process is selecting the inputs to accurately model the in-place properties of the existing pavement layers. If materials adhering to FAA specifications are selected, the layer moduli are fixed as described previously. Unfortunately, the Advisory Circular does not provide much guidance on selecting these inputs. Essentially, as noted in the introduction (chapter 1) this is the motivation behind this research study.

There is a process to assess the structural capacity of an existing PCC pavement structure, and that process is discussed herein to examine its potential applicability to flexible pavement overlay design. For existing PCC pavements, the structural condition of the existing pavement is expressed in terms of the Structural Condition Index (SCI). The SCI is derived from the Pavement Condition Index (PCI) and is essentially the summation of the structural elements of the PCI, as listed in table 4. As with the PCI, the SCI scale ranges from 100 (no structural distress) to 0. Because the SCI only considers the structural-related distresses, the SCI will always be equal to or greater than the PCI. The SCI replaces the C_b and C_r factors used to characterize the existing pavement condition in Advisory Circular 150/5320-6D procedure.

Table 4. Rigid pavement distresses used to calculate SCI (from Table 4-1, FAA 2009).

Distress	Severity Level
Corner break	Low, medium, high
Longitudinal/transverse/diagonal cracking	Low, medium, high
Shattered slab	Low, medium, high
Shrinkage cracking[1]	n/a
Joint spalling	Low, medium, high
Corner spalling	Low, medium, high

[1] Used to describe a load-induced crack that extends only part of the way across a slab. The SCI does not include conventional shrinkage cracks due to curing or other non-load-related problems.

An SCI of 80, corresponding to 50 percent of slabs in the traffic area exhibiting a structural crack, is defined as FAA's failure criterion. The failure curve for PCC pavements is illustrated in figure 11. If load-related distresses are observed on the pavement (in which case the SCI is less than 100), the cumulative fatigue damage used (CDFU) is defined as 100 percent. If there are no structural-related distress on the pavement (in which case the SCI is equal to 100), an additional step is required to determine where the pavement is on the failure curve (i.e., the CDFU). There are separate failure curves for CDFU for pavements with stabilized and aggregate bases. This is done within FAARFIELD by modeling the existing pavement in Life mode, rather than Design mode, to determine the CDFU for the traffic applied to date. This value is then entered, and the overlay thickness is designed as usual.

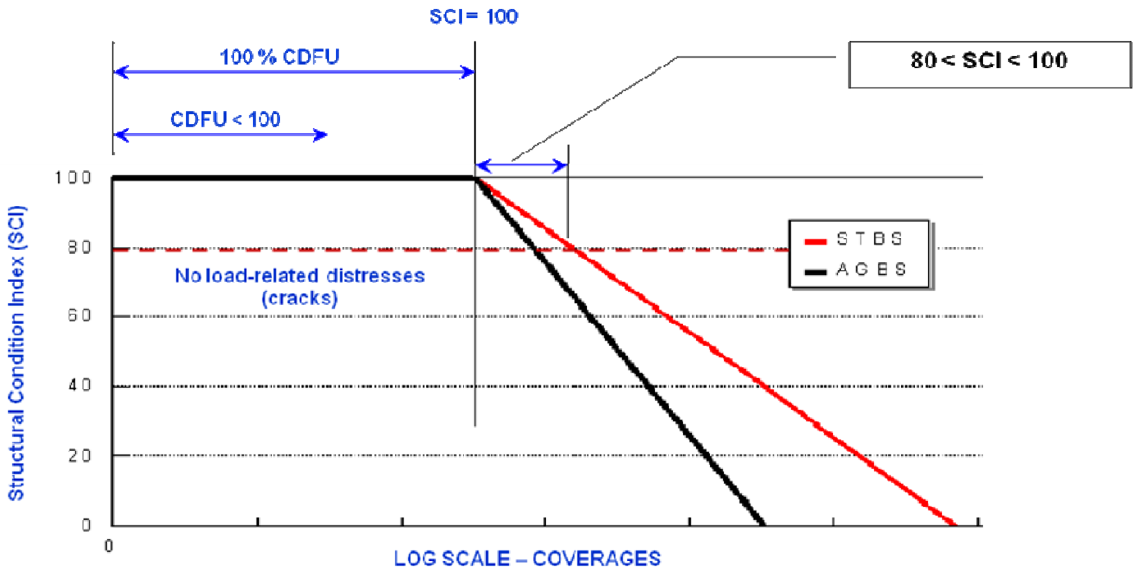


Figure 11. Rigid pavement failure curve.

SCI and CDFU are not currently used as indicators of the existing structural capacity of HMA pavements but offer one potential method of characterizing the existing pavement and will be explored under this study.

Unified Facilities Criteria (UFC) Overlay Design Procedures

The airfield pavement design procedures for the U.S. Army Corps of Engineers, Naval Facilities Engineering Command, and Air Force Civil Engineer Support Agency are jointly published in UFC 3-260-02, *Pavement Design for Airfields* (Army 2001). These procedures follow the same general approach, although there are differences as to how some items are handled (such as traffic levels and minimum base/subbase course thickness requirements). The UFC provides two methods for thickness design of flexible pavements: (a) CBR method and (b) layered elastic method. Both approaches are included in the pavement design and analysis program, Pavement-Transportation Computer Aided Structural Engineering (PCASE).

CBR Method

This method involves the determination of the CBR of the material to be used for a given layer. This CBR is then used to determine the thickness required above the layer to prevent deformation in that layer using the appropriate design nomograph. The design philosophy and process are nearly identical to the FAA's design approach in Advisory Circular 150/5320-6D. Sample nomographs for the Army and Air Force are presented in figures 12 and 13, respectively. Additional considerations are provided for poor subgrades (CBR < 3), frost protection, expansive soils, and saturated subgrades.

The overlay design method for an HMA overlay of an existing HMA pavement is also similar to the design procedure outlined in FAA Advisory Circular 150/5320-6D. This method is essentially a thickness deficiency method where the thickness of a new flexible pavement is compared to the existing pavement structure and the overlay thickness is determined by the deficiency. As with the conventional FAA design method, equivalency factors are used to convert the thickness of a higher quality material to a lower quality material. Figure 14 shows the PCASE screen to determine the required HMA overlay thickness.

Layered Elastic Method

This design procedure attempts to eliminate/reduce cracking of the HMA surface course and rutting in the wheel path by limiting the horizontal tensile strain induced at the bottom of the HMA layer(s) and the vertical compressive strain on the subgrade. A cumulative damage concept is used to handle the variations in the HMA properties and the subgrade strength caused by cyclic climatic conditions.

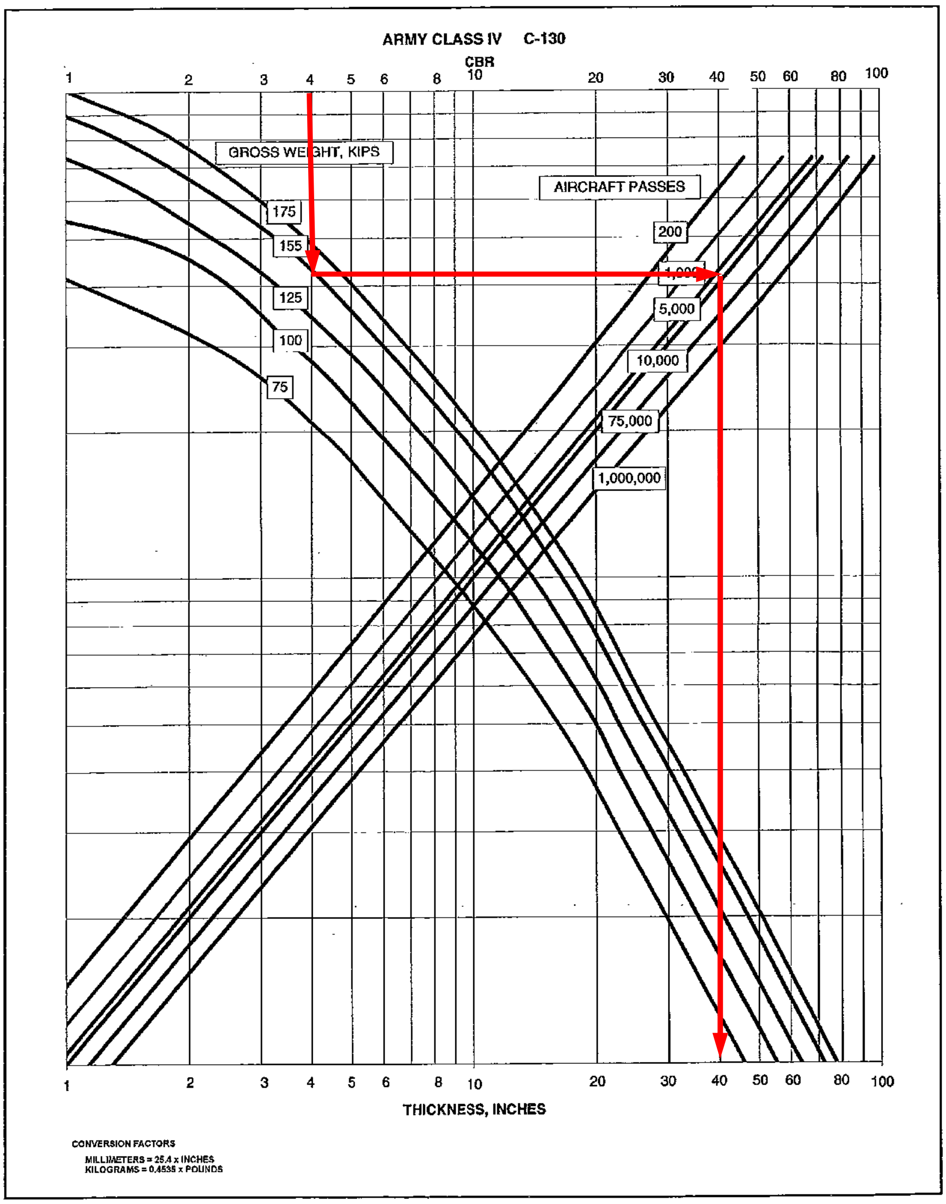


Figure 12. Flexible pavement design curve for Army Class IV airfields (C-130 aircraft) with runway ≤ 5,000 feet, types B and C traffic areas (from Army 2001).

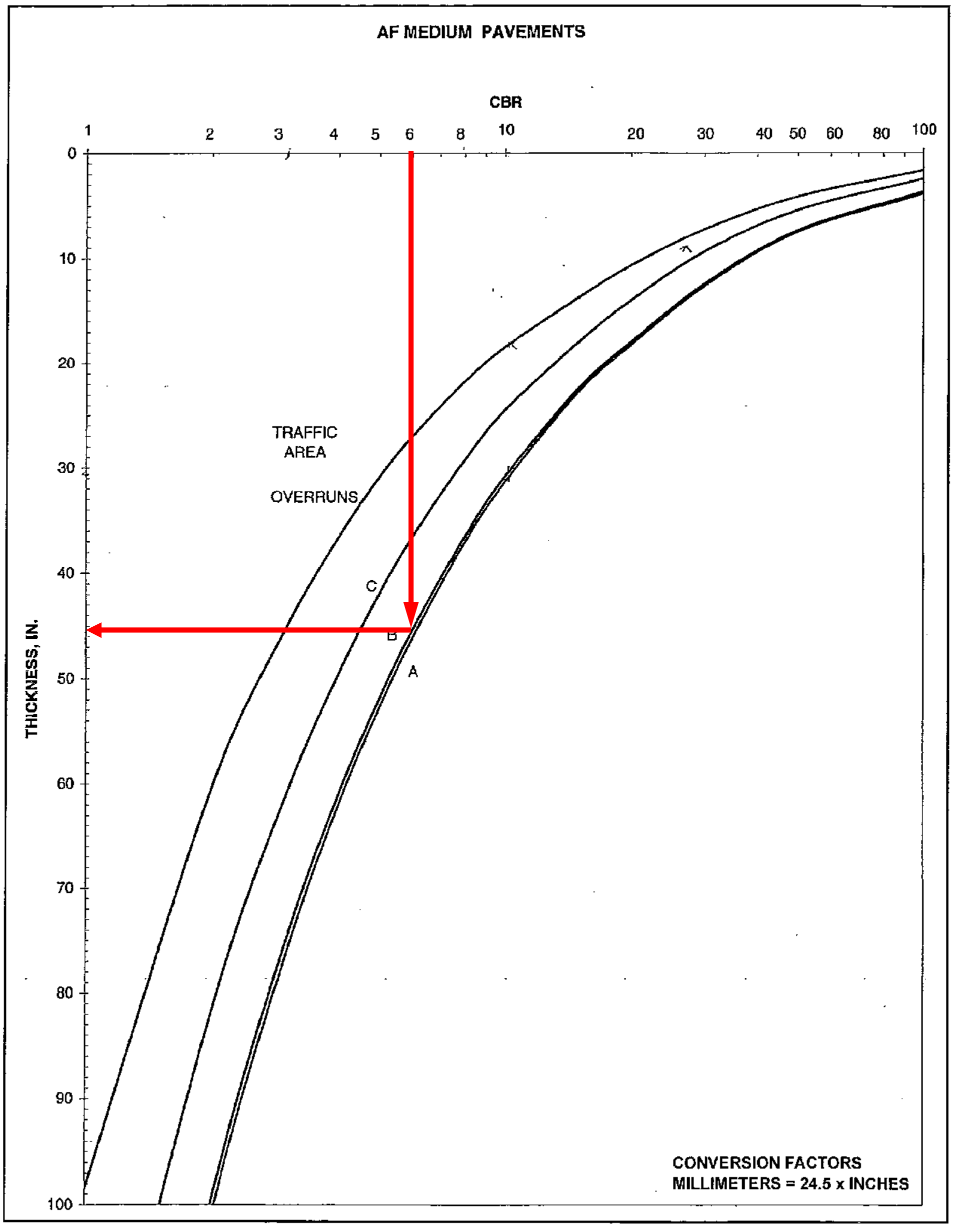


Figure 13. Flexible pavement design curve for Air Force heavy-load pavement (from Army 2001).

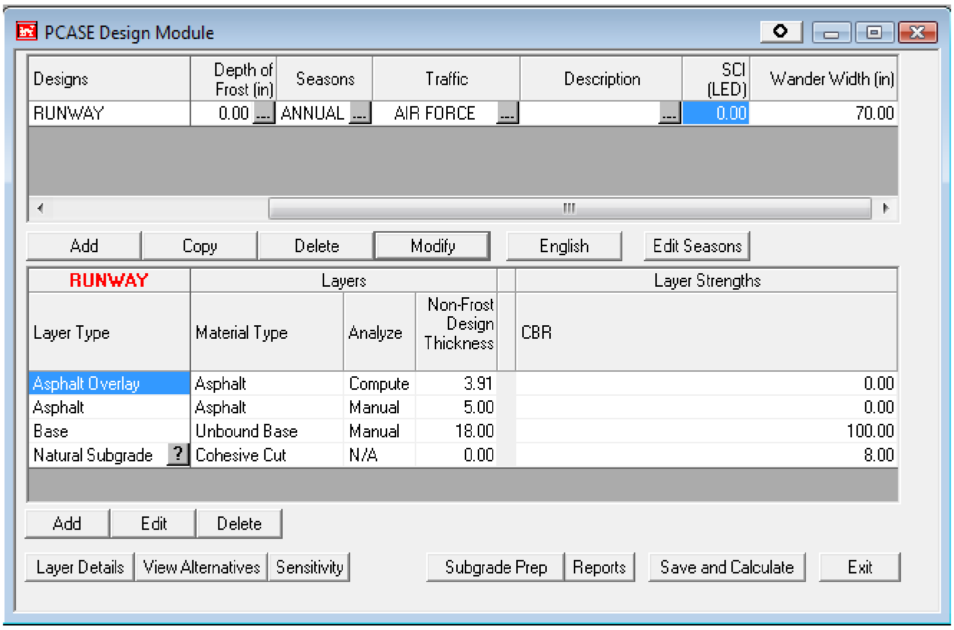


Figure 14. PCASE overlay design screen using CBR method.

The approach is similar to that used in FAA Advisory Circular 150/5320-6E with some notable differences. For one, the pavement responses are determined using the elastic layer program, JULEA (FAA uses the LEAF program). The performance curves are also different. The subgrade strain model (to evaluate rutting in the subgrade), which was developed from the analysis of field test data, is defined by the following equation:

$$N = 10{,}000\left(\frac{0.000247 + 0.000245\log M_R}{S_S}\right)^{0.0658 M_R^{0.559}} \qquad \text{(Eq. 2)}$$

where:

N = Allowable repetitions
M_R = Resilient modulus of the subgrade, psi
S_S = Vertical strain at the top of the subgrade, in/in

The model to evaluate fatigue in the HMA is defined by the following equation:

$$N = 10^{2.68 - 5.0\log S_A - 2.665\log E} \qquad \text{(Eq. 3)}$$

where:

N = Allowable strain repetitions
S_A = Tensile strain of asphalt, in/in
E = Elastic modulus of HMA, psi

These equations were developed using provisional laboratory fatigue data from Heukelom and Klomp (1962).

There are a few other notable highlights of this design procedure. The subgrade support is defined by the resilient modulus, as determined from the repetitive triaxial test. A minimum of six tests are recommended for each soil group. Also, different seasons can be defined to account for variations in subgrade support conditions, pavement temperatures, and moisture conditions throughout the year.

For design of overlays over an existing PCC pavement, the structural condition of the existing PCC pavement is taken into consideration by using a "condition factor" in the thickness design equations. However, as with the FAA design procedure, a similar approach to account for the structural condition of the existing HMA pavement is not available.

The Asphalt Institute Overlay Design Procedure

As with the previous procedures discussed, the Asphalt Institute overlay design procedure is based on a structural deficiency approach that builds off the requirements for new design. Therefore, it is important to first introduce and understand the new design procedure.

The Asphalt Institute pavement design procedure involves the determination of the thickness to satisfy two different strain criteria: (a) the vertical compressive strain at the top of the subgrade layer and (b) the horizontal tensile strain at the bottom of the HMA layer. The thickness for each criterion is determined, and the greater requirement is selected as the design cross section.

The following inputs are required for development of flexible pavement structural designs using the Asphalt Institute method:

- Design Subgrade Value – This value represents the subgrade resilient modulus that is equal to or greater than 85 percent of the values in the lot (six to eight samples are recommended). The resilient moduli can be determined directly from laboratory testing or can be approximated from other tests (CBR, Resistance Value, or plate bearing tests).
- Mean Annual Air Temperature – The 30-year mean annual temperature is recommended for design. The purpose of this input is to account for the effect of temperature on HMA pavement performance.
- Projected Aircraft Traffic Mix – Traffic is represented by a 358,000-lb DC-8-73 aircraft, which is defined as the "standard" aircraft. All aircraft are converted to the standard aircraft through the use of equivalency factors that convert the number of strain applications of a given aircraft to the equivalent strain applications of the standard aircraft. Figure 15 shows a sample curve for determining the equivalency factor. Charts are available for different aircraft types, different load levels (for some aircraft), and different pavement thicknesses. The "x" on the chart represents the lateral offset from the centerline, each of which needs to be checked.

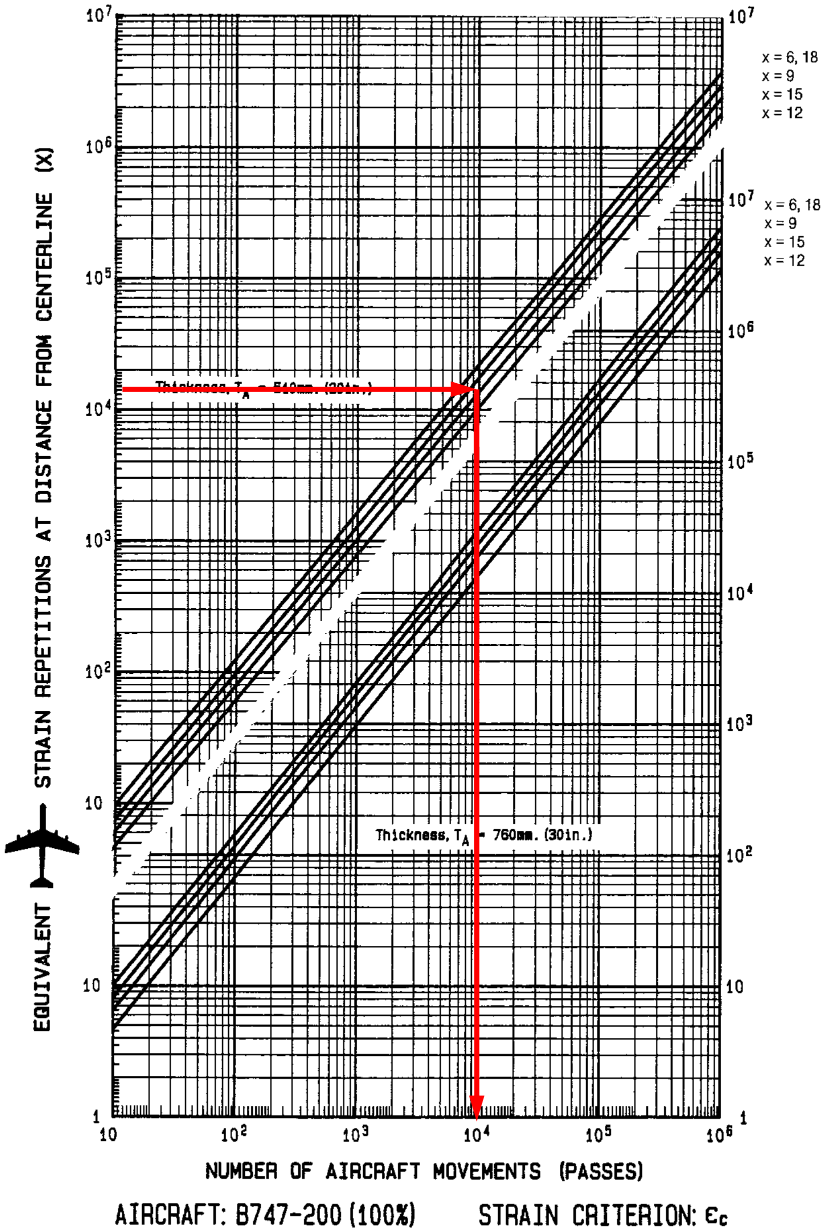


Figure 15. Equivalent strain repetitions for B747-200 (from Figure X-3, AI 1987).

From these inputs, each strain criterion is analyzed following these three steps (these steps are illustrated graphically in figure 16):

1. Determine the Allowable Traffic Value, N_a – This variable represents the number of equivalent strain repetitions of the standard aircraft that a full-depth HMA pavement can withstand for the specific inputs. This value is determined from a family of curves for each strain criterion. Figure 17 shows two sample charts to evaluate the subgrade vertical compressive strain on the subgrade; similar charts are available to evaluate the HMA horizontal tensile strain.
2. Determine the Predicted Traffic Value, N_p – This variable represents the number of equivalent strain applications of the standard aircraft that will actually occur during the selected time period. The design manual outlines an 11-step process for determining this value for each strain criterion, which includes the determination of the critical strain location and cumulative (from each aircraft type) applications of the equivalent standard aircraft (see figure 18 for a sample worksheet).
3. Determine the full-depth HMA pavement thickness, T_A – This is the required thickness to satisfy the strain criteria for the specific inputs. It is determined by a simultaneous solution of N_a and N_p. The design thickness is selected as the greater of the thicknesses required for the two strain criteria, rounded to the nearest ½ inch. Figure 19 shows a sample chart used to determine the required thickness to limit subgrade vertical compressive strain.

The overlay design procedure follows a structural deficiency approach in which a new pavement structure is designed and compared to the existing pavement structure. For the Asphalt Institute procedure, the existing pavement is characterized by the effective thickness. The steps involved in the Asphalt Institute overlay design method are described below:

1. Determine the design subgrade resilient modulus, mean annual air temperature, and projected aircraft traffic for the design period.
2. Determine the thickness of a full-depth HMA pavement (T_A) required to satisfy the subgrade, environmental, and the traffic factors (i.e., determine the required new pavement design as previously outlined).
3. Convert the thickness of the existing pavement structure into the Effective Thickness (T_e) by using the appropriate conversion factors (see table 5 for guidance).
4. Determine the required HMA overlay thickness as the difference between the full-depth HMA pavement thickness and the effective thickness of the existing pavement structure ($T_A – T_e$).

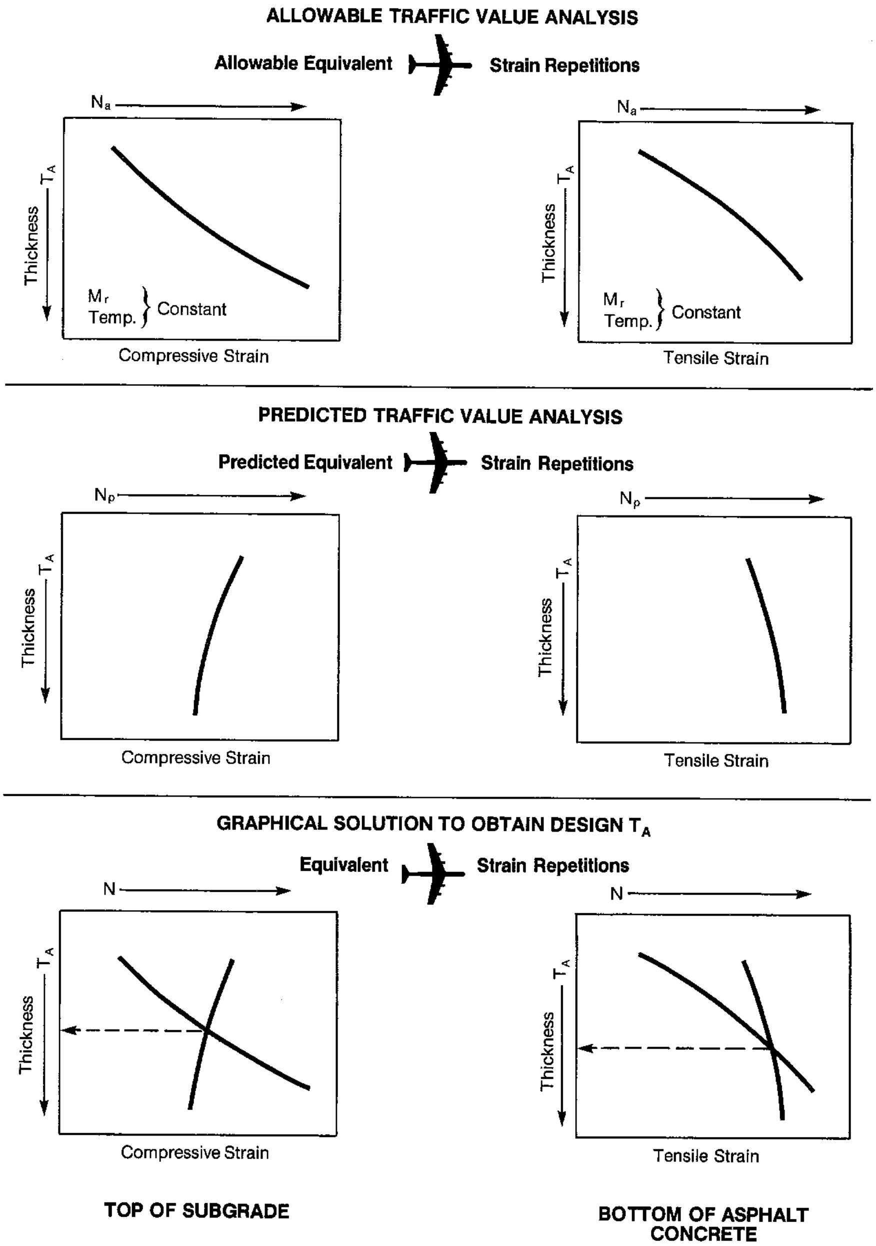


Figure 16. Steps to determine design thickness (from Table V-2, AI 1987).

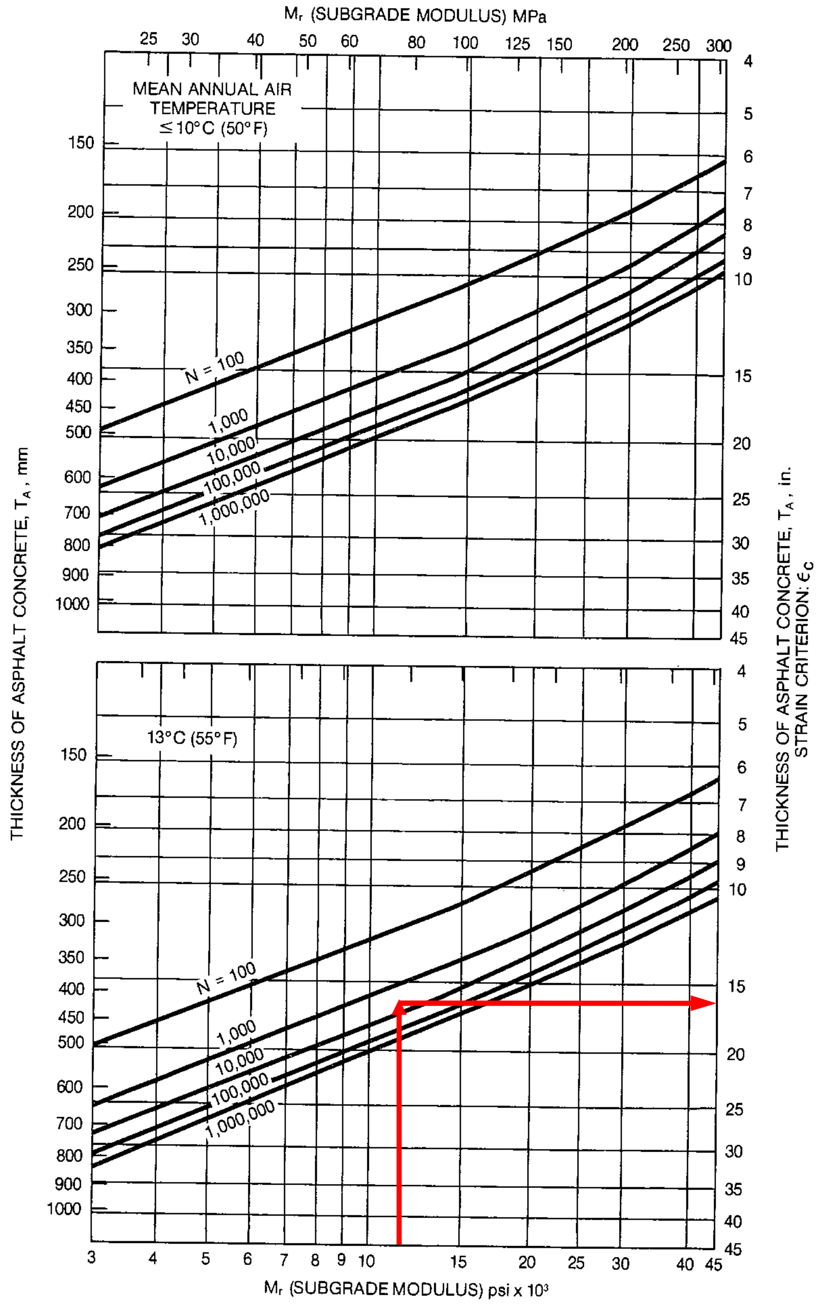


Figure 17. Required pavement thickness to limit subgrade vertical compressive strain (from Figure V-3, AI 1987).

NUMBER OF EQUIVALENT STRAIN REPETITIONS

TYPE OF STRAIN: ε_c DESIGN PERIOD: 20 YEARS

AIRCRAFT	MOVEMENTS IN DESIGN PERIOD	DISTANCE FROM CENTERLINE, x				
		1.8m (6 ft)	2.7m (9 ft)	3.7m (12 ft)	4.6m (15 ft)	5.5m (18 ft)
THICKNESS, T_A = 250 mm (10 in.)						
1. B767-200	142,000	–	1,600	3,300	4,100	3,700
2. DC-8-73	70,500	28,000	59,000	59,000	28,000	6,000
3. B747-SP	61,500	8,000	6,900	5,100	6,200	8,000
4. DC-10-10 (100%)	27,250	–	20,000	180,000	600,000	800,000
5. DC-10-10 (80%)	27,250	–	1,700	7,500	18,000	21,000
6. MD-82	20,200	1,100	1,300	950	400	–
7.						
8.						
9.						
10.						
SUM		37,100	90,500	255,850	656,700	838,700
THICKNESS, T_A = 510 mm (20 in.)						
1. B767-200	142,000	–	530	900	1,200	1,000
2. DC-8-73	70,500	28,000	59,000	59,000	28,000	6,000
3. B747-SP	61,500	1,700	1,600	1,200	1,500	1,700
4. DC-10-10 (100%)	27,250	–	5,100	33,000	100,000	130,000
5. DC-10-10 (80%)	27,250	–	370	900	1,500	1,700
6. MD-82	20,200	400	450	350	190	–
7.						
8.						
9.						
10.						
SUM		30,100	67,050	95,350	132,390	140,400
THICKNESS, T_A = 760 mm (30 in.)						
1. B767-200	142,000	–	300	490	590	540
2. DC-8-73	70,500	28,000	59,000	59,000	28,000	6,000
3. B747-SP	61,500	2,400	2,100	1,700	1,900	2,400
4. DC-10-10 (100%)	27,250	–	2,700	16,000	40,000	50,000
5. DC-10-10 (80%)	27,250	–	140	310	480	530
6. MD-82	20,200	100	110	90	60	–
7.						
8.						
9.						
10.						
SUM		30,500	64,350	77,590	71,030	59,470
THICKNESS, T_A = 1020 mm (40 in.)						
1. B767-200	142,000	–	230	350	420	390
2. DC-8-73	70,500	28,000	59,000	59,000	28,000	6,000
3. B747-SP	61,500	17,000	14,000	9,200	12,000	17,000
4. DC-10-10 (100%)	27,250	–	1,900	9,900	24,000	29,000
5. DC-10-10 (80%)	27,250	–	100	210	320	350
6. MD-82	20,200	20	21	19	15	
7.						
8.						
9.						
10.						
SUM		45,020	75,251	78,679	64,755	52,740

NOTE: Circle the maximum sum for each thickness. Each circled number and its corresponding thickness is used to plot one point of the Actual Traffic Value Curve.

Figure 18. Sample aircraft traffic worksheet (from Figure V-12, AI 1987).

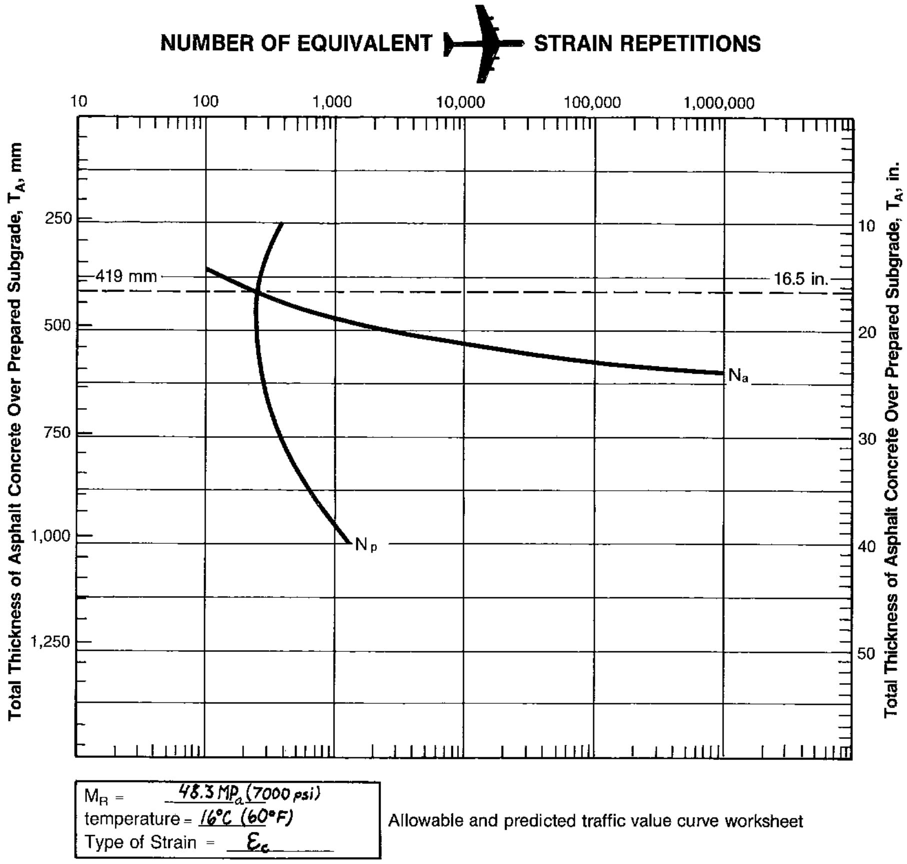


Figure 19. N_s and N_p curves for subgrade vertical compressive strain (from Figure V-7, AI 1987).

As in the conventional FAA design method, considerable variability is associated with the selection of the conversion factors for converting the thickness of the existing pavement structure to an effective thickness for overlay design. The guidance, however, is more thorough than that presented in Advisory Circular 150/5320-6D and offers a potential approach that could be used in FAARFIELD.

The design procedure also provides guidance for selecting the required HMA overlay thickness of an existing PCC pavement, as presented in figure 20. The overlay thicknesses in this chart are based on the slab length and mean annual temperature differential and are designed to minimize the effects of both horizontal tensile strains and vertical shear stresses.

Table 5. Asphalt Institute conversion factors (from Table VII-1, AI 1987).

(These conversion factors apply ONLY to pavement evaluation for overlay design. In no case are they applicable to original thickness design.)

Classification of Material	Description of Material	Conversion Factors*
I	a) Native subgrade in all cases b) Improved Subgrade**–predominantly granular materials–may contain some silt and clay but have P.I. of 10 or less c) Lime modified subgrade constructed from high plasticity soils–P.I. greater than 10.	0.0
II	Granular subbase or base–reasonably well-graded, hard aggregates with some plastic fines and CBR not less than 20. Use upper part of range if P.I. is 6 or less; lower part of range if P.I. is more than 6.	0.1-0.2
III	Cement or lime-fly ash stabilized subbases and bases** constructed from low plasticity soils–P.I. of 10 or less.	0.2-0.3
IV	a) Emulsified or cutback asphalt surfaces and bases that show extensive cracking, considerable raveling or aggregate degradation, appreciable deformation in the wheel paths, and lack of stability. b) Portland cement concrete pavements (including those under asphalt surfaces) that have been broken into small pieces 0.6 meter (2 ft) or less in maximum dimension, prior to overlay construction. Use upper part of range when subbase is present; lower part of range when slab is on subgrade. c) Cement or lime-fly ash stabilized bases** that have developed pattern cracking, as shown by reflected surface cracks. Use upper part of range when cracks are narrow and tight; lower part of range with wide cracks, pumping or evidence of instability.	0.3-0.5
V	a) Asphalt concrete surface and base that exhibit appreciable cracking and crack patterns. b) Emulsified or cutback asphalt surface and bases that exhibit some fine cracking, some raveling or aggregate degradation, and slight deformation in the wheel paths but remain stable. c) Appreciably cracked and faulted portland cement concrete pavement (including such under asphalt surfaces) that cannot be effectively undersealed. Slab fragments, ranging in size from approximately one to four square meters (yards), and have been well-seated on the subgrade by heavy pneumatic-tired rolling.	0.5-0.7
VI	a) Asphalt concrete surfaces and bases that exhibit some fine cracking, have small intermittent cracking patterns and slight deformation in the wheel paths but remain stable. b) Emulsified or cutback asphalt surface and bases that are stable, generally uncracked, show no bleeding, and exhibit little deformation in the wheel paths. c) Portland cement concrete pavements (including such under asphalt surfaces) that are stable and undersealed, have some cracking but contain no pieces smaller than about one square meter (yard).	0.7-0.9
VII	a) Asphalt concrete, including asphalt concrete base, generally uncracked, and with little deformation in the wheel paths. b) Portland cement concrete that is stable, undersealed, and generally uncracked. c) Portland cement concrete base, under asphalt surface, that is stable, non-pumping and exhibits little reflected surface cracking.	0.9-1.0

*Values and ranges of Conversion Factors are multiplying factors for conversion of thickness of existing structural layers to equivalent thickness of asphalt concrete.

**Originally meeting minimum strengths and compaction requirements.

TEMPERATURE DIFFERENTIAL* (°C)

Slab Length (m)	17	22	28	33	39	44	Slab Length (Ft)
3	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	10 or Less
4.5	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	15
6	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	125mm (5 in.)	140mm (5.5 in.)	20
7.5	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	125mm (5 in.)	150mm (6 in.)	175mm (7 in.)	25
9	100mm (4 in.)	100mm (4 in.)	125mm (5 in.)	150mm (6 in.)	175mm (7 in.)	200mm (8 in.)	30
10.5	100mm (4 in.)	115mm (4.5 in.)	150mm (6 in.)	175mm (7 in.)	215mm (8.5 in.)	Use Alternative 2 or 3	35
12	100mm (4 in.)	140mm (5.5 in.)	175mm (7 in.)	200mm (8 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	40
13.5	115mm (4.5 in.)	150mm (6 in.)	190mm (7.5 in.)	225mm (9 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	45
15	125mm (5 in.)	175mm (7 in.)	215mm (8.5 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	Use Alternative 2 or 3	50
18	150mm (6 in.)	200mm (8 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	Use Alternative 2 or 3	Use Alternative 2 or 3	60
	30	40	50	60	70	80	

TEMPERATURE DIFFERENTIAL* (°F)

*Temperature differential (Δ_t) is the difference between the highest normal daily maximum temperature and the lowest normal daily minimum temperature for the hottest and coldest months, based on a 30-year average. See Tables VII-2 and VII-3 for maximum and minimum daily temperature at locations throughout the United States.

NOTE: There are several alternatives to the thicknesses in Sections B and C of this chart. See Art 7.09.

Figure 20. Chart for selecting HMA overlay thickness for a PCC pavement (from Figure VII-6, AI 1987).

SUMMARY OF DESIGN PARAMETERS

Table 6 presents a summary of the design inputs required for both new flexible pav design and HMA overlay design for the procedures discussed. The failure criteria for new overlay design, and the overlay design approach, are also presented for each procedure.

Table 6. Summary of design inputs for flexible pavement design procedures.

	FAA 150/5320-6D	FAA 150/5320-6E	U.S. Army COE		Asphalt Institute
			CBR	Layer Elastic	
New Design Inputs					
Subgrade CBR	X	X	X		
Subgrade Resilient Modulus		X		X	X
Subgrade Moisture Content				X	
Subbase CBR	X		X		
Subbase Modulus		X		X	
Base CBR			X		
Base Modulus		X		X	
HMA Surface Modulus		X			
HMA Surface Thickness		X			
Poisson's Ratio (all layers)				X	
Slip (bonding condition)				X	
Aircraft Type/Model	X	X	X	X	X
Take-off Weight	X	X	X	X	
Annual Departures	X	X	X	X	
Annual Growth Rate	X	X	X	X	
Traffic Area Type			X	X	
Total Movements					X
Design Life	X	X	X	X	
Average Daily Max. Temp. (by month)				X	
Average Daily Mean Temp. (by month)				X	
Mean Annual Temperature					X
Failure Criteria	Subgrade Rutting	Subgrade Rutting HMA Fatigue (option)	Subgrade Rutting	Subgrade Rutting HMA Fatigue	Subgrade Rutting HMA Fatigue
Overlay Design Inputs					
Layer Type	X	X	X	X	X
Layer Thickness	X	X	X	X	X
Layer Condition	X				X
Equivalency Factors	X				
Conversion Factors					X
HMA Overlay Modulus		X		X	
Normal Daily Maximum Temperature					X
Normal Daily Minimum Temperature					X
PCC Slab Length					X
Failure Criteria	Subgrade Rutting	Subgrade Rutting HMA Fatigue (option)	Subgrade Rutting	Subgrade Rutting HMA Fatigue	Subgrade Rutting HMA Fatigue
Overlay Design Approach	Str. Deficiency	Limiting Strain on Subgrade	Str. Deficiency	Limiting Strain	Strain Deficiency

REVIEW OF PERFORMANCE CRITERIA

Subgrade Rutting

The current FAA design method uses the vertical strain at the top of the subgrade layer as the controlling criterion, which is designed to control rutting in the subgrade layer only (note: FAA design assumes the other layers will not rut if they are designed and constructed in accordance with their construction specifications). The user has the option of enabling the asphalt strain computation by deselecting the “No AC CDF” check box in the Options screen of the FAARFIELD program, but HMA fatigue rarely will control the design results within FAARFIELD.

The FAA failure model used to estimate the number of coverages to failure for a given vertical strain at the top of the subgrade is presented in the following equations (used in both LEDFAA 1.3 and FAARFIELD):

$$C = \left(\frac{0.004}{\varepsilon_v}\right)^{8.1} \quad \text{when } C \le 12{,}100 \qquad \text{(Eq. 4)}$$

$$C = \left(\frac{0.002428}{\varepsilon_v}\right)^{14.21} \quad \text{when } C > 12{,}100 \qquad \text{(Eq. 5)}$$

where:

C = Number of coverages to failure
ε_v = Vertical strain at the top of the subgrade (computed using the LEAF subroutine in FAARFIELD)

The subgrade rutting performance criteria presented above is originally based on the models developed by the U.S. Army Waterways Experiment Station (WES) from the analysis of field test data. The analysis identified that the relationship between allowable repetitions and strain magnitude is slightly different for subgrade soils with different resilient moduli. The failure model is of the form:

$$C = 10{,}000\left(\frac{0.000247 + 0.000245.\log_{10}(E_{SG})}{\varepsilon_v}\right)^{0.0658 E_{SG}^{\;0.559}} \qquad \text{(Eq. 6)}$$

This model expresses the coverages to failure, C, as a function of the vertical strain at the top of the subgrade, ε_v, and subgrade modulus, E_{SG}. When LEDFAA 1.2 was developed, it was found to be difficult to make the thickness designs compatible with those of FAA Advisory Circular 150/5320-6D over the full practical range of subgrade strength and aircraft departures. When LEDFAA 1.3 was developed, the developers were able to achieve compatibility between the design procedures by using a failure model independent of the subgrade modulus. A two-slope model was proposed, since the original model produced highly conservative designs at high coverages. The slope of equation 5 is the same as equation 6 with the subgrade modulus set at 15,000 psi (Hayhoe, Kawa, and Brill 2004). A comparison of the LEDFAA 1.2 failure model to the LEDFAA 1.3/FAARFIELD failure models is shown in figure 21.

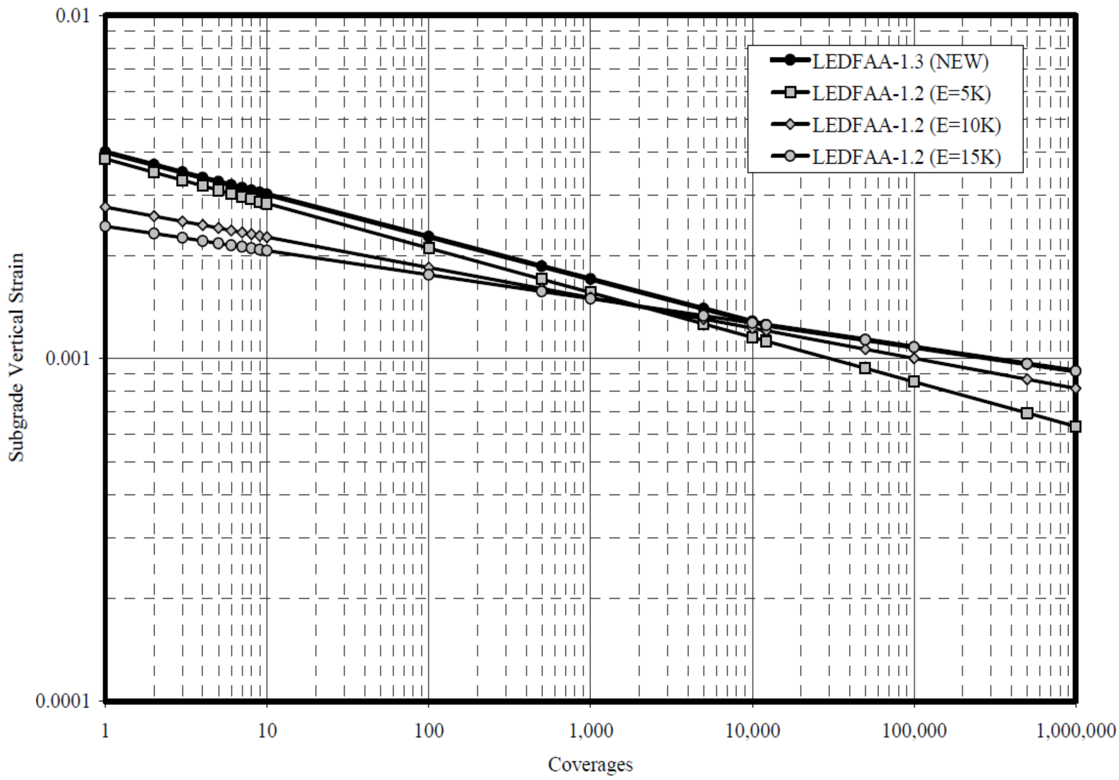


Figure 21. Comparison of subgrade failure models (Garg, Guo, and McQueen 2004).

The subgrade failure model is based purely on compressive strain at the top of subgrade and is independent of subgrade material properties. Some research suggests that stress-strain relationships in the subgrade are not adequately accounted for and the performance model does not address deformation in other pavement layers (Bejarano and Thompson 1999).

HMA Fatigue

Fatigue cracking is another critical parameters associated with the performance of HMA overlays. Repeated loading causes cracking at locations where the critical tensile strains and stresses occur. Extensive research has been conducted to predict the occurrence of bottom-up fatigue (alligator) cracking using both empirical and mechanistic-empirical methodologies.

As previously mentioned, fatigue cracking is not one of the default failure criteria in FAARFIELD, but can be activated by the user if desired. The FAA fatigue cracking failure model is based on the horizontal tensile strain at the bottom of the HMA surface (or overlay) layer, as follows:

$$\log_{10}(C) = 2.68 - 5 * \log_{10}(\varepsilon_h) - 2.665 * \log_{10}(E_A) \quad \text{(Eq. 7)}$$

where:

C = Number of coverages to failure.
ε_h = Horizontal strain at bottom of surface HMA layer.
E_A = HMA modulus, psi.

The tensile strains at the bottom of the underlying HMA or other stabilized layers were not historically considered in the FAA design. However, the FAA has provided the option to evaluate the tensile strain at the bottom of all HMA layers in the latest version of FAARFIELD. The HMA overlay is considered to be a fully bonded to the underlying layer in FAARFIELD, so the critical tensile strains would occur at bottom of the underlying HMA layers and not the surface layer (or overlay). It is likely that for many overlay designs, the strain at the bottom of the HMA overlay is actually a compressive strain rather than tensile strain. As discussed further in the sensitivity analysis in Chapter 3, fatigue of the HMA surface layer was never the controlling failure mode for the evaluated sections.

In empirical models, fatigue damage is correlated to loads, pavement layer thickness, environmental factors, pavement age, and other factors. The South Dakota Department of Transportation developed an empirical fatigue cracking regression model, which is of the following form:

$$F = 100 - 0.11726\ AGE^{2.2} \qquad \text{(Eq. 8)}$$

The Fatigue Cracking Index (F), which ranges from 0 to 100, is dependent on the current age of the pavement and is determined through expert opinion and regression analysis. Since pavement age is the only variable included in this model, it is not possible to use this model to study the impact of traffic, climate, and pavement structure (Jackson et al. 1996). Aliand and Tayabji (1998) also developed a method to predict fatigue cracking in terms of damage. They correlated a damage ratio to fatigue cracking using growth curves. Since empirical models can be applied only to specific data sets or area, they cannot be used universally. Hence, researchers began to realize the importance of mechanistic-empirical based fatigue cracking models.

Freeman et al. (2009) found that aged HMA does not necessarily perform the same as new HMA, based on the evaluation of HMA pavements at several military airfields. With current fatigue equations, the allowable load applications decreases as the modulus increases. However, testing of aged HMA samples suggests that the observed decrease in fatigue in the current equations is a result of the conditions under which the models were developed (Freeman et. al 2008). The research conducted under the study suggests fatigue performance increases as the modulus (or stiffness) increases. From that study, the fatigue criteria were developed for new and aged HMA pavements. For new HMA, the equation takes the following form:

$$\mathrm{Log}_{10}(r) = 2.68 - 5.\mathrm{Log}_{10}(S_A) - 2.665.\mathrm{Log}_{10}(E) \qquad \text{(Eq. 9)}$$

where:

r = Allowable strain repetition.
S_A = Tensile strain of HA, in/in.
E = Elastic modulus of HMA, psi.

For aged HMA, the following equation was developed:

$$Log10(r) = 7.94 - \left[\frac{LN(\mu s}{2.61}\right]^2 + \frac{E}{438{,}000\,psi} \qquad \text{(Eq. 10)}$$

where:

r = Allowable strain repetition for aged, field HMA.
$\mu\varepsilon$ = Microstrain of HMA, in/in x 10^{-6}.
E = HMA standardized in-place modulus, psi.

Mechanistic-empirical predictive models correlate the fatigue damage to the critical tensile strain and the stiffness of the HMA layer. The following equation shows a universal form of the fatigue cracking failure model for flexible pavements (Finn, 1973; Finn et al. 1973, 1977):

$$N_f = k_1\left(\frac{1}{\varepsilon_t}\right)^{k_2}\left(\frac{1}{E}\right)^{k_3} \qquad \text{(Eq. 11)}$$

where:

N_f = Number of load repetitions to fatigue cracking.
ε_t = Tensile strain at the critical location.
E = HMA stiffness.
k_1, k_2, k_3 = Regression coefficients.

The MEPDG also uses a model similar to the form described in equation 11 to predict fatigue cracking susceptibility.

The location of the critical stresses and strains depends on several factors; the most important factors include the stiffness of the layer and the aircraft gear configuration. Fatigue cracking typically initiates at the bottom of the HMA layer along the wheel path where maximum strain occurs. The cracks eventually propagate to the surface and initially manifest as one or more longitudinal cracks; after repeated loading, the cracks become interconnected and spread over larger areas.

The maximum horizontal tensile strain developed in the pavement system might not be the most critical value. Because the stiffness of a layered asphalt pavement structure varies with depth, it will eventually determine the location of the critical strain that causes fatigue failure.

Some of the reasons for higher magnitudes of tensile stresses and strains to occur at the bottom of all bound HMA layers include:

- Relatively thin or weak HMA layers for the magnitude and repetitions of imposed loading.
- Higher wheel loads and tire pressures than used for designed.
- Localized weak areas in unbound granular base course or subgrade soils.
- Weak unbound granular layers due to inadequate compaction, high moisture content, or the presence of a high ground water table.

The FAA design approach should employ a similar method and consider the tensile stresses and strains at the bottom of all bound layers rather than just the surface HMA layer.

As noted previously, the FAA recently released an updated version of FAARFIELD (version 1.305) that incorporates the computation of the fatigue damage in each HMA layer (not just the surface layer). The default failure model for HMA fatigue continues to be the Heukelom and Klomp (1962) equation, which is implemented in FAARFIELD 1.302. However, a new HMA fatigue failure model based on the ratio of dissipated energy change (RDEC) approach is being evaluated in the upcoming version (Shen and Carpenter 2007). The RDEC approach provides a methodology to study fatigue performance of airport pavements at low strain levels with shortened fatigue testing. The parameters of the RDEC model can be adjusted based on gradation parameters, volumetrics, and dynamic modulus of the HMA mixture. This model, however, was not evaluated as a part of this study.

Other Performance Models

While the available design programs address subgrade rutting and fatigue of the bound layers, there are other performance measures that can be considered. The following provides a discussion of work performed on modeling indicators such as deformation in the HMA layers, deformation of granular layers, and others.

HMA Permanent Deformation

As mentioned previously, permanent deformation (rutting) within the HMA layers is not considered in FAA pavement design. The FAA claims to handle the HMA layer rutting through proper asphalt mix design. In other words, they claim that rutting in the HMA should not occur, at least not to the extent that it needs to be considered in pavement thickness design, if the material is designed and constructed using their P-401 specification. This view is not shared by all designers.

The HMA surface can experience rutting under high wheel pressures and/or high temperatures, particularly where aircraft "stacking" (or static loading) may occur. Rutting in the HMA layer can occur from additional densification of the material under loading or from shear failure within the layer. Although most research on HMA surface rutting has been conducted for roadway pavements (with tire pressures typically 100 psi or less), some research has been conducted for airport pavements (tire pressures around 200 psi), and work performed by Monismith et al. (2000) is presented.

The model developed is based on the maximum shear stress and the corresponding shear strain. Monismith et al. (2000) indicate that permanent shear strain in the HMA, under simple loading, is assumed to accumulate according to the following expression:

$$\gamma^i = a' \cdot \exp(b'\tau)\gamma^e n^c \qquad \text{(Eq. 12)}$$

where:

γ^i = Permanent (inelastic) shear strain at a 2-inch depth.
τ = Shear stress determined at this depth using multilayer elastic analysis.
γ^e = Corresponding elastic shear strain.
n = Number of axle load repetitions.
a', b', c = Experimentally determined coefficients.

The permanent strain is then related to rut depth with the following general relationship:

$$rd_{hma} = k(\gamma_p)_{\max} \qquad \text{(Eq. 13)}$$

where:

rd_{hma} = Rut depth in HMA, inches.
k = Coefficient relating strain to rut depth (3 to 4 for 4-inch HMA layer).
$(\gamma_p)_{\max}$ = Maximum permanent strain, in/in.

Based on the research conducted, the permanent strain is estimated as follows:

$$\gamma^i = 2.00 \cdot \exp(0.045\tau)\gamma^e n^{0.22} \qquad \text{(Eq. 14)}$$

The majority of the HMA layer rutting typically occurs within the top 3 to 5 inches. Hence, increasing the total thickness of a poor quality HMA mixture does not help in reducing the rut depths (NCHRP 2004).

As mentioned, much more work had been conducted for highway design. The HMA permanent deformation models presented in the Mechanistic-Empirical Pavement Design Guide (MEPDG) are based on field-calibrated statistical analysis of laboratory tests. The model is of the general form:

$$\frac{\varepsilon_p}{\varepsilon_r} = a_1 T^{a_2} N^{a_3} \qquad \text{(Eq. 15)}$$

where:

ε_p = Accumulated plastic strain at *N* repetitions of load, in/in
ε_r = Resilient strain of the asphalt material as a function of mix properties, temperature and time rate of loading, in/in.
N = Number of load repetitions.
T = Temperature, °F.
a_i = Non-linear regression coefficients.

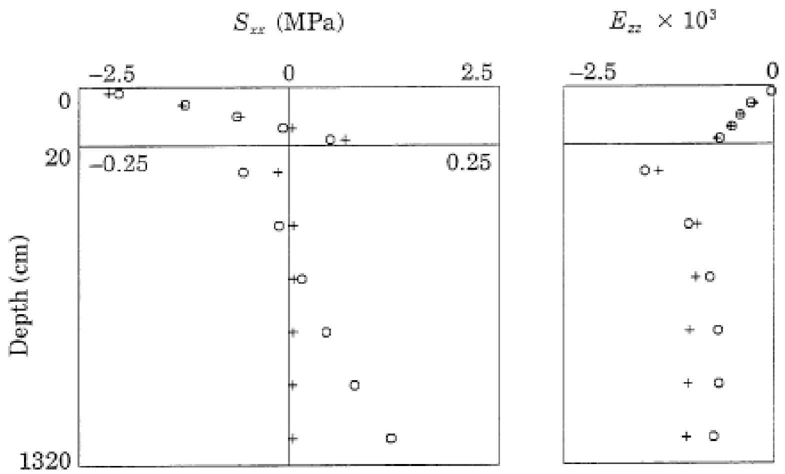
While it is not imperative that the FAA considers the HMA layer rutting in the overlay thickness design, this issue should be explicitly addressed in the mix design stage.

Base Layer Permanent Deformation

As with HMA surface layer permanent deformation, the FAA claims to handle base layer rutting through proper mix design. Again, this view is not universally shared. If any of the unbound base layers are of inferior quality, increasing the thickness of the poor-quality layer will only result in higher magnitudes of rutting.

Additionally, many layered elastic programs predict tensile stresses (and sometimes of significant magnitude) within the aggregate layers, despite the fact that unbound aggregate base is generally thought of as a cohesionless material with little tensile strength. However, ongoing airfield pavement research has observed that base aggregate does not respond as a linear, isotropic material, which layered elastic modeling assumes (Tutumluer and Thompson 1998).

Advanced modeling techniques are being developed to overcome current modeling limitations, including artificial neural networks, customized finite element programs, and others. However, these programs are not yet state-of-the-practice. These advanced programs are capable of modeling anisotropic, non-linear behavior and the no-tension boundary condition. The effects of a no-tension boundary condition is illustrated in figure 22 (Taciroglu and Hjelmstad 1998), and results in higher stresses in the HMA layer.



o linear elasticity + no-tension elasticity

Figure 22. Bending stress and vertical strain under the wheel (Taciroglu and Hjelmstad 1998).

Although base aggregate is generally a cohesionless material, there is a limited tensile strength observed in the material, possibly from the overburden pressure and the interlocking of aggregate particles during compaction or possibly due to the anisotropic nature of base aggregate (Seyhan and Tutumluer 2002). Cohesive properties of approximately 30 psi were estimated for research at the National Airport Pavement Test Facility (Kim and Tutumluer 2006). This research also indicated an angle of internal friction of 61.7-degrees for P-209 aggregate. Unbound base layer permanent deformation (rutting) prediction models were also developed from the research, but the limitation of the derived model is that it does not allow a tensile stress, which may be predicted using layered elastic analysis.

The base layer permanent deformation prediction model described in the MEPDG is based on the model proposed by Tseng and Lytton (1989). The model is of the general form:

$$\delta_a(N) = \beta_1 \left(\frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{N}\right)^{\beta}} \varepsilon_v h \qquad \text{(Eq. 16)}$$

where:

δ_a = Permanent deformation for the layer.
N = Number if traffic repetition.
$\varepsilon_0, \beta, \rho$ = Material properties.
ε_r = Resilient strain imposed in laboratory tests to obtain the above listed material properties.
ε_v = Average vertical strain in the layer as obtained from the primary response model.
h = Thickness of the layer.
β_1 = Calibration factor for the unbound granular and subgrade materials.

If the rutting is above typical design values, the engineer should improve the quality of the unbound layer in question.

Thermal Cracking

There are two kinds of thermal cracking in HMA pavements: low temperature cracking and thermal fatigue cracking. The mechanism of low-temperature cracking is shown in figure 23. As the pavement temperature decreases, the tensile stresses within the HMA increases. However, the tensile strength increases to a maximum and then decreases when cracks begin to initiate.

If the tensile stresses are lower than the tensile strength, then the pavement will not crack under a single temperature cycle, but would eventually crack after a certain number of cycles. This is the mechanism behind thermal fatigue cracking (Huang 2004).

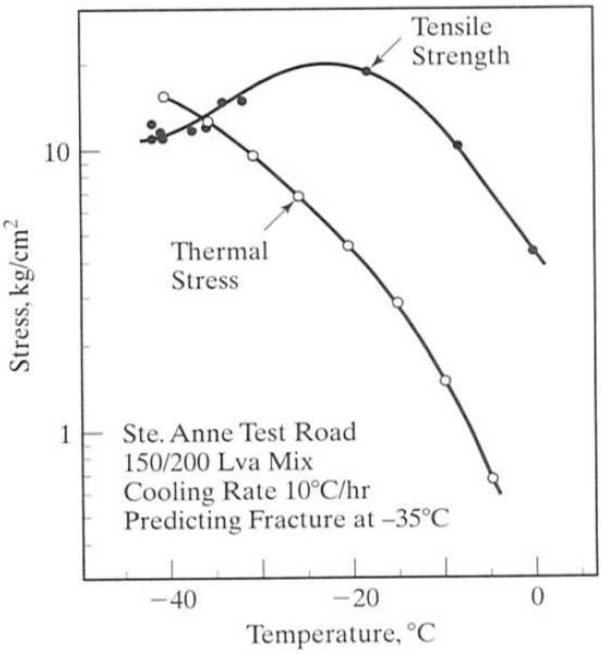


Figure 23. Mechanism of low-temperature cracking
(Huang 2004, adapted from McLeod 1970).

Several mechanistic thermal cracking models have been developed, including those by Finn et al. (1986), Ruth et al. (1982), Lytton et al. (1983), and Shahin and McCullough (1972). The latter two are the most comprehensive models that analyze both low-temperature and thermal fatigue cracking. The model developed by Lytton et al. (1983) was based on visco-elastic fracture mechanics. It predicted the amount of thermal cracking as a function of time but was dependent on estimated mixture properties rather than mixture properties directly measured at low temperatures. This motivated the SHRP A-005 researchers to develop of a new thermal cracking prediction model (TCMODEL), which predicted the amount of cracking versus time using a mechanics-based approach (Kim 2009). The salient features of the model include (Kim 2009):

- Mixture characterization that included visco-elastic behavior of the mixture.
- Thermal stress predictions that accounted for time-dependent relaxation and non-linear cooling rates.
- Stress predictions as a function of depth.

The thermal fatigue cracking model proposed by Shahin and McCullough (1972) is of the form:

$$N_f = K_1 \left(\frac{1}{\varepsilon_t} \right)^{K_2} \qquad \text{(Eq. 17)}$$

In this equation, N_f is the allowable number of cycles to cause thermal fatigue cracking under tensile strain ε_t , and K_1 and K_2 are fatigue constants obtained from strain tests. Two sets of HMA values were proposed for the fatigues constants: one each of low and very high stiffness.

Hajek and Haas (1972) proposed two empirical criteria for thermal cracking based on extensive studies with highway and airport pavements. The model developed for cracking on airport pavements is presented below:

$$TCRACK = 218 + 1.28ACTH + 2.52MTEMP + 30PVN - 60COFX \qquad \text{(Eq. 18)}$$

where:

TRACK = Transverse crack average spacing, meters.
MTEMP = Minimum temperature recorded on site, ^{o}C.
PVN = Dimensionless Pen-Vis Number (McLoed).
COFX = Coefficient of thermal contraction, mm/100 mm/^{o}C.
ACTH = Thickness of HMA mix layer, cm.

The MEPDG uses the indirect tensile test to predict the thermal cracking susceptibility. The model is an enhanced version of the TCMODEL (NCHRP 2004). The MEPDG model is of the form:

$$C_f = \beta_1 N\left(\frac{\log C / h_{ac}}{\sigma}\right) \qquad \text{(Eq. 19)}$$

where:

C_f = Observed amount of thermal cracking.
β_1 = Regression coefficient determined through field calibration.
N(z) = Standard normal distribution evaluated at (z).
σ = Standard deviation of the log of the depth of cracks in the given pavement.
C = Crack depth.
h_{ac} = Thickness of asphalt layer.

There are two stages of progression for climate-related thermal cracks: transverse cracking and block cracking. Transverse cracking occurs first, typically at a regular interval across a paving lane. As longitudinal cracks and intermediate transverse cracks occur to form a block-like pattern, it is referred to as block cracking. Transverse cracking is predicted using the model described above and block cracking is handled through material and construction variables (NCHRP 2004).

Reflective Cracking

Control of reflective cracking is an important factor to be considered in the overlay design of existing HMA pavements. It is often one of the first distresses that occur on an HMA overlay and can often be the controlling factor as to determining when an overlaid pavement needs to be rehabilitated again. None of the current design methods incorporate a reflective cracking design input for HMA overlays on HMA pavements; the Asphalt Institute does have a procedure to consider reflective cracking for HMA overlays on PCC pavements. A rule-of-thumb is that crack propagation occurs at approximately 1 inch per year through the overlay.

Several studies have been performed to investigate the prevention and mitigation of reflective cracking in HMA pavements (Sousa et al. 2005, Von Quintus et al. 2009b). AAPTP 05-04 includes decision trees for selecting the appropriate reflective cracking mitigation strategy, which depends on the type and condition of the existing pavement. The decision trees were prepared using data from previous research undertakings published in the literature. Figure 24 shows the decision tree designed to provide guidance for mitigation of reflective cracking in HMA overlays of existing HMA pavements.

The MEPDG incorporates an empirical model to predict the percentage of cracks that propagate through the overlay as a function of time using a sigmoidal function of the following form:

$$RC = \frac{100}{1 + e^{a+bt}} \qquad \text{(Eq. 20)}$$

where:

RC = Percent of cracks reflected, percent.
T = Time, years.
a, b = Fitting parameters.

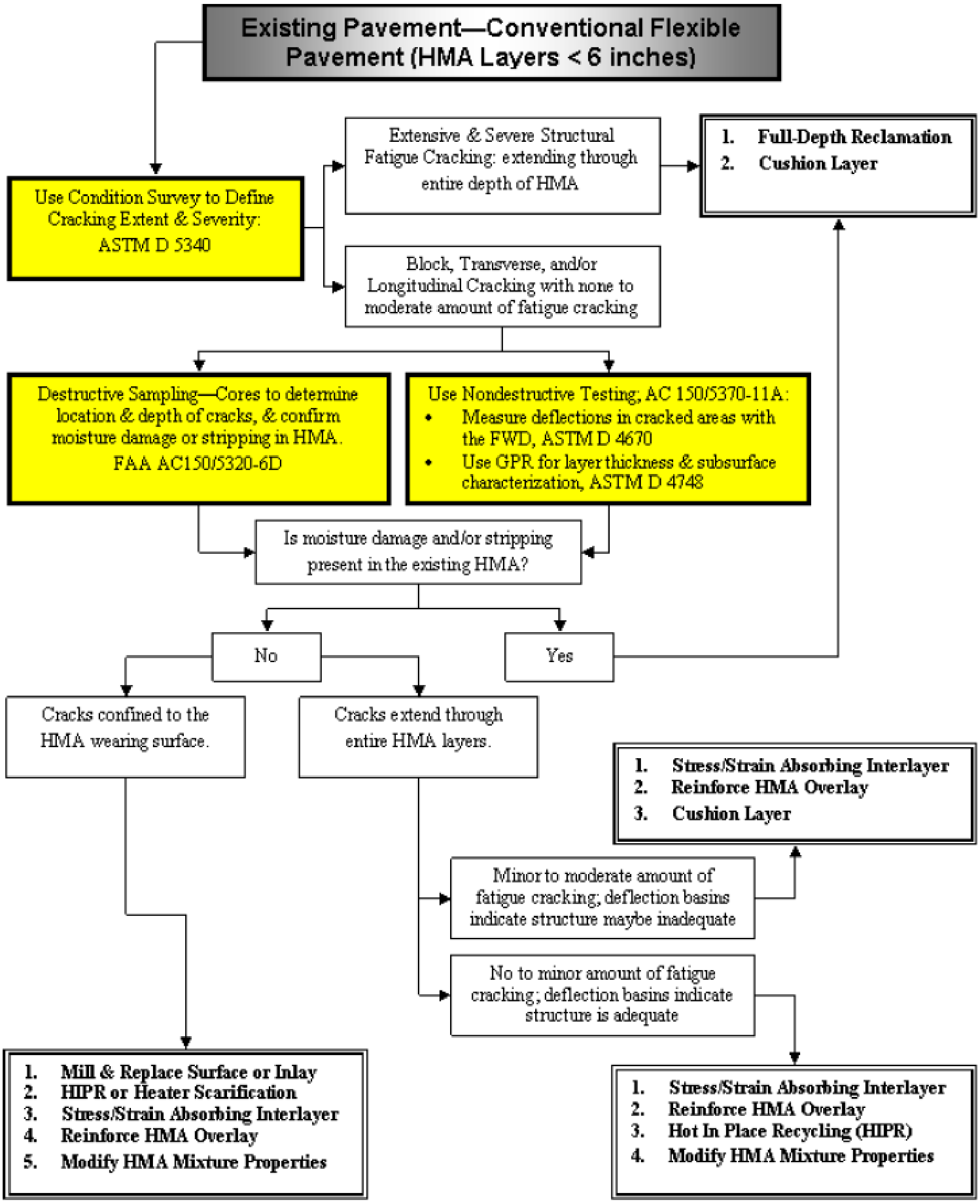


Figure 24. Decision tree for mitigation of reflective cracks in HMA overlays of existing HMA pavements (Von Quintus et al. 2009b).

The model parameters are a function of overlay thickness and type of existing pavement. This model is used to estimate the amount of cracks that have reflected from a non-surface layer to the surface after a certain period of time. A sample plot of the percent cracks reflected through the surface after a given period of time for various overlay thicknesses using the MEPDG is shown in figure 25.

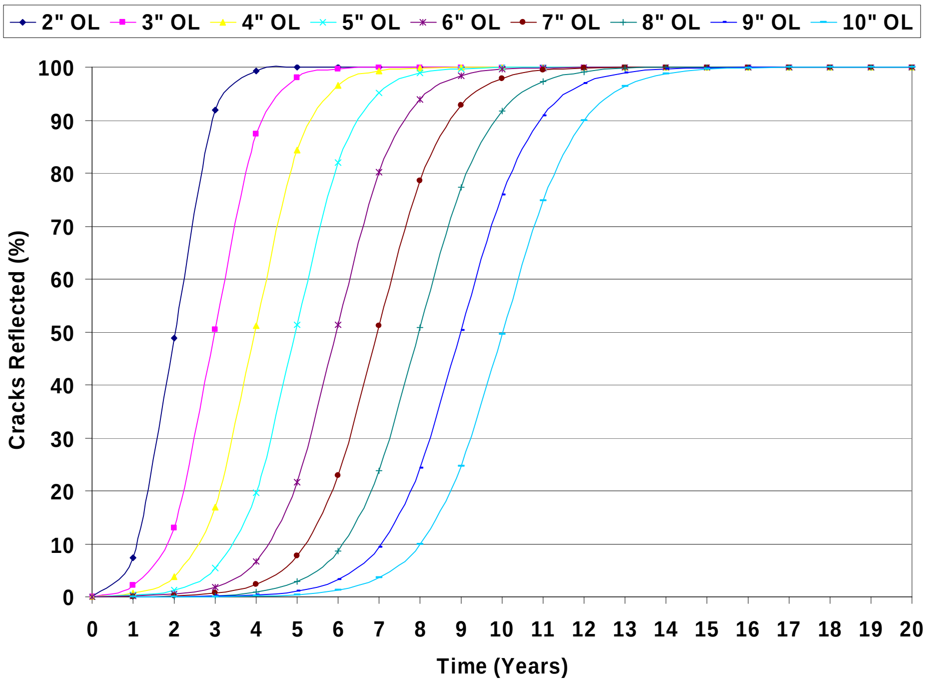


Figure 25. Impact of overlay thickness on reflective cracking.

Based on finite element modeling and verification with field sites, Sousa et al. (2005) have developed an overlay design method to resist reflective cracking in HMA pavements. This design process consists of determining several parameters from some determined/estimated parameters, followed by the determination of an allowable number of loads to failure.

The parameters that are determined/estimated are:

1. Moduli and thickness of pavement layers (in cracked state).
2. Representative air temperatures (maximum and minimum air temperature, and mean average monthly air temperature by Shell design method).

The parameters that are selected are:

1. Design cracking percentage.
2. Overlay material (modulus).

From these inputs, the following parameters are then calculated:

1. Aging adjustment factor.
2. Temperature adjustment factor.
3. Field adjustment factor.
4. Design value of Von Mises Strain.
5. Allowable traffic repetitions (ESAL) before failure.

The process is an iterative design process in which the material and/or thickness of the overlay is varied until the allowable traffic is equal to or greater than the design traffic.

The following approach can be considered for inclusion in airport pavement design:

1. Use Sousa et al's method to determine the effect of modulus and thickness of the cracked pavement layer on the design (overlay) thickness.
2. Use FAARFIELD to determine the effect of thickness/modulus of the existing pavement on the design thickness.
3. Develop adjustment factors for depth/modulus of existing pavement in FAARFIELD, to accommodate extra thickness required for preventing reflective cracking as determined in Sousa et al's method.

Figures 26 and 27 show the results of a sensitivity analysis using Sousa et al's method. The results of the sensitivity analysis with FAARFIELD are shown is figures 28 and 29.

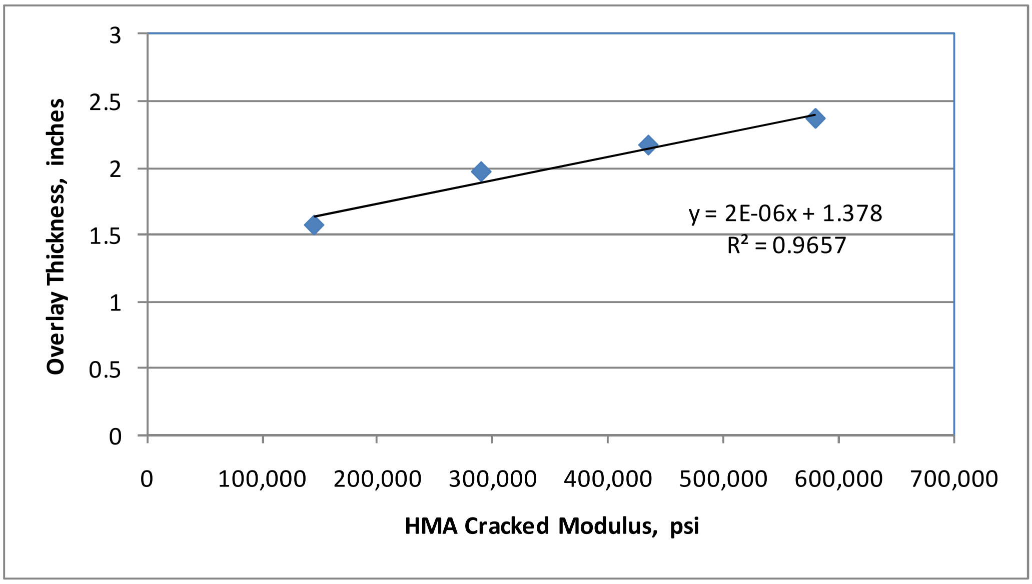


Figure 26. Plot of modulus of cracked layer versus overlay thickness.

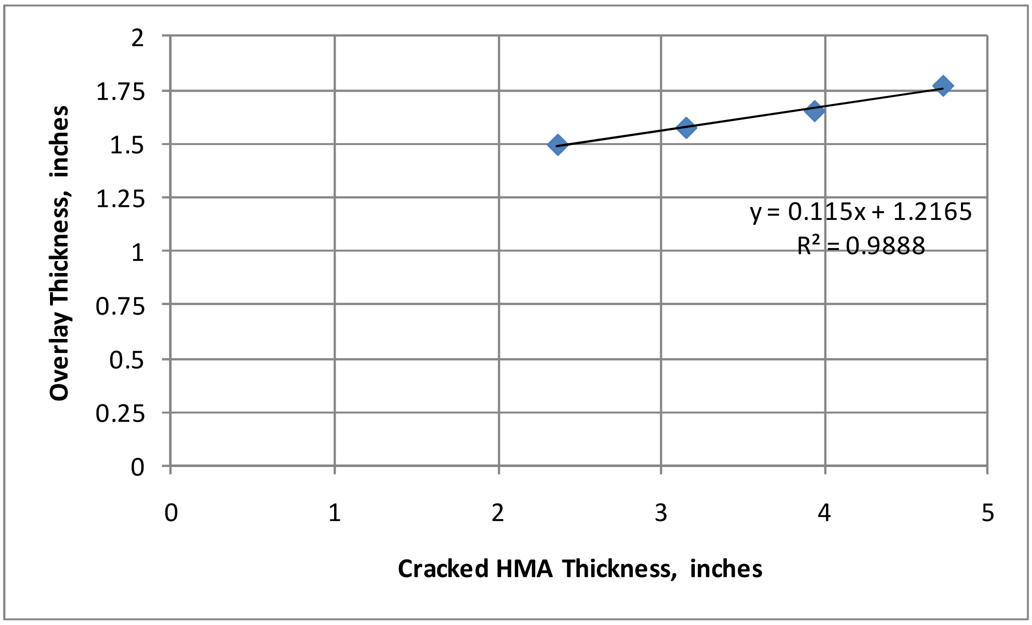


Figure 27. Plot of thickness of cracked layer versus overlay thickness.

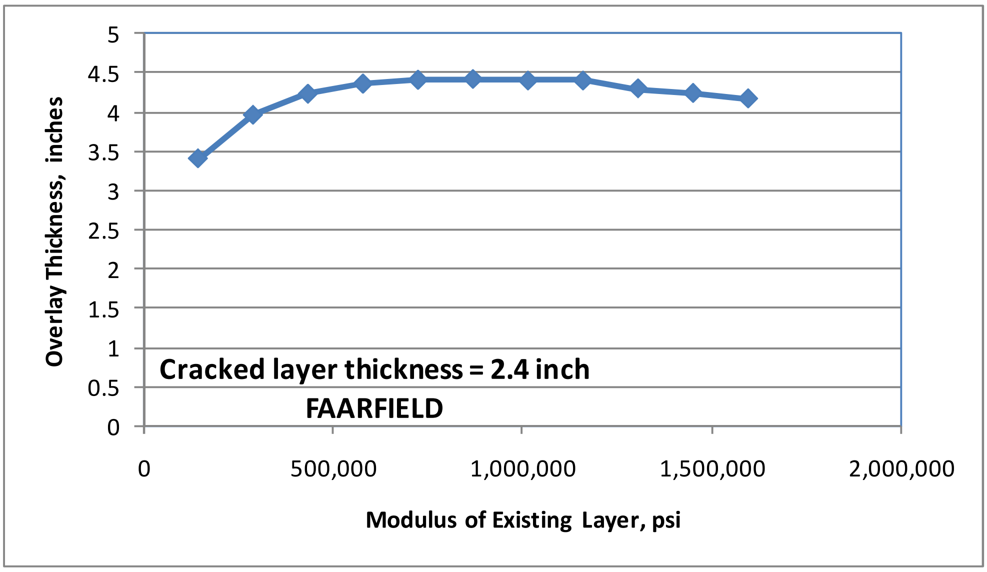


Figure 28. Plot of modulus of existing layer versus overly thickness.

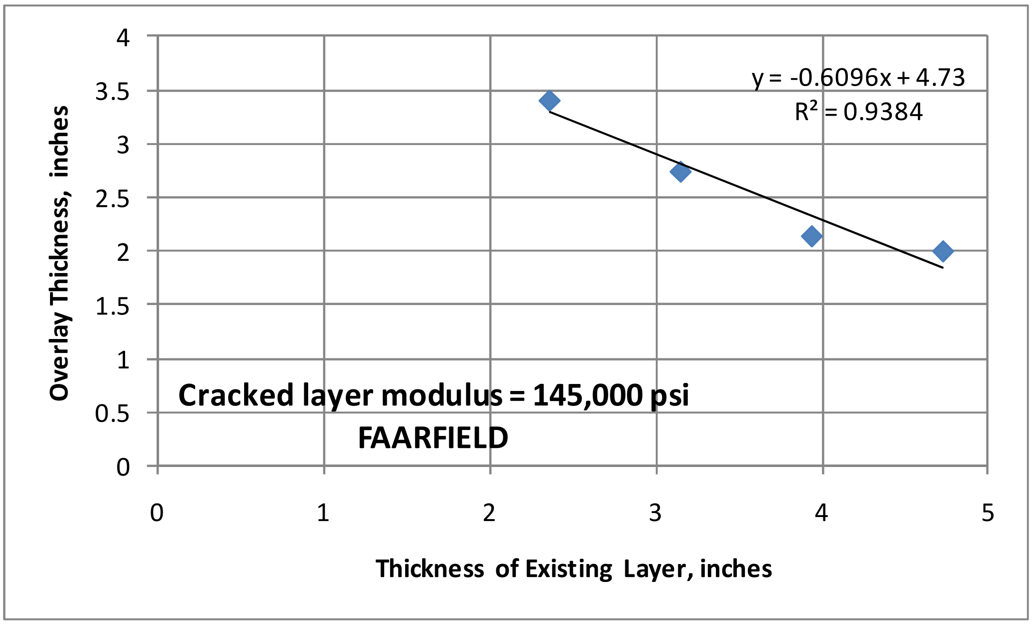


Figure 29. Plot of thickness of existing layer versus overlay thickness.

From the approach proposed by Sousa et al. (2005), it can be seen that a stiffer/thicker cracked layer is worse than a less stiff/thin cracked layer. For a cracked layer that is both thicker and stiffer, provisions should be made for a thicker overlay to reduce the chance of reflective cracking. Therefore, if the existing layer is known to be cracked, then the thickness and the modulus of the cracked layer should be "modified" first.

As presented in the sensitivity analysis plots, the lowest modulus considered is 145,000 psi, and the thinnest pavement considered is 6 inches. Therefore, one option is to consider the increase in overlay thickness by an appropriate amount in cases where the thickness of the cracked layer is greater than 6 inches and/or the modulus of the cracked layer is greater than 145,000 psi.

Figure 26 (from Sousa et al. 2005) shows that for every 145,000 psi increase in modulus, there is a need for an extra ½ inch of overlay. Meanwhile, figure 29 (sensitivity of FAARFIELD) shows that an additional ½ inch of overlay can be obtained by considering a 2-inch reduction in the thickness of the existing (cracked) layer.

Similarly, figure 27 shows that for a 100 percent increase in the cracked layer thickness, there is an increase in the design overlay thickness of ¼ inch, while figure 29 shows that reducing the cracked layer thickness by 1 inch would yield an additional overlay thickness of ¼ inch.

Hence, the steps can be summarized as follows:

- Determine the thickness ($t_{measured}$) and modulus of the existing cracked layer (from evaluation and modulus at standard conditions).

- For every 145,000 psi above a modulus of 145,000 psi (referring to the existing layer modulus), reduce the thickness of the existing cracked layer by 2 inches ($t_1 = t_{measured} - 2$). Use proportional values for other moduli.
- For every 6 inches of thickness above 6 inches (referring to the existing layer thickness), reduce the thickness of the existing cracked layer determined in step 2 by 1 inch ($t_{input} = t_1 - 1$). Use proportional values for other thicknesses.
- Use the thickness determined in step 3 as the input thickness of the existing cracked layer in FAARFIELD.

Note that these conclusions are based on certain values of the relevant parameters (in Sousa et al's method), as presented in table 7. Separate simulations can be carried out for different values of the following parameters to develop "thickness reduction factors" as previously discussed.

Top-Down Cracking

Traditionally, fatigue cracking has been assumed to be bottom-up cracking due to the flexural stresses and strains that develop at the bottom of the bound HMA layers. However, recent studies have demonstrated that fatigue cracking can also initiate at the top and propagate downwards (top-down cracking). The MEPDG provides models that describe both bottom-up and top-down fatigue cracking (NCHRP 2004). The general form of the model is similar to the fatigue cracking prediction model shown in equation 11, with the only difference being the regression parameter that provides a correction for different asphalt layer thicknesses.

Table 7. Values of relevant parameters used in Sousa et al's method in the example.

Parameter	Input
% Cracking at End of Design Life (min. >1 max 10)	10
Maximum Air Temperature (°C)	40
Mean Monthly Air Temp (°C) (Shell Design)	18
Minimum Air Temperature (°C)	-5
HMA Cracked Moduli (MPa) (@Tmm)	1000
HMA Cracked Thickness (cm)	12
Granular Layer Moduli (MPa)	100
Granular Layer Thickness (cm)	30
Subgrade Moduli (MPa)	100
Traffic	86-kN ESAL

While new and improved design philosophies are available to include top-down cracking in the pavement design phase, the most effective approach is to handle this in the asphalt mix design phase (Emery 2006).

Delamination

Delamination or debonding describes a condition where the HMA lifts or layers lose adhesion between each other and can become separated. Typical design and construction practices include a certain level of bonding between the layers. However, the appropriate amount and testing techniques are still under debate.

A study carried out by the Washington State Department of Transportation aimed at identifying potential debonding mechanisms, determining debonding impacts on pavement performance, and identifying the role of tack coat in debonding. The preliminary study concluded that debonding is most likely caused by poor tack coat between layers or water infiltration due to distress or inadequate compaction (Muench and Moomaw 2008). Several recommendations that could help to prevent debonding are presented in the preliminary report and include the following:

- Monitoring progress on tack coat studies.
- Use of undiluted tack coat.
- Use of proper emulsions.
- Use of tack coat between all HMA layers.
- Develop/adopt a test tack coat uniformity and application rate.
- Investigate new methods to reduce or eliminate tack tracking.
- Develop specification to remove thin debonded layers after milling.

Based on the literature review, it appears the issue of delamination of HMA overlays can be handled through proper mix design and construction specifications and practices.

SUMMARY

This chapter summarizes the more common overlay design procedures and performance models for airfield pavements obtained by conducting a literature review. In particular, this chapter includes a discussion of the available HMA overlay design procedures used by the FAA, U.S. Army Corps of Engineers, and the Asphalt Institute, with an emphasis on identifying the inputs used for each procedure, the failure mechanisms analyzed, and the strengths and weaknesses of each procedure. Additionally, behavior of performance models and typical failure mechanisms of HMA pavements and overlays are reviewed. Suggestions are also made where their possible incorporation into the FAA design procedure and FAARFIELD may be appropriate.

Different design approaches characterize the effect of an existing pavement's condition on the design of an HMA overlay. Potential impacts include reflection cracking, rutting, fatigue cracking, thermal cracking, and delamination. The approach that least considers the potential effect of these factors on HMA overlay design is that incorporated in the FAA's FAARFIELD program. An examination of other procedures and a consideration of overlay performance highlights the need to extend the capabilities of FAARFIELD to better address factors in design that are known to affect HMA overlay performance.

Based on a review of the available design procedures and performance models, the following items should be considered for future improvements to the FAA's HMA overlay design procedure:

- In FAARFIELD, all layers are assumed to be bonded. To adequately model some existing pavement structures, FAARFIELD should allow unbonded layers to be modeled. Although layers will not (or at least should not) be constructed as unbonded, there are many cases where the layers become unbonded over time. This feature will allow the user to investigate the effect of the existing unbonded layer and will help determine the need for and depth of milling.
- The ability to model variable inputs due to changes in moisture conditions, temperature fluctuations, and seasonal changes should be considered in future versions of FAARFIELD.
- Other failure modes should be considered in future versions of FAARFIELD. In particular, rutting/deformation in layers other than the subgrade and reflective cracking are two common distresses mechanisms that deserve consideration in HMA overlay design.

These recommendations are further discussed in Chapter 6.

CHAPTER 3. FAARFIELD HMA OVERLAY DESIGN SENSITIVITY ANALYSIS

INTRODUCTION

The design (and therefore the implied performance) of HMA overlays is dependent on the inputs used in the design analysis. Some inputs have little effect on the required thickness while others have a significant influence. Sensitivity analyses provide useful information regarding the influence of each input value on the design procedure. Sensitivity analyses of the FAARFIELD HMA overlay design procedure have been performed to analyze the influence of input parameters used to characterize the existing pavement structure as well as other factors important to the design (and performance) of HMA overlays. The results can be useful in determining where more care is needed in evaluating or characterizing inputs to the overlay design process. More specifically for this project, it can be used to assess the impact of changing inputs such as layer thickness and layer modulus on the required overlay thickness.

The design of HMA overlays in FAARFIELD is divided into HMA overlay of flexible pavement and HMA overlay of rigid pavement. An HMA overlay of rubblized PCC is a special case of an HMA overlay of a flexible pavement. The design of HMA overlays of existing HMA/PCC pavements (referred to as composite pavements from here forward) generally follows the overlay of rigid pavement approach; however, the pavement response (and therefore design) of composite pavements is dependent on the thickness of the HMA overlay in comparison to the thickness of the underlying PCC pavement.

The typical airport flexible pavement consists of an HMA surface, stabilized base (or aggregate base), and aggregate subbase on subgrade. When a pavement reaches a condition level that warrants rehabilitation, a detailed evaluation is often performed to determine the existing pavement layer properties and to assess their load-carrying capability. While the FAA procedure does provide guidance on evaluating in-place pavements, there is little guidance on incorporating the evaluation results into the rehabilitation design. Additionally, there are “default” settings in FAARFIELD for these layers that are fixed and can not be modified, including the layer moduli. Essentially only the layer thicknesses can be modified (when selecting standard P-XXX specification materials), along with the subgrade modulus. These default values are more than likely not what would be determined from a detailed evaluation. However, “undefined” and “variable” layer types within FAARFIELD allow the user to adjust the modulus of the pavement layers. Poisson’s ratio and the bonding condition are fixed and cannot be adjusted, regardless of the selected layer type.

The design of an HMA overlay of a flexible pavement is carried out in the same manner as new flexible pavement design: the required thickness is determined to meet subgrade compressive strain requirements (to limit subgrade rutting). The program will check for fatigue in the HMA layers (FAARFIELD version 1.302 analyzes the tensile strain at the bottom of the HMA surface layer; FAARFIELD version 1.305, released during this project, will analyze horizontal strain and HMA cumulative damage at the bottom of all P-401/P-403 layers), but it is not the default performance criterion. Strains are calculated within FAARFIELD using the layer elastic program LEAF. The procedure does not currently directly use the condition of the existing flexible pavement as part of the model, although FAARFIELD does use the Structural Condition Index (SCI) and Cumulative Damage Factor Used (CDFU) as part of the rigid pavement overlay procedure. Stresses for composite (or rigid) pavement overlay design are determined using finite element analysis.

The design of an HMA overlay of rigid pavements is based on the analysis of the PCC slab and the continued deterioration of the underlying slab. The required pavement thickness is determined by analyzing the tensile stress at the bottom of the slab for structural cracking, with stresses determined using three-dimensional finite element modeling. The deterioration of the PCC slab is included in the overlay design procedure based on the reduction of the SCI in discrete increments over the analysis period. The overlay thickness requirement is determined by summing the incremental damage over the design period. If SCI is 100 (no structural distress) at the beginning of the analysis, CDFU is also considered. Assessing the appropriate SCI value becomes difficult for existing composite pavements, as the PCC is not visible for inspection.

Although LEDFAA estimates 1-inch per year for crack propagation, FAARFIELD does not account for reflective crack propagation. It also does not specifically address materials-related performance, such as rutting within the HMA overlay and thermal cracking. These performance issues are intended to be addressed through the construction material specifications.

HMA OVERLAY DESIGN PARAMETERS

Sensitivity analyses of the FAARFIELD design procedure presented herein focuses on the appropriate adjustments to the design inputs to reflect the existing pavement conditions and any pre-overlay treatments. Understanding how each of the available design inputs influence the results helps determine which inputs to focus on for characterizing the existing pavement and which inputs can be adjusted to account for anticipated performance differences in the HMA overlay based on pre-overlay repair efforts, materials, and so on. It also helps determine if additional design procedure recommendations are applicable. The common flexible pavement design inputs in FAARFIELD considered for this sensitivity analysis include the following:

- Traffic data, including aircraft weights and volumes.
- Subgrade modulus.
- Aggregate subbase modulus.
- Existing HMA surface and base modulus.
- New HMA overlay modulus.

The following additional inputs for composite pavements are also analyzed:

- Existing PCC modulus/flexural strength.
- SCI.
- CDFU.

These inputs are discussed in detail in the following sections. Although climate can play a significant role in pavement performance, it is only indirectly addressed through the variation in modulus values. For example, the HMA modulus would be higher in a colder climate and lower in a warmer climate. The rubblized PCC layer is a special case of the P-209 layer type, so a separate analysis is not conducted.

Traffic Inputs

FAARFIELD, and previously LEDFAA, incorporates the use of the entire traffic mix. The design procedure no longer determines a “design aircraft” as the one aircraft representing the entire aircraft fleet, as in previous versions of the FAA pavement design procedure. The damage to the pavement from each aircraft is analyzed and includes consideration of gear placement (offset from centerline) and wander.

Three traffic data sets were selected for use in these sensitivity analyses and are representative of a commercial (or primary) airport, reliever (or secondary) airport, and general aviation (or executive) airport. The aircraft models, weights, and volumes for the three mixes are summarized in tables 8, 9, and 10, respectively.

Table 8. Summary of commercial airport traffic mix used in sensitivity analysis.

Aircraft Model	Gear Configuration	Weight, lbs	Average Annual Departures
A300-B4 std	2D	365,747	900
A319-100 opt	D	150,796	6,023
A320-200 Twin std	D	162,922	9,379
A321-100 std	D	183,866	2,713
A340-300 std	2D/D1	608,245	359
A380-800	2D/3D2	1,239,000	266
B717-200 HGW	D	122,000	3,038
Adv. B727-200 Option	D	210,000	265
B737-500	D	134,000	4,615
B737-700	D	155,000	32,823
B747-400B Combi	2D/2D2	877,000	1,510
B757-200	2D	256,000	20,312
B767-300 ER	2D	413,000	7,997
B777-300 ER	3D	777,000	1,664
DC10-30/40	2D/D1	583,000	1
DC9-51	D	122,000	9
L-1011	2D	498,000	4
MD11ER	2D/D1	633,000	609
MD83	D	161,000	361
RegionalJet-700	D	72,500	8,928
RegionalJet-200	D	47,450	2,193
Dual Whl-30	D	35,600	4,778
Dual Whl-20	D	16,300	800
Single Wheel-10	S	6,536	371
Single Wheel-15	S	14,550	1,470
Single Wheel-12.5	S	12,000	1,417
Single Wheel-5	S	5,100	1,932
Single Wheel-5	S	3,800	3,453

Table 9. Summary of reliever airport traffic mix used in sensitivity analysis.

Aircraft Model	Gear Configuration	Weight, lbs	Average Annual Departures
A300-600 std	2D	380,500	365
A319-100 opt	D	150,796	731
A320-200 Twin std	D	162,922	2,192
Adv. B727-200 Option	D	210,000	365
Adv. B737-200 QC	D	128,600	365
B737-300	D	140,000	731
B737-400	D	150,500	1,096
B757-200	2D	256,000	1,096
DC9-32	D	109,000	1,462
DC9-51	D	122,000	1,096
MD83	D	161,000	1,096
Challenger-CL-604	D	48,200	5,481
Dual Wheel-45	D	45,415	7,673
Dual Wheel-20	D	17,120	244
Single Wheel-12.5	S	12,500	365
Dual Wheel-45	D	43,000	1,096
Hawker-800	D	27,520	487
Citation-VI/VII	S	23,200	1,218
Gulfstream-G-V	D	90,900	3,248
Dual Wheel-20	D	24,500	812
Learjet-35A/65A	D	18,000	2,842
Single Wheel-15	S	14,600	1,624
Baron-E-55	S	5,424	122
Skyhawk-172	S	2,350	122
Conquest-441	S	9,850	122
Skyhawk-172	S	2,558	244
Bonanza-F-33A	S	3,412	284
Navajo-C	S	6,536	81

The distribution of aircraft weights for the traffic mixes is summarized in figure 30. The commercial traffic mix primarily consists of aircraft greater than 100,000 lbs, while the general aviation traffic mix generally includes aircraft less than 30,000 lbs. The reliever traffic mix has mostly aircraft between 30,000 and 100,000 lbs, but also has some lighter and heavier aircraft. As one would expect, the aircraft gear types vary with the traffic mixes, as illustrated in figure 31: the general aviation traffic primarily having single wheel gear, the reliever traffic having primarily dual wheel gear, and the commercial having larger gear types in the mix. The sensitivity analyses performed on the various inputs are repeated for each traffic scenario to investigate whether the same trends hold with different traffic levels.

Table 10. Summary of general aviation airport traffic mix used in the sensitivity analysis.

Aircraft Model	Gear Configuration	Weight, lbs	Average Annual Departures
Single Wheel-5	S	5,500	6,800
Single Wheel-3	S	3,650	16,150
Single Wheel-20	S	22,000	170
Skyhawk	S	2,558	34,000
Stationair	S	3,612	14,450
Conquest	S	9,925	3,400
Single Wheel-15	S	14,800	935
Challenger	D	48,200	1,870
Single Wheel-12.5	S	12,500	2,550
Dual Wheel-30	D	28,660	1,020
Single Wheel-20	S	23,500	255
Learjet	D	18,000	1,360
Single Wheel-15	S	14,630	255

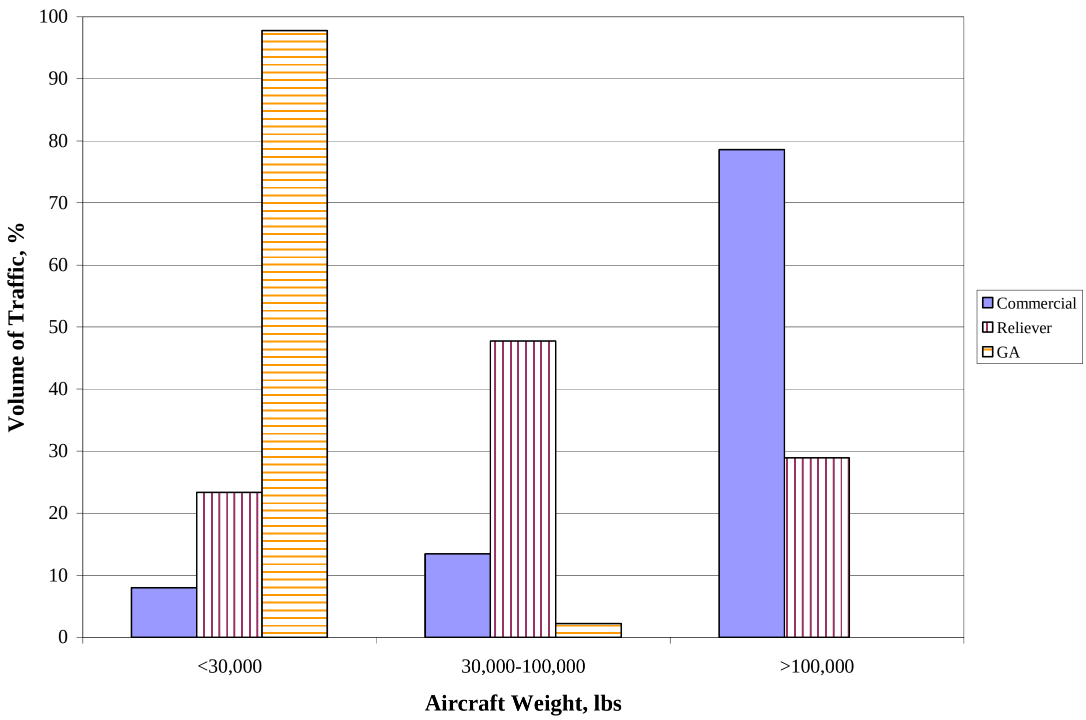


Figure 30. Aircraft weight distribution in traffic mixes.

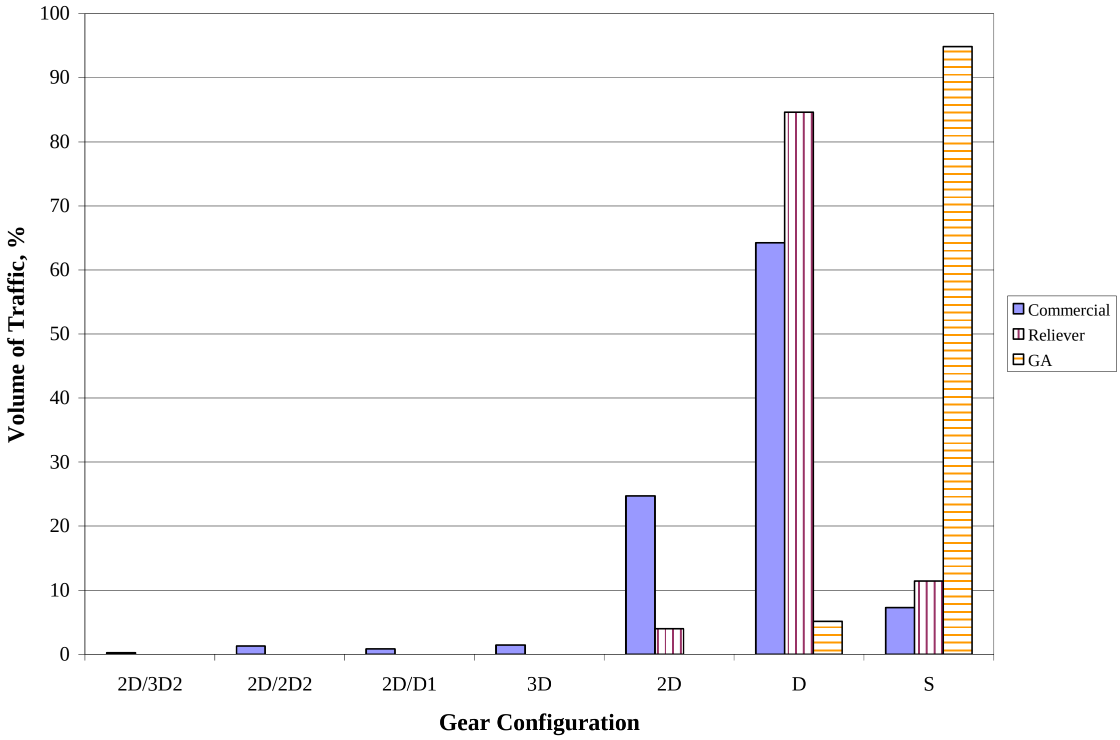


Figure 31. Gear configuration distribution in traffic mixes.

Pavement Layer Inputs

The sensitivity analyses were conducted using flexible pavement cross sections associated with the traffic mix data, as summarized in table 11.

Table 11. Summary of flexible pavement cross sections.

Layer Property	Traffic Mix		
	Commercial	Reliever	General Aviation
HMA Surface, in	4.0	2.0[1]	4.0
HMA Base, in	4.0	4.0	-
Aggregate Subbase (P-209), in	12.0	-	9.0
Aggregate Subbase (P-154), in	-	20.0	-
Subgrade Modulus, psi	17,200	12,000	5,400

[1] Modeled as “undefined” layer because of thickness.

The FAARFIELD design program has several default pavement layers available (see table 12). The default layers within FAARFIELD were used to determine the “baseline” HMA overlay requirements for each traffic mix and corresponding cross section. In order to evaluate the change in modulus for a layer, the “undefined” layer was used. This does not affect the results because only the modulus (E) of the layer is used in the calculation, regardless of the layer name.

Additionally, for evaluating the influence of the existing HMA modulus, the asphalt-stabilized surface and base courses were combined to form a "composite" modulus value (in pavement evaluation and backcalculation, these layers are often combined into a single layer). The initial composite value was selected to result in the same overlay thickness requirement as the baseline model.

Table 12. Summary of default pavement layer properties.

Pavement Layer	Default Modulus, psi	Poisson's Ratio
P-401 HMA Overlay	200,000	0.35
P-401 HMA Surface	200,000	0.35
P-501 PCC Surface	4,000,000	0.15
P-401/P-403 Base Course	400,000	0.35
P-209 Aggregate Base/Subbase	75,000[1]	0.35
P-154 Aggregate Subbase	40,000[1]	0.35
"Undefined"	1,000 – 4,000,000	0.35

[1] Value used in design is a function of thickness and underlying layer modulus using the "Modulus" procedure.

The composite pavement analyses were conducted using the commercial traffic mix and the cross section summarized in table 13.

Table 13. Summary of composite pavement cross section.

Layer Property	Design Value
Existing HMA Overlay, in	3.0[1]
PCC Pavement, in	17.0
PCC Flexural Strength, psi	600
HMA Base, in	6.0
Aggregate Subbase (P-209), in	6.0
Subgrade Modulus, psi	17,200

[1] Existing HMA overlay is not modeled in FAARFIELD but is subtracted from the calculated HMA overlay thickness for bare PCC.

Additional considerations are made for the design of HMA overlays of composite pavements. As mentioned previously, the design is generally carried out using the rigid pavement overlay design approach, which considers the SCI and Cumulative Damage Factor Used (CDFU). The SCI is typically the structural-related distresses in the PCI procedure and is used in FAARFIELD to represent the structural integrity of the existing PCC pavement. However, the SCI can also be related to the C_b factor found in previous versions of the FAA pavement thickness design procedure, with the following correlation (FAA 2004):

$$SCI = 100 * C_b - 25 \qquad \text{(Eq. 21)}$$

where:

SCI = Structural Condition Index.
C_b = Structural integrity condition factor of existing PCC pavement (generally 1.0 to 0.75).

In broad terms, SCI is 100 when there is no structural cracking, and 67 is the lowest allowable input value. An SCI of 80 is consistent with structural failure of the PCC pavement. When SCI is 100, the CDFU is used to represent the damage that has been done before initiation of the first crack. During the design procedure, the structural condition of the PCC pavement is analyzed in increments to determine structural deficiency. For this analysis, the baseline SCI is assumed to be 83 and the CDFU is 100.

SENSITIVITY ANALYSIS MATRIX

The sensitivity analyses performed using FAARFIELD are summarized in table 14.

Table 14. Summary of flexible pavement design sensitivity inputs.

FAARFIELD Input	Flexible Design	Composite Design	Flexible Remaining Life
Aircraft Weight	✓		
Aircraft Volume	✓		
HMA Overlay Modulus	✓		✓
HMA Overlay Thickness			✓
Existing HMA Modulus	✓		
Existing HMA Thickness (constant modulus)	✓		
PCC flexural Strength		✓	
PCC SCI		✓	
PCC CDFU		✓	
Aggregate Subbase Modulus	✓		
Subgrade Modulus	✓		

The sensitivity analysis results for flexible pavement design are based on the rutting cumulative damage criteria because it controls the design results. Although there is an option to evaluate HMA fatigue, the program available at the time of this analysis only evaluates the tensile strain at the bottom of the HMA surface layer (even if other stabilized layers are located below it), and the layers are assumed to be bonded. As such, it is rare to find a case in which HMA fatigue damage controls the design. In these analyses, the HMA cumulative fatigue damage was found to be near zero even for the highest aircraft weight cases (note: additional runs using version 1.305 of FAARFIELD, which analyzes fatigue damage for all HMA layers, were performed and are presented later in this chapter).

SENSITIVITY ANALYSIS RESULTS

The sensitivity of the input variable in relation to the thickness (or remaining life) is estimated using the following general form:

$$S_{x,T} = \frac{\partial T}{\partial x} * \frac{x}{T} \qquad \text{(Eq. 22)}$$

This approach essentially determines the slope of the change in result for the incremental change in input. Figure 32 illustrates examples of high (input 1) and low (input 2) sensitivity inputs. There is very little change in the result for input variable 2, while there is a much higher change in result for input 1. The resulting average sensitivity values are 3.0 and 0.2 for inputs 1 and 2, respectively.

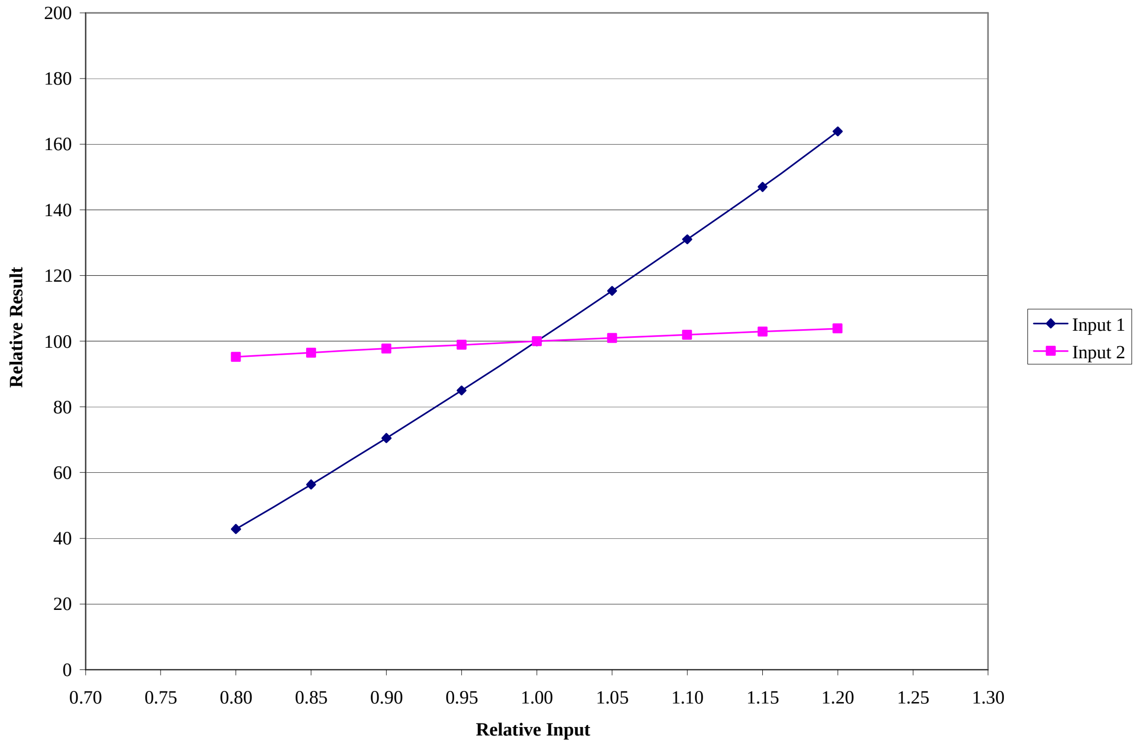


Figure 32. Example of high and low sensitivity inputs.

HMA Overlay of Flexible Pavement

A 5, 10, 15, and 20 percent change (increase and decrease) in the design input is generally used in this analysis, although a larger change is used for evaluating the HMA modulus, including moduli up to approximately 500,000 psi (higher HMA moduli are assessed later in this section). The FAARFIELD design results are summarized in table 15.

Table 15. Summary of FAARFIELD sensitivity runs.

Input Change	Required HMA Overlay Thickness, in		
	Commercial	Reliever	General Aviation
Aircraft Weight			
-20%	3.06	2.00	4.08
-15%	4.03	2.32	4.56
-10%	5.04	3.22	5.03
-5%	6.08	4.11	5.49
Baseline	7.15	5.02	5.93
+5%	8.25	5.93	6.36
+10%	9.37	6.88	6.77
+15%	10.51	7.82	7.18
+20 %	11.72	8.78	7.58
Aircraft Volume			
-20%	6.81	4.68	5.79
-15%	6.90	4.78	5.83
-10%	6.99	4.87	5.86
-5%	7.07	4.95	5.90
Baseline	7.15	5.02	5.93
+5%	7.22	5.08	5.96
+10%	7.29	5.14	5.99
+15%	7.36	5.20	6.01
+20 %	7.43	5.25	6.04
HMA Overlay Modulus			
-20%	7.75	5.52	6.40
-15%	7.58	5.38	6.26
-10%	7.43	5.25	6.14
-5%	7.28	5.13	6.03
Baseline	7.15	5.01	5.93
+5%	7.03	4.91	5.83
+10%	6.91	4.81	5.74
+15%	6.81	4.72	5.66
+20 %	6.70	4.63	5.58
+50%	6.21	4.21	5.20
+75%	5.89	3.93	4.96
Existing HMA Modulus			
-20%	7.70	5.25	6.23
-15%	7.55	5.19	6.15
-10%	7.41	5.13	6.07
-5%	7.29	5.07	6.00
Baseline	7.16	5.01	5.93
+5%	7.05	4.96	5.86
+10%	6.93	4.91	5.80
+15%	6.82	4.86	5.74
+20 %	6.72	4.81	5.69
+25%	6.62	4.76	5.64
+45%	6.23	4.58	5.45
+75%	5.73	4.34	5.25
+85%	5.57	4.26	5.19

Table 15. Summary of FAARFIELD sensitivity runs (continued).

Input Change	Required HMA Overlay Thickness, in		
	Commercial	Reliever	General Aviation
Existing HMA Thickness			
-20%	8.93	6.30	6.73
-15%	8.49	5.99	6.53
-10%	8.04	5.67	6.33
-5%	7.61	5.34	6.13
Baseline	7.16	5.01	5.93
+5%	6.72	4.69	5.73
+10%	6.28	4.35	5.53
+15%	5.84	4.02	5.33
+20 %	5.40	3.68	5.13
Aggregate Subbase Modulus			
-20%	7.26	5.42	6.07
-15%	7.24	5.33	6.04
-10%	7.21	5.23	6.01
-5%	7.18	5.13	5.97
Baseline	7.13	5.01	5.93
+5%	7.08	4.89	5.88
+10%	7.03	4.77	5.84
+15%	6.97	4.65	5.79
+20 %	6.90	4.52	5.74
Subgrade Modulus			
-20%	11.36	8.44	7.05
-15%	10.05	7.46	6.75
-10%	8.96	6.56	6.47
-5%	8.01	5.73	6.19
Baseline	7.15	5.02	5.93
+5%	6.39	4.36	5.67
+10%	5.71	3.77	5.43
+15%	5.11	3.23	5.19
+20 %	4.56	2.72	4.96

The resulting average sensitivities are summarized in figure 33. As illustrated in figure 33, the most significant influence of the included design inputs is aircraft weight, as is generally known. Besides aircraft weight, the design procedure is most sensitive to the subgrade modulus input. The existing pavement layers (bound and unbound) generally do not have as significant an influence as these other inputs; however, the thickness of the existing HMA layer does have an influence of approximately 1:1 overlay on thickness.

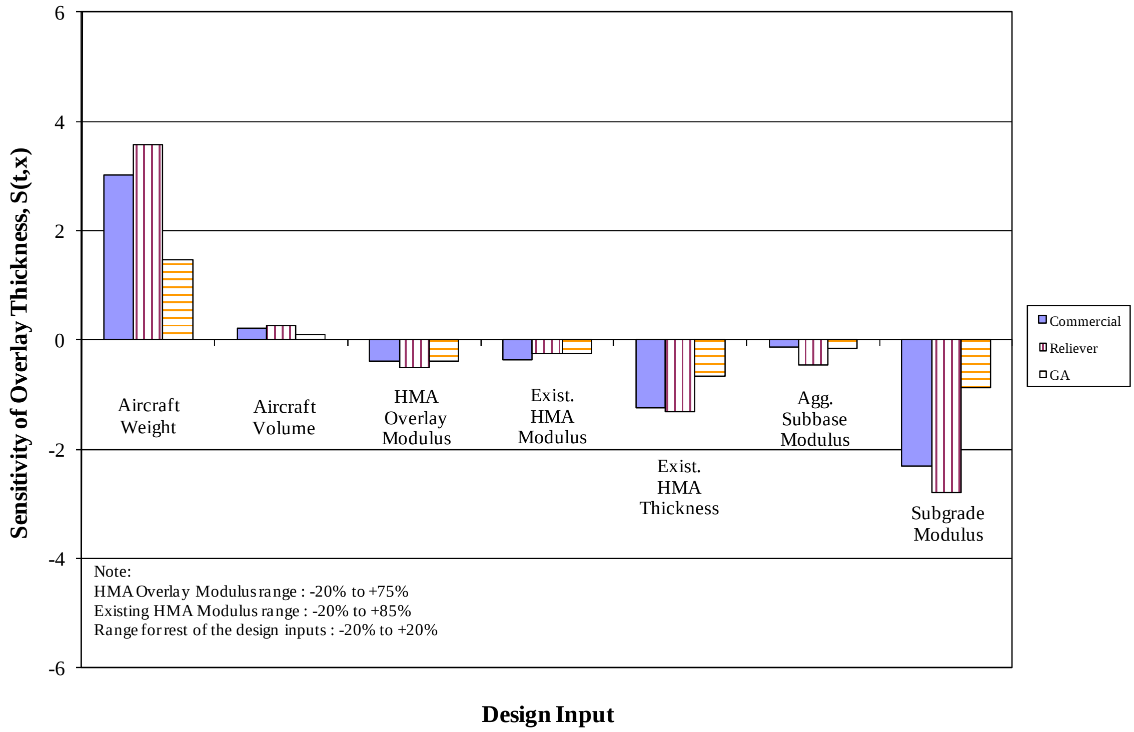


Figure 33. Summary of average thickness design sensitivity to flexible pavement inputs.

The magnitude of the sensitivity varies by traffic mix, but the sensitivity by percent change (5, 10, 15, and 20 percent) is relatively consistent for each input for the three traffic mixes, as illustrated in figures 34 through 40. The design thickness for the general aviation traffic mix is generally less sensitive to the change in design inputs than the other two traffic mixes.

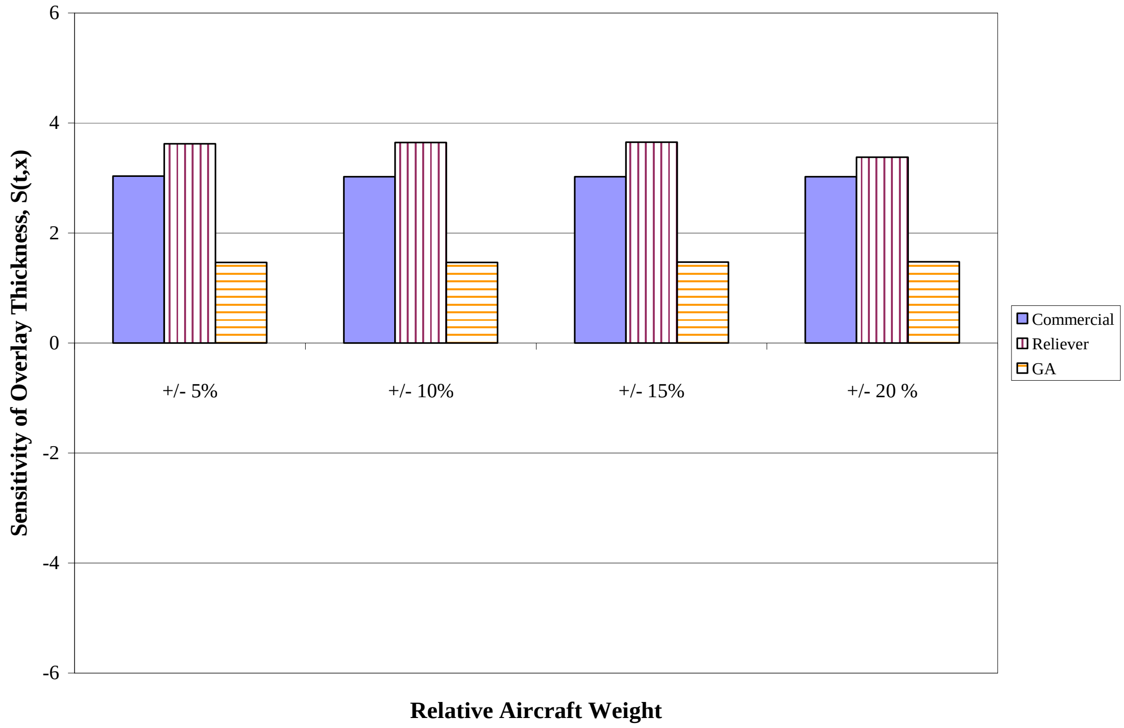


Figure 34. Summary of design sensitivity to aircraft weight.

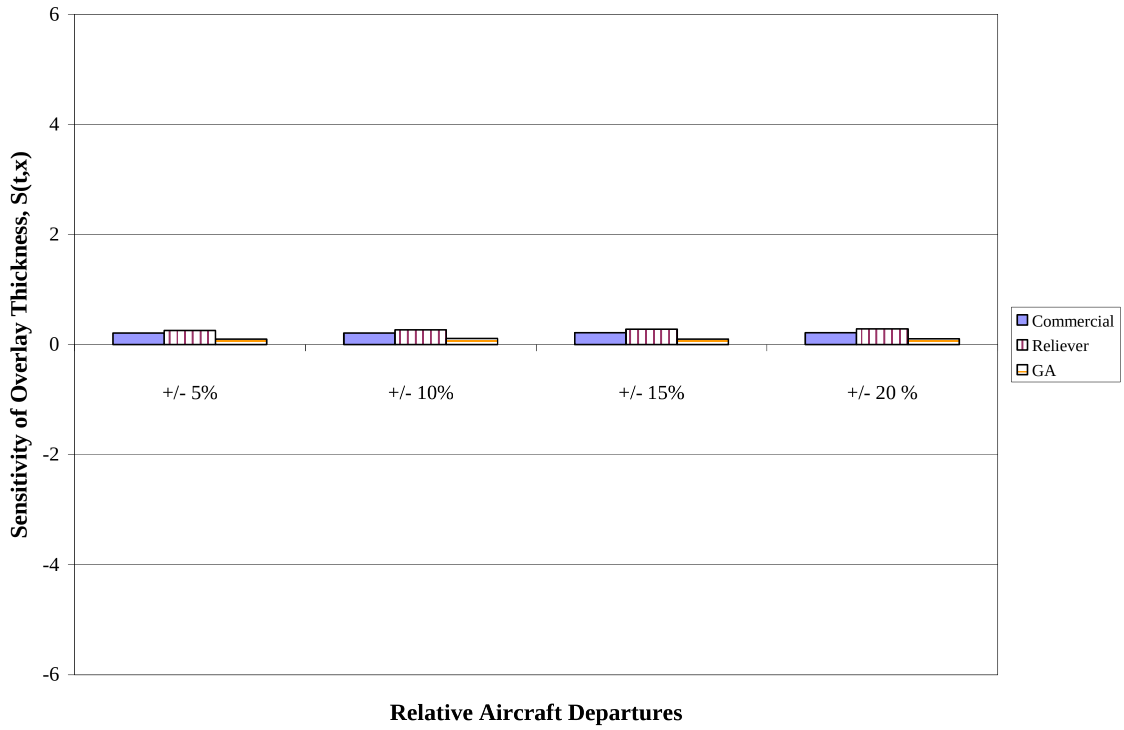


Figure 35. Summary of design sensitivity to aircraft departures.

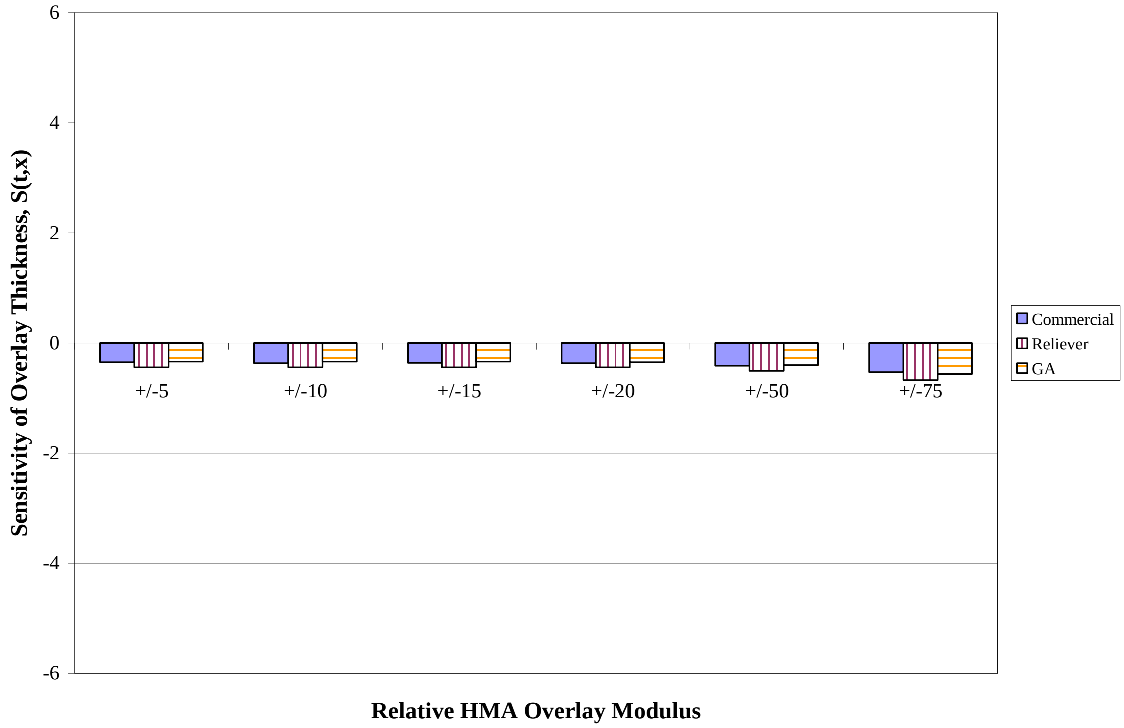


Figure 36. Summary of design sensitivity to HMA overlay modulus.

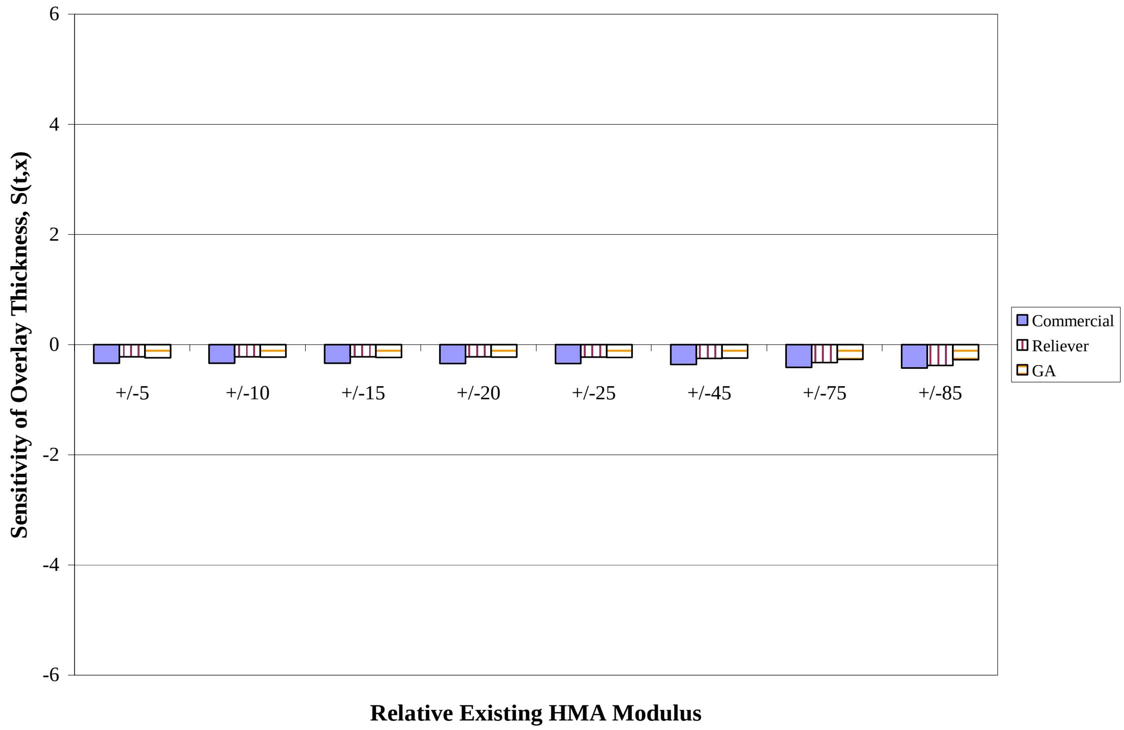


Figure 37. Summary of design sensitivity to existing HMA modulus.

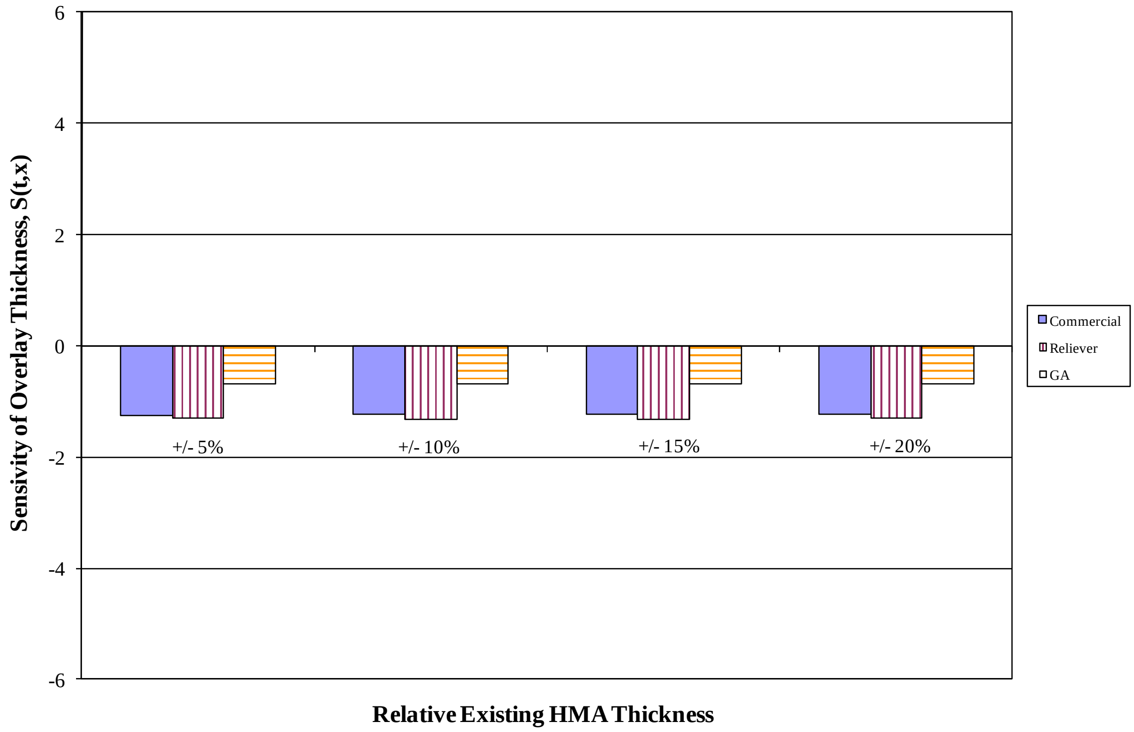


Figure 38. Summary of design sensitivity to existing HMA thickness.

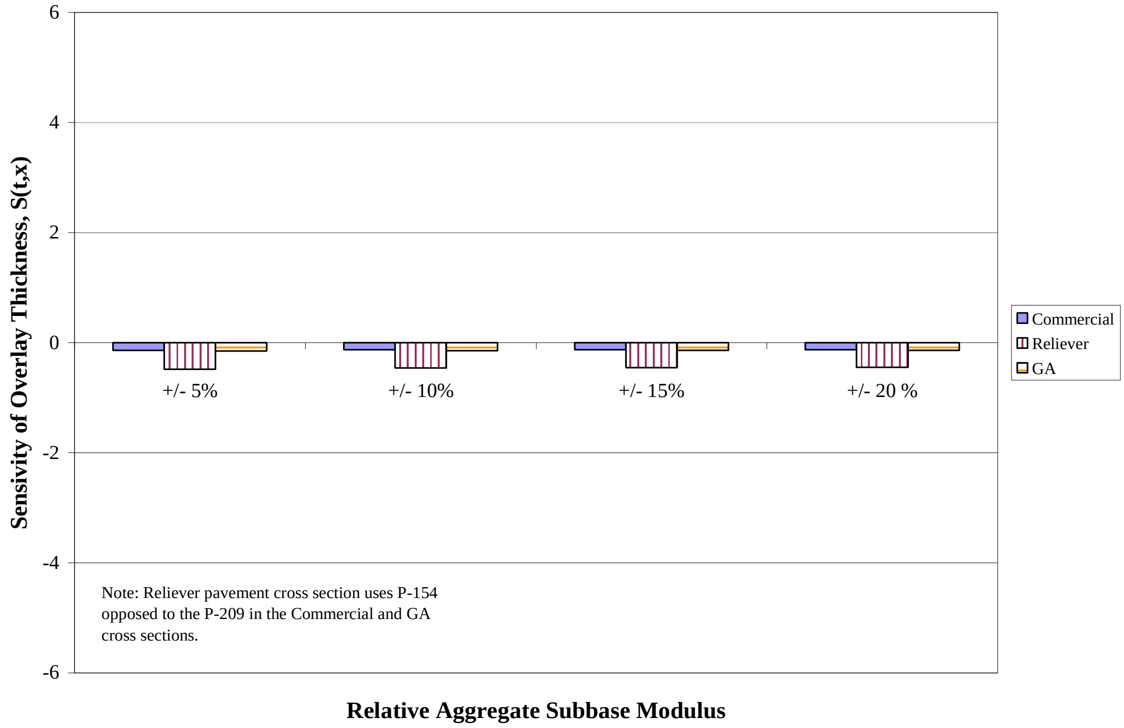


Figure 39. Summary of design sensitivity to aggregate subbase modulus.

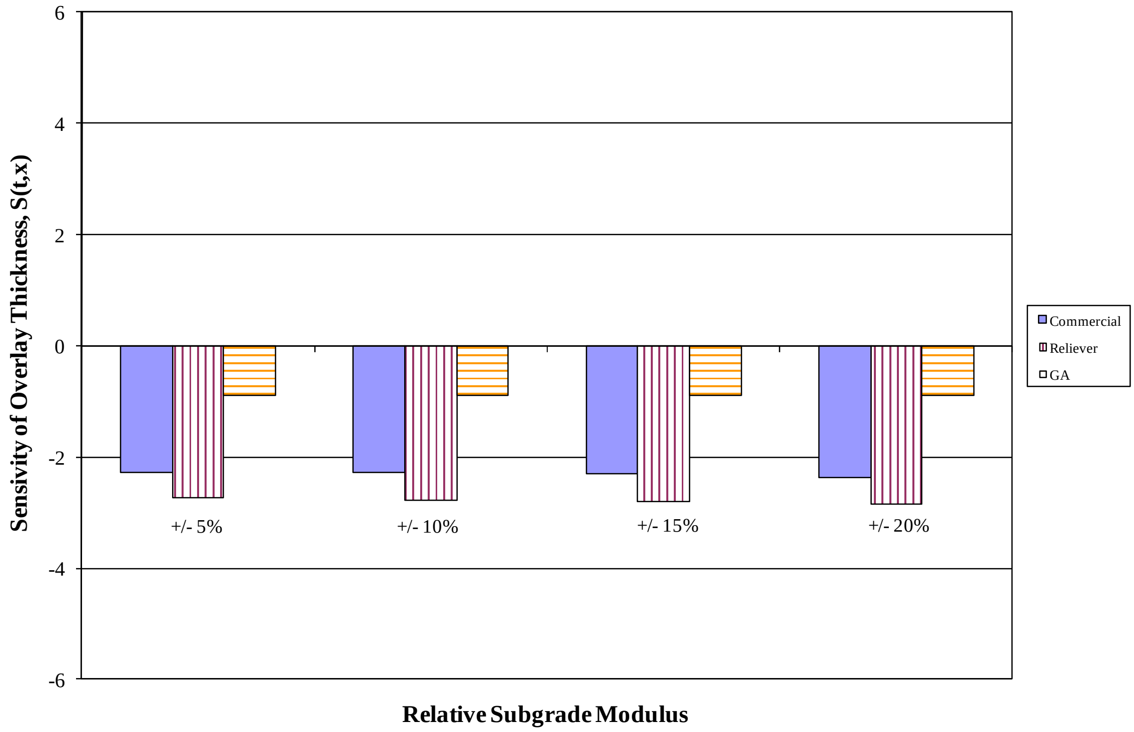


Figure 40. Summary of design sensitivity to subgrade modulus.

Another way to show the influence of the inputs is presented in figures 41 through 47; these charts present the percent change in overlay thickness for the change in design input. As illustrated in these figures, most design inputs generate less than a 10 percent change in the required HMA overlay, even with a 20 percent change in the design input; the aircraft weight, existing HMA thickness, and subgrade modulus are again the notable exceptions.

The effect of higher modulus values (i.e., beyond a 20 percent change in the default value) of the existing HMA layer and the HMA overlay on the required overlay thickness are shown in table 16.

Another consideration is the influence of design inputs on pavement remaining life or cumulative damage factor. For this analysis, the bound pavement layers are investigated for the commercial traffic mix. In this analysis, instead of determining the change in overlay thickness, the overlay thickness is held constant (except for that specific sensitivity) and the remaining life is determined using FAARFIELD for each change of input. Results obtained from FAARFIELD are summarized in table 17.

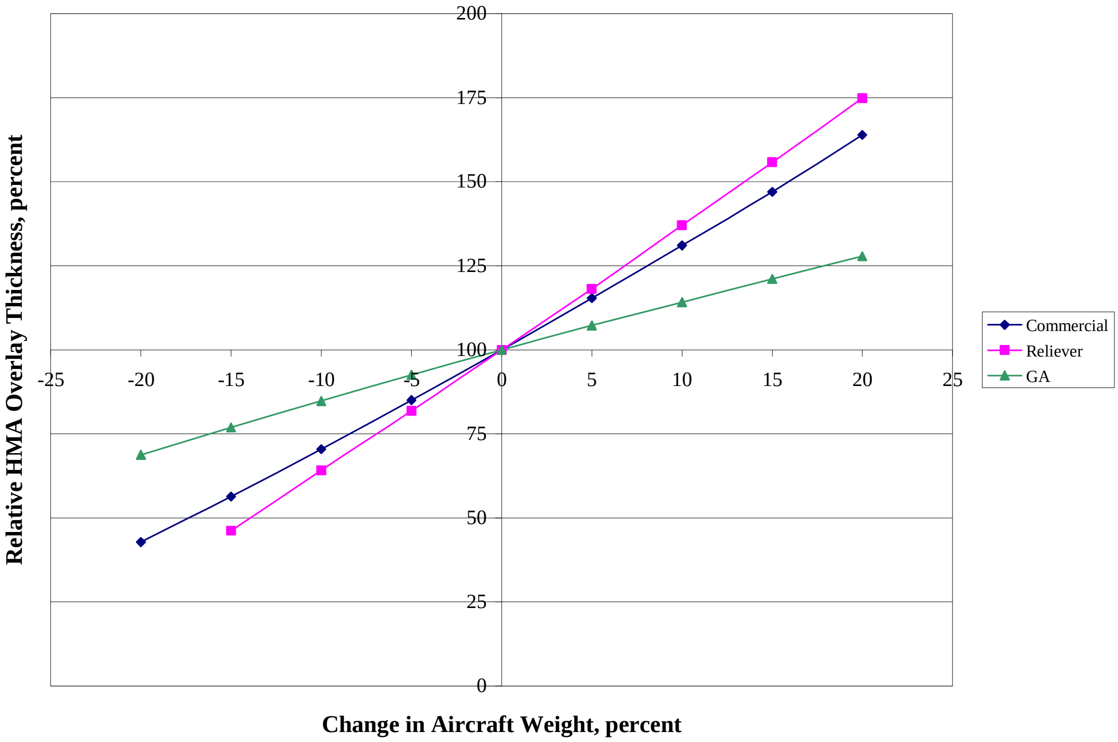


Figure 41. Change in HMA overlay thickness with change in aircraft weight.

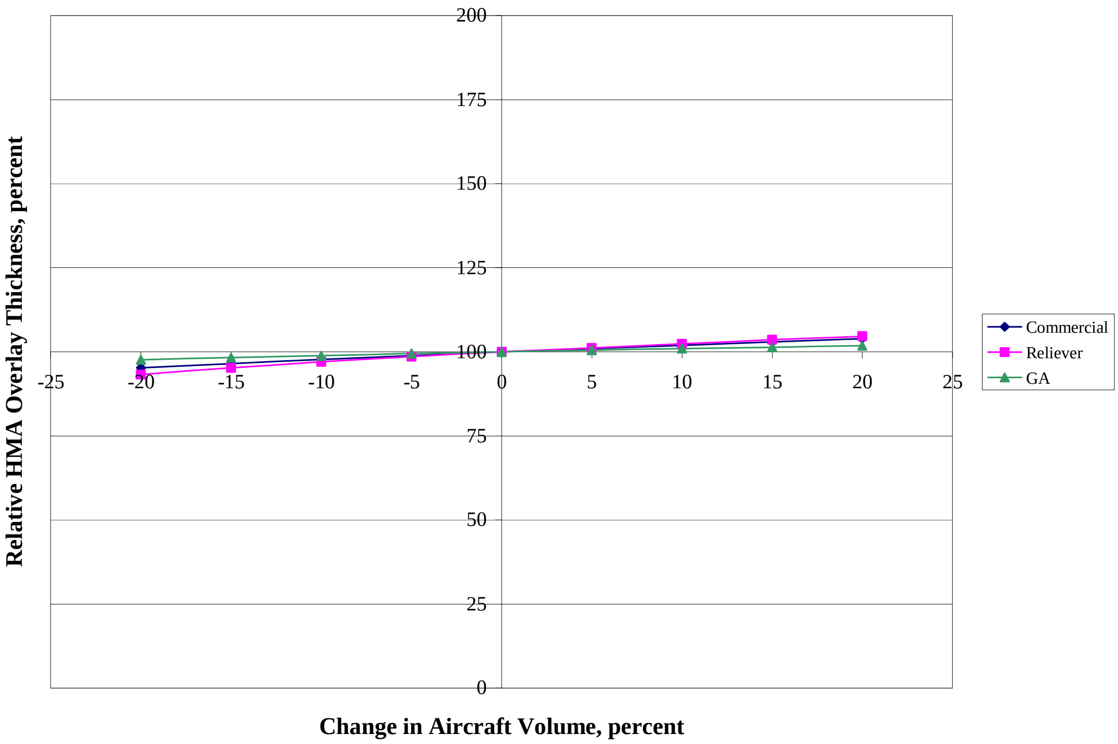


Figure 42. Change in HMA overlay thickness with change in aircraft departures.

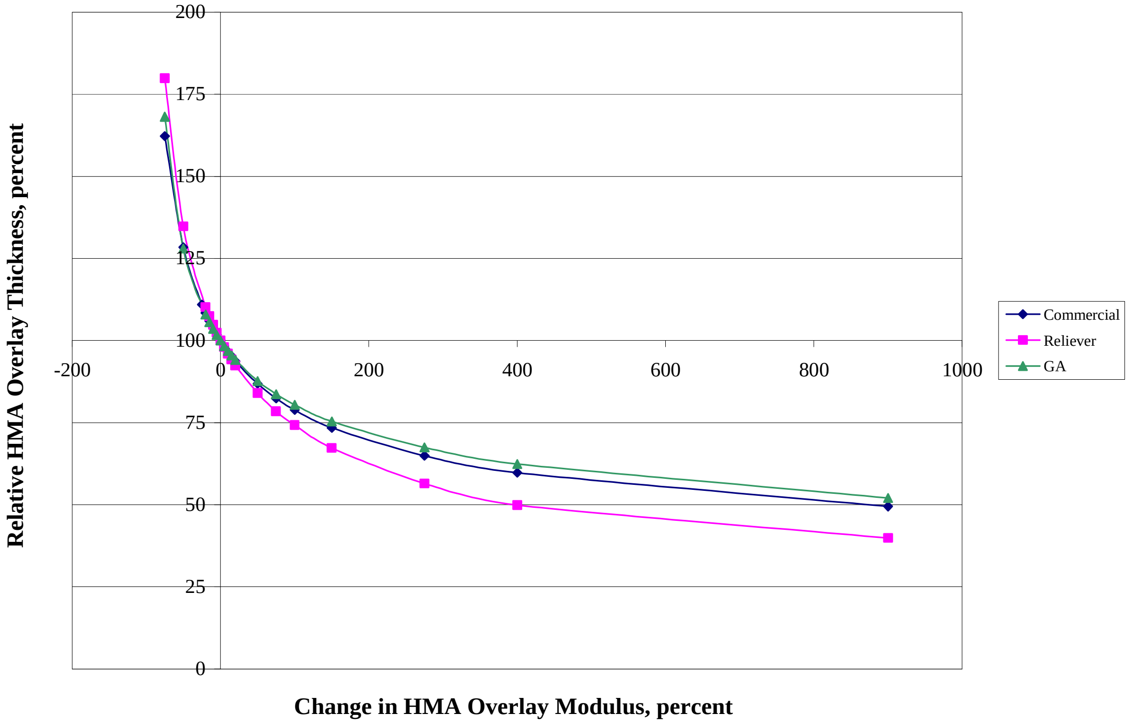


Figure 43. Change in HMA overlay thickness with change in HMA overlay modulus.

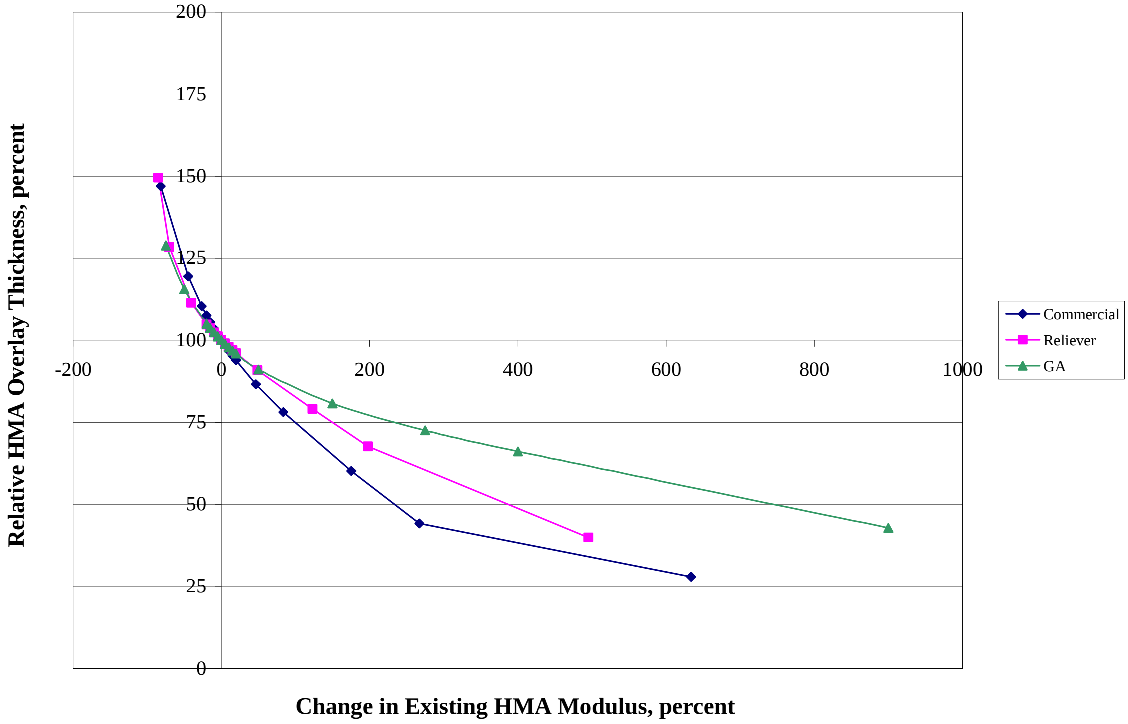


Figure 44. Change in HMA overlay thickness with change in existing HMA modulus.

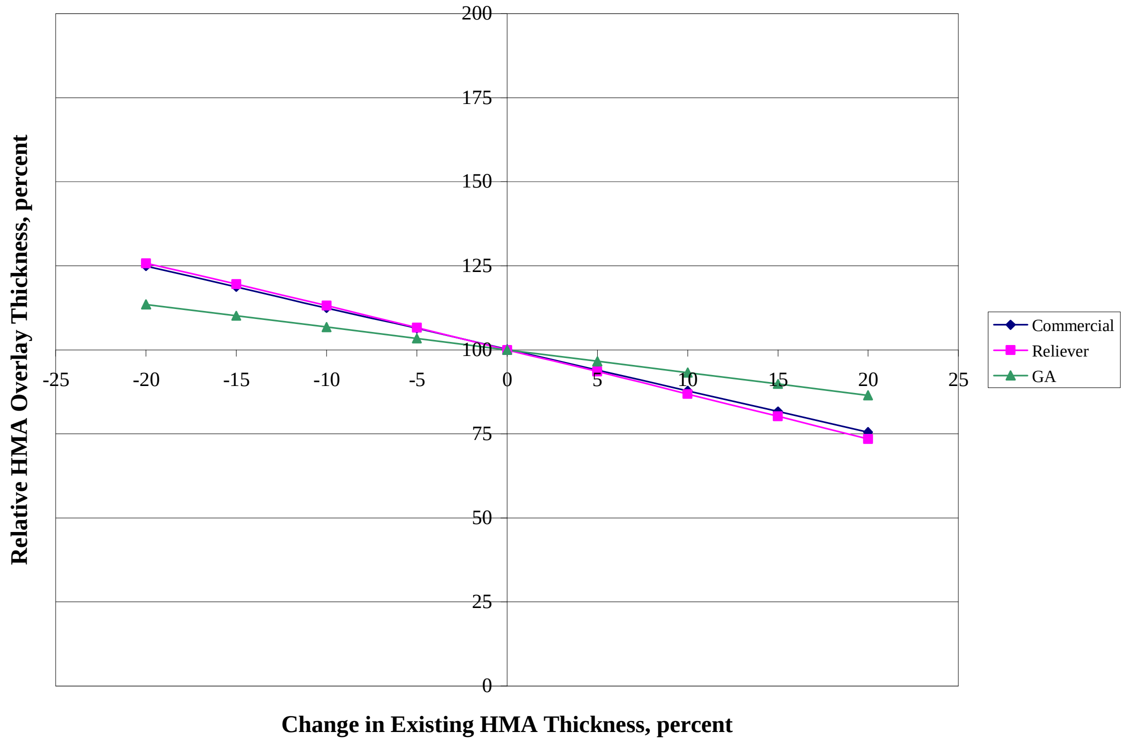


Figure 45. Change in HMA overlay thickness with change in existing HMA thickness.

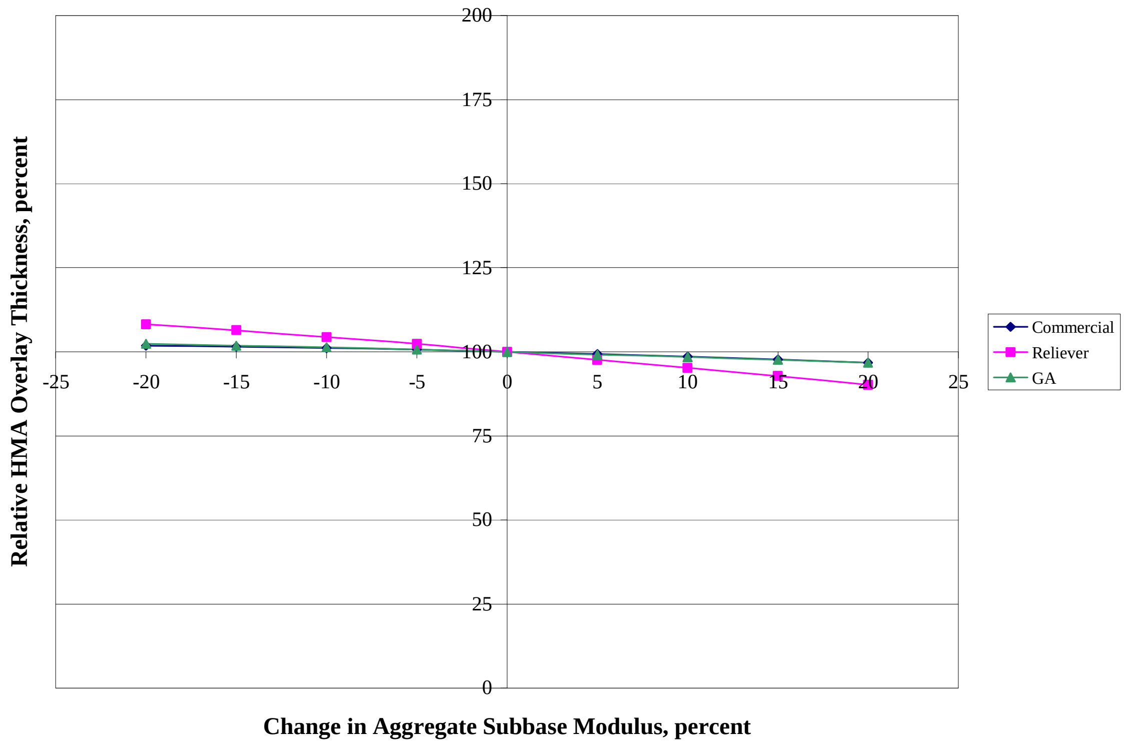


Figure 46. Change in HMA overlay thickness with change in aggregate subbase modulus.

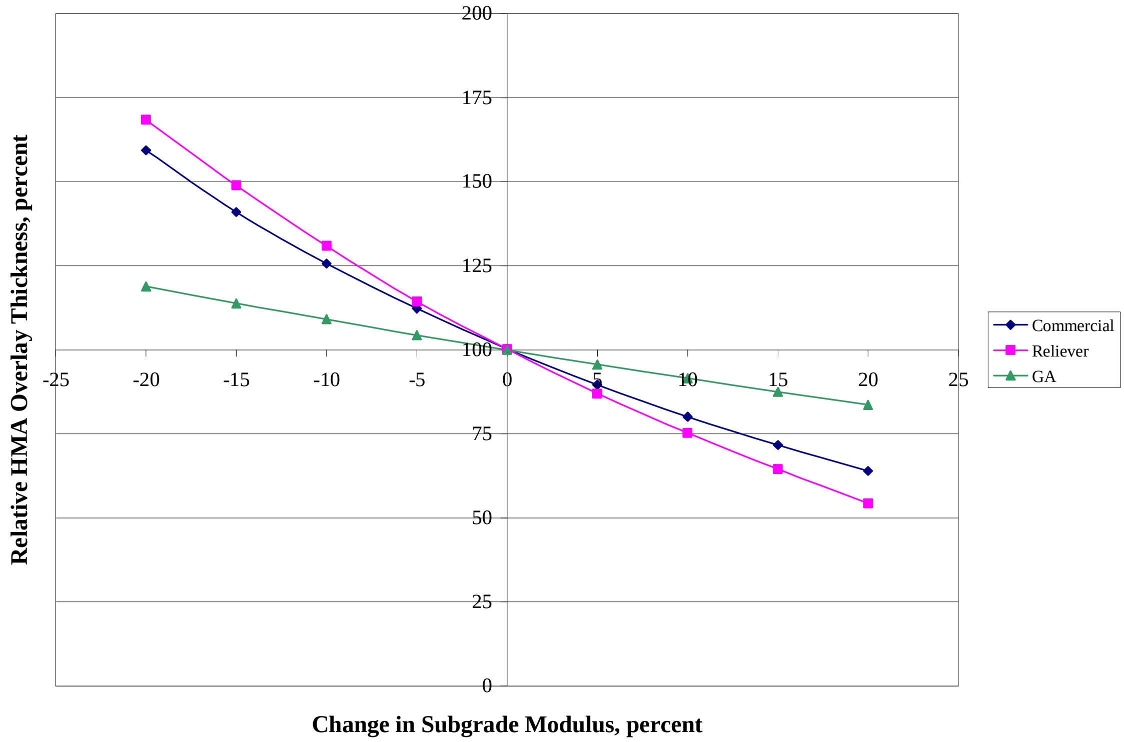


Figure 47. Change in HMA overlay thickness with change in subgrade modulus.

Table 16. Summary of FAARFIELD sensitivity runs for high HMA modulus values.

Input Change	**Required HMA Overlay Thickness, in**		
	Commercial	**Reliever**	**General Aviation**
HMA Overlay Modulus (psi)			
400,000	5.64	3.72	4.77
500,000	5.25	3.37	4.47
750,000	4.64	2.83	4.00
1,000,000	4.27	2.50	3.70
2,000,000	3.54	2.00*	3.09
Existing HMA Modulus (psi)			
400,000	6.20	4.82	5.05
500,000	5.59	4.55	4.79
750,000	4.31	3.96	4.3
1,000,000	3.16	3.39	3.92
2,000,000	2.00*	2.00*	2.54

* Minimum overlay thickness reached in FAARFIELD

Table 17. Summary of FAARFIELD sensitivity runs.

Input	Remaining Life, Years	Cumulative Damage Factor
HMA Overlay Modulus		
-20%	13.80	1.45
-15%	15.30	1.31
-10%	16.80	1.19
-5%	18.40	1.09
Baseline (200,000 psi)	20.00	1.00
+5%	21.70	0.92
+10%	23.40	0.85
+15%	25.20	0.79
+20 %	27.10	0.74
+50% (300,000 psi)	39.10	0.51
+100% (400,000 psi)	> 50	0.32
+275% (750,000 psi)	> 50	0.12
+400% (1,000,000 psi)	> 50	0.08
+900% (2,000,000 psi)	> 50	0.03
HMA Overlay Thickness		
-20%	7.50	2.68
-15%	9.70	2.07
-10%	12.40	1.61
-5%	15.80	1.27
Baseline	20.00	1.00
+5%	25.20	0.79
+10%	31.60	0.63
+15%	39.20	0.51
+20 %	48.70	0.41
Existing HMA Modulus		
-20%	14.10	1.42
-15%	15.50	1.29
-10%	17.00	1.18
-5%	18.50	1.08
Baseline (272,500 psi)	20.00	1.00
+5%	21.60	0.93
+10%	23.20	0.86
+15%	24.80	0.80
+20 %	26.50	0.75
+50% (408,750 psi)	37.5	0.53
+100% (545,000 psi)	> 50	0.34
+175% (750,000 psi)	> 50	0.20
+267% (1,000,000 psi)	> 50	0.13
+634% (2,000,000 psi)	> 50	0.04

Table 17. Summary of FAARFIELD sensitivity runs (continued).

Input	Remaining Life, Years	Cumulative Damage Factor
Existing HMA Thickness		
-20%	5.90	3.37
-15%	8.20	2.45
-10%	11.10	1.80
-5%	15.00	1.33
Baseline	20.00	1.00
+5%	26.40	0.76
+10%	34.60	0.58
+15%	45.00	0.44
+20 %	58.10	0.34

The resulting average sensitivities are summarized in figure 48. The results are relatively sensitive to each input, with the results being more influenced by layer thickness as compared to layer modulus.

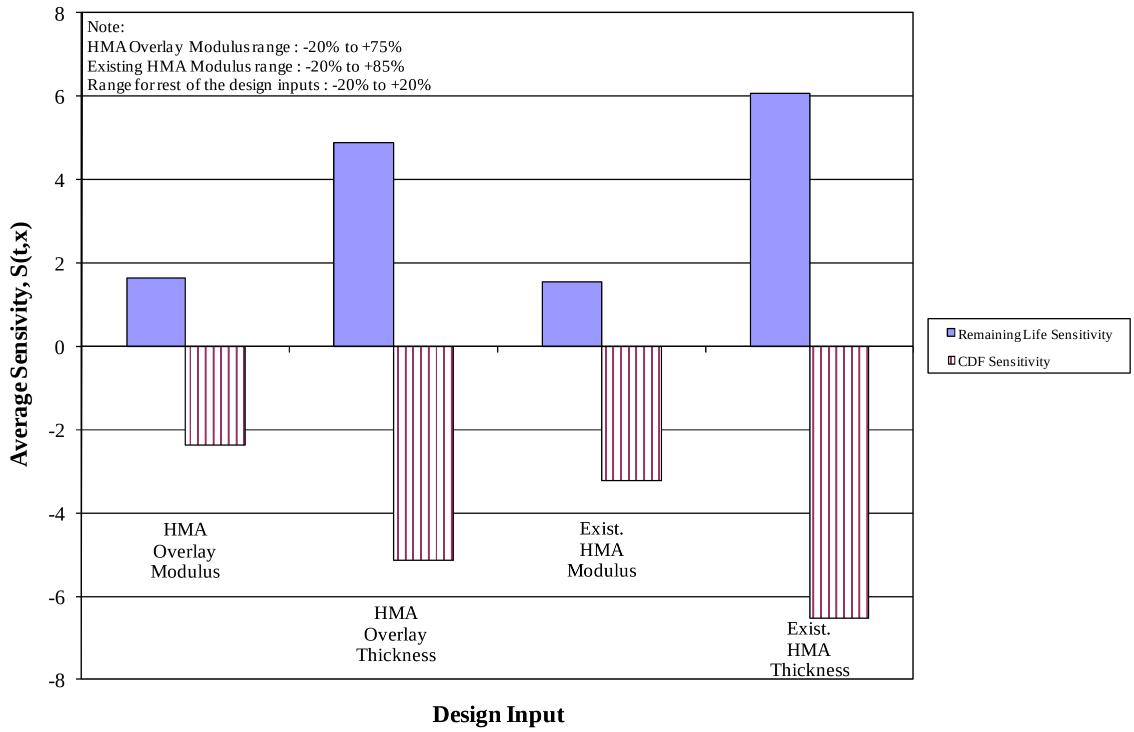


Figure 48. Summary of remaining life and CDF sensitivity to modulus and thickness.

HMA Overlay of Composite Pavement

FAARFIELD design results for the composite pavement section are summarized in table 18. Because of lengthy run times for rigid pavements in FAARFIELD, only the commercial traffic case is analyzed for the composite pavement.

Table 18. Summary of composite pavement FAARFIELD sensitivity runs.

Input Change	Required HMA Overlay Thickness, in[1]
Existing PCC Flexural Strength	
-15%	14.50
-10%	11.62
-5%	9.28
Baseline	7.16
+5%	5.35
+10%	3.60
+15%	2.19
PCC SCI	
-20%	8.81
-15%	8.24
-10%	7.74
-5%	7.47
Baseline	7.16
+5%	6.99
+10%	6.78
+15%	6.62
+20 %	6.50
PCC CDFU	
-20%	5.72
-15%	5.83
-10%	5.92
-5%	6.02
Baseline	6.12
+5%	6.22
+10%	6.33
+15%	6.40
+20 %	6.50

[1] Using commercial traffic mix.

The resulting average sensitivities are summarized graphically in figure 49. As illustrated, the most significant influence by far of the included design inputs is PCC flexural strength, followed by SCI.

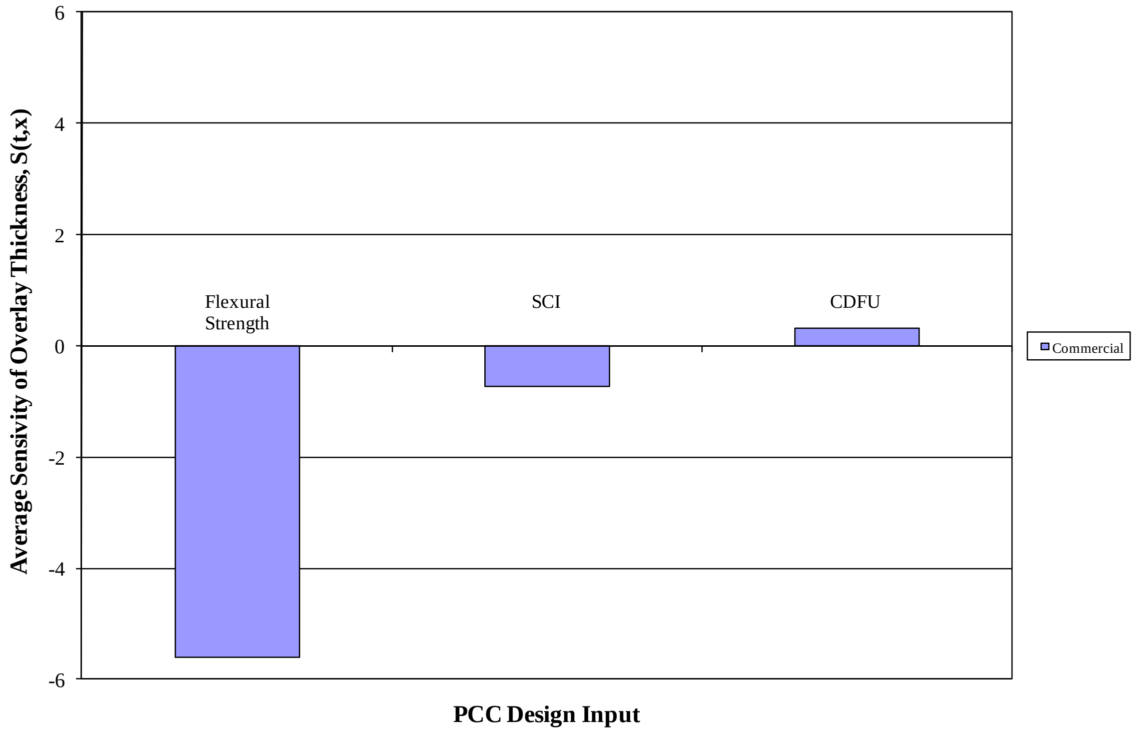


Figure 49. Summary of average design sensitivity to composite pavement inputs.

The percent change in HMA overlay thickness for the change in design input is illustrated in figures 50, 51, and 52. As illustrated, the change in overlay thickness in relation to PCC flexural strength and SCI are not as linear as for CDFU, nor as linear as the inputs for a flexible pavement. The PCC flexural strength results in the greatest relative change in overlay thickness requirement of any of the design inputs.

Additional Modeling Using FAARFIELD 1.305

During this project, the FAA released a new version of FAARFIELD (v1.305) that incorporates the computation of horizontal tensile strain and fatigue damage in all asphalt-stabilized layers. Additional runs using version 1.305 were performed to evaluate the sensitivity of the HMA overlay design parameters on the fatigue damage in the underlying asphalt layers.

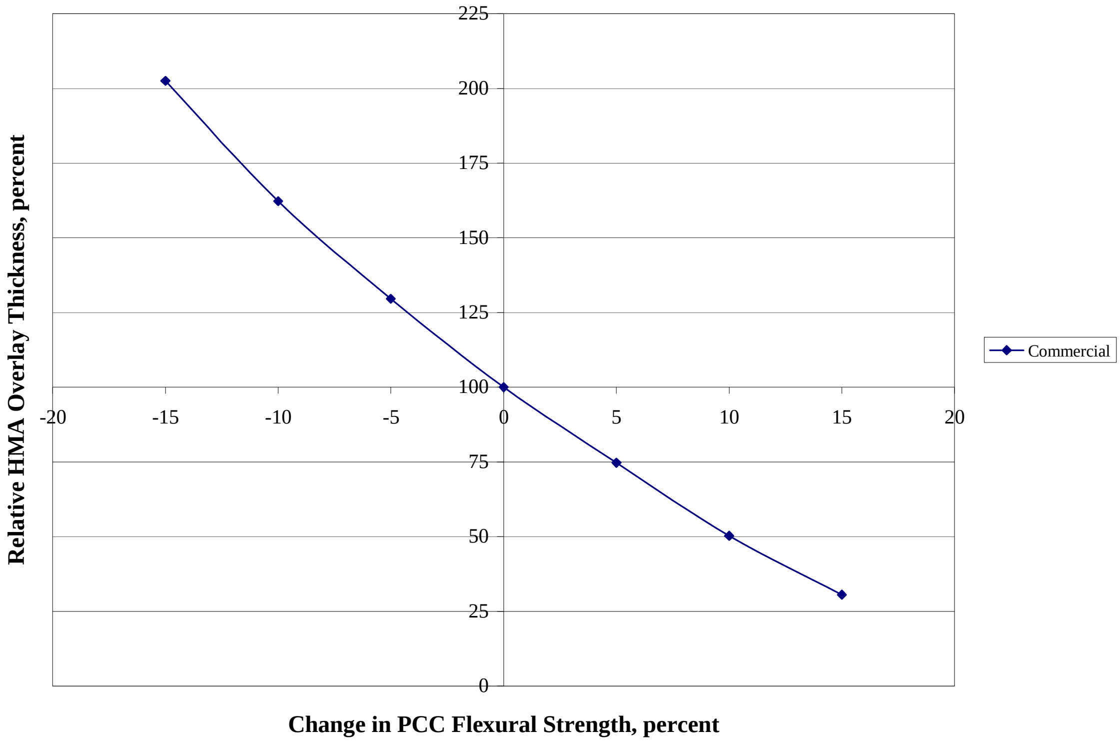


Figure 50. Change in HMA overlay thickness with change in PCC flexural strength.

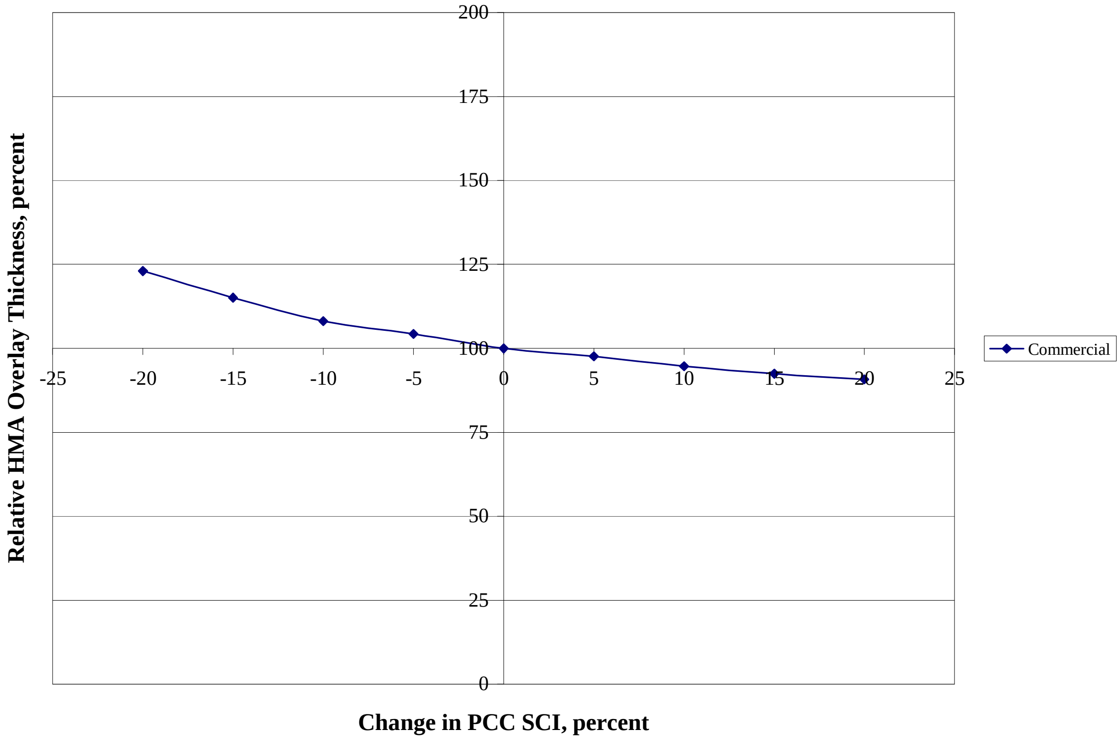


Figure 51. Change in HMA overlay thickness with change in PCC SCI.

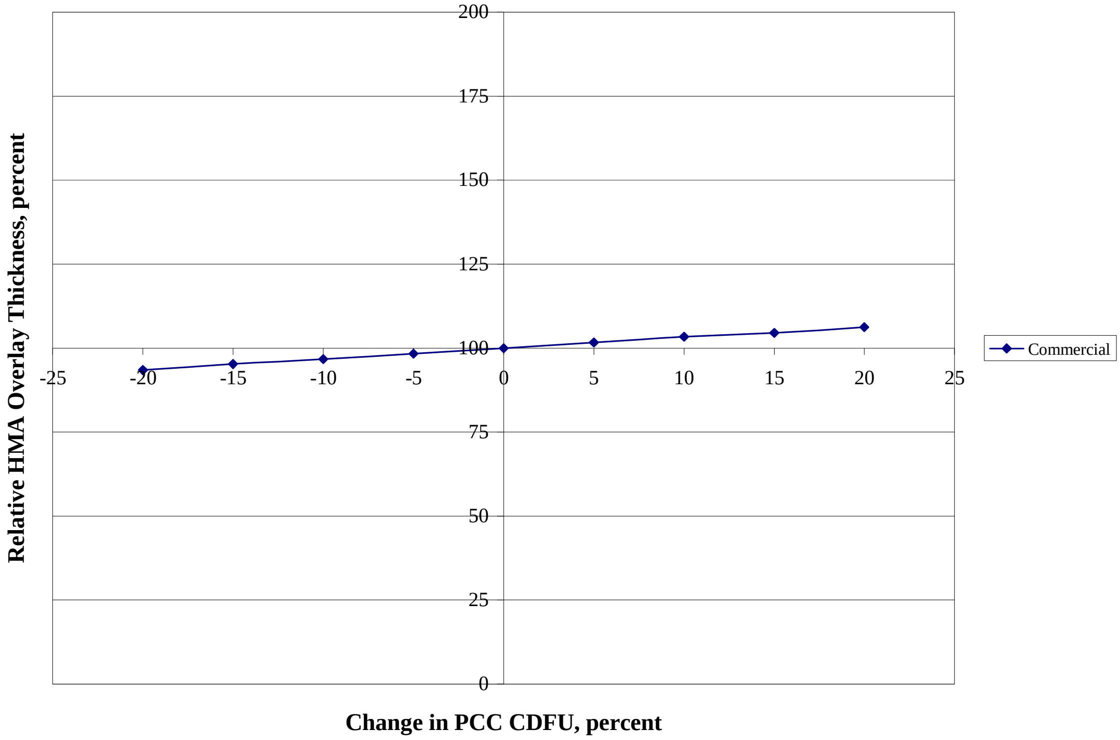


Figure 52. Change in HMA overlay thickness with change in PCC CDFU.

The sensitivity analysis using FAARFIELD 1.305 was conducted using the commercial aircraft traffic mix and the pavement layer inputs previously discussed in this chapter. The following inputs were analyzed:

- Aircraft weight.
- HMA overlay modulus.
- Existing HMA modulus.
- Existing HMA thickness.
- Subgrade modulus.

Figure 53 shows the influence of aircraft weight on the calculated fatigue damage in the HMA base layer. Fatigue damage in the HMA overlay and existing HMA surface layers were found to be negligible, while the CDF of the HMA base layer hit a maximum of 36 percent. In all cases, subgrade rutting was the controlling failure criteria.

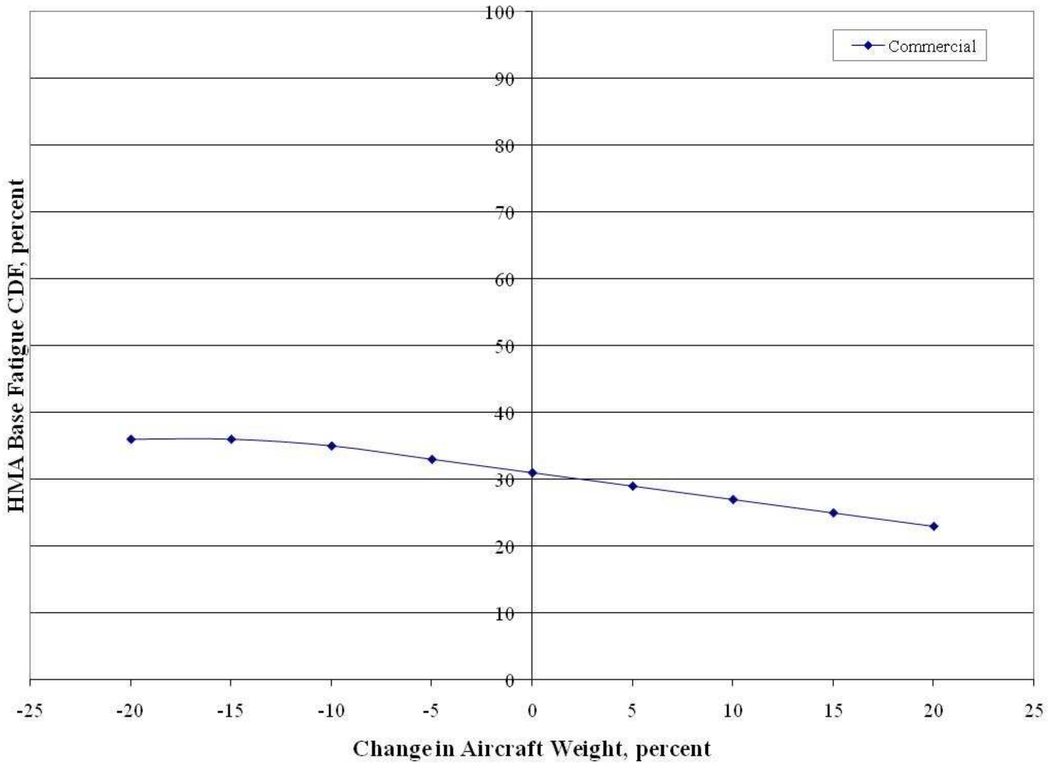


Figure 53. HMA base layer fatigue CDF with change in aircraft weight.

Figures 54 and 55 show the influence of the existing HMA surface modulus and the HMA overlay modulus on the required HMA overlay thickness. Figure 54 shows that the properties of the existing HMA surface do not typically influence the failure mode: the required overlay thickness based on rutting is generally greater than the thicknesses required to address HMA fatigue. HMA fatigue only becomes the controlling criteria when the modulus of the existing HMA surface is very low (below 100,000 psi). However, when the existing HMA surface has deteriorated to such an extent, it would not likely be a good candidate for an overlay.

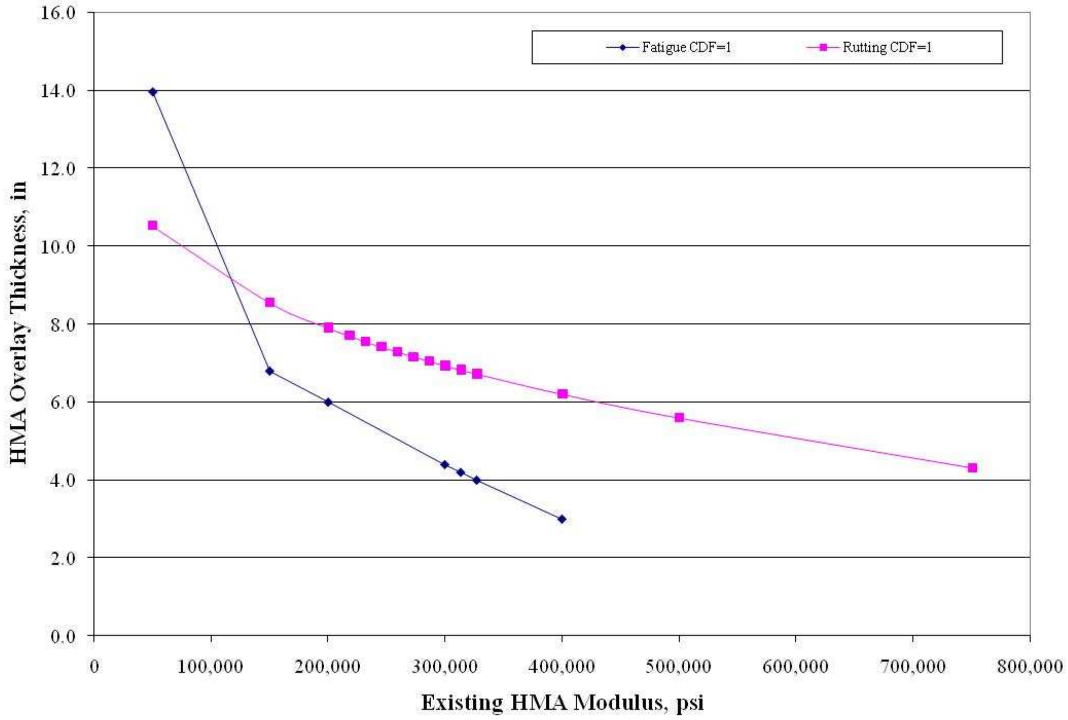


Figure 54. Influence of existing HMA modulus on the HMA overlay thickness.

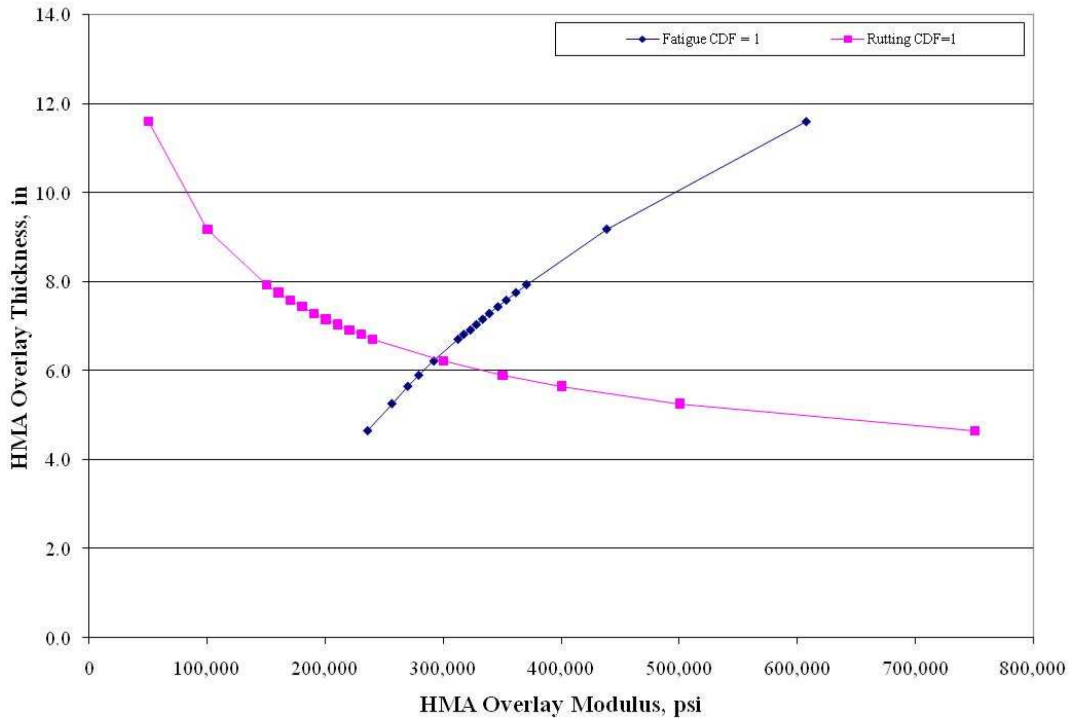


Figure 55. Influence of HMA overlay modulus on the HMA overlay thickness.

As illustrated in figure 55, the modulus of the HMA overlay does have a significant impact on the required overlay thickness to address HMA fatigue, particularly when it exceeds 300,000 psi. Therefore, if a stiff overlay mixture is anticipated, the HMA fatigue failure criteria should be checked.

Figure 56 shows the effect of a change in the existing HMA surface thickness on the fatigue CDF of the HMA layers. The HMA overlay fatigue CDF is negligible in the design cases considered, while the existing HMA surface fatigue CDF is between 45 and 50 percent.

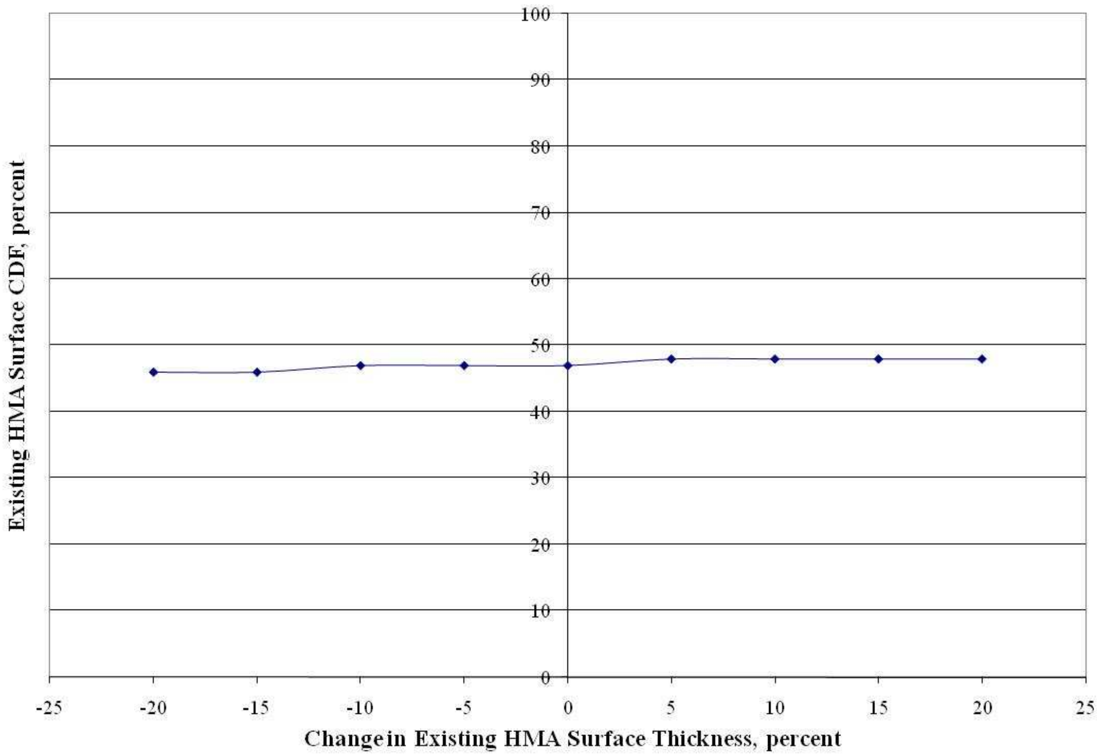


Figure 56. Change in existing HMA surface CDF with change in thickness of existing HMA surface.

Figure 57 shows the influence of the subgrade modulus on the HMA overlay thickness. The required overlay thickness for HMA fatigue is much less than the thicknesses for rutting failure throughout the range of subgrade moduli. In fact, when designing to meet fatigue requirements, the calculated subgrade damage is typically orders of magnitude higher. Hence, the thickness required to address subgrade rutting is the controlling factor.

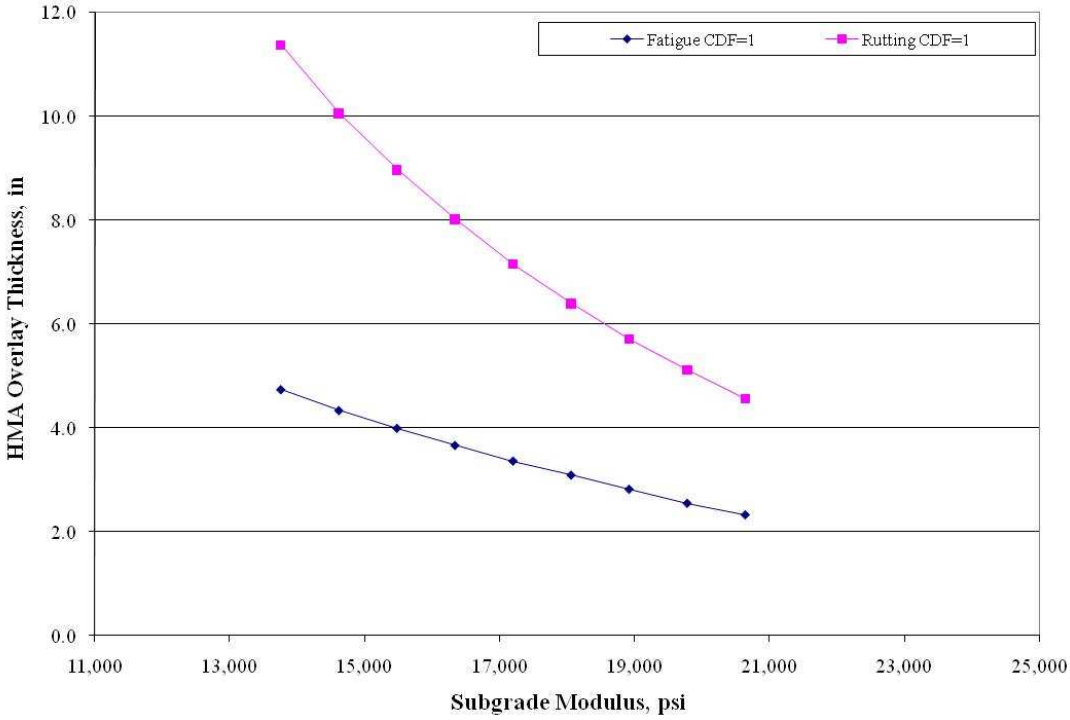


Figure 57. Influence of subgrade modulus on HMA overlay thickness.

Discussion

While aircraft traffic, particularly aircraft weight, plays a significant role in the thickness design, the means and methods of collecting traffic data are not included in this study. Rather, the focus is on the characterization of the existing pavement layers. Based on the analyses conducted, in terms of their effect on overlay thickness for flexible pavements, the existing materials-related inputs have the following relative ranking from greatest to lowest effect:

- Subgrade support.
- HMA thickness.
- HMA modulus.
- Granular base/subbase modulus.

Based on the sensitivity analyses performed, the significance of changes to the inputs is less for the general aviation aircraft mix than for commercial or reliever aircraft mixes.

For composite pavements, the strength of the PCC material has the greatest influence. What the composite pavement design doesn't directly consider is the condition of the existing HMA overlay. Unless the overlay is completely removed during rehabilitation through milling or other means, this layer needs to be accounted for in the design (discussed further elsewhere).

As mentioned previously, Advisory Circular 150/5320-6E indicates the importance of characterizing the existing pavement layers, but it does not provide much guidance on how to account for existing conditions within the rehabilitation design. The following discussion primarily focuses on the use of deflection testing (and backcalculation results) and visual distress data (such as PCI data) for characterizing the existing pavement layers.

In Advisory Circular 150/5320-6E, subgrade support continues to be based on correlation to laboratory CBRs using the following relationship:

$$E_{subgrade} = 1{,}500 * CBR \qquad \text{(Eq. 23)}$$

Determining subgrade CBR or elastic modulus can be done with a variety of evaluation techniques and respective correlation equations. The determined subgrade property can be directly input into FAARFIELD within the allowable range. Because subgrade rutting is the primary performance model used in FAARFIELD and the resulting overlay thickness is very sensitive to this input, care should be taken in determining a design value.

Typical values for the other pavement layers are provided in table 19. As summarized, in-place modulus values can vary considerably from the default layer type modulus in FAARFIELD. FAA Advisory Circular 150/5370-11A provides some guidance on selecting design inputs based on backcalculation, which are summarized in table 20. These are, however, limited to base and subbase layers.

The use of the "undefined" layer is likely most appropriate for the HMA surface layer when the existing modulus is more than 10 percent different than the default P-401 moduli in the program. For example, based on the sensitivity results, an existing HMA modulus of 160,000 psi modeled with the default layer type (200,000 psi—a 20 percent difference) would result in an overlay design that is essentially ½-inch deficient, resulting in a 30 percent reduction in structural life of the pavement.

An alternative to using an undefined layer for the existing HMA is to determine an equivalent thickness based on the ratio of the modulus. The existing HMA thickness has a greater influence than the existing HMA modulus (a 5 percent thickness change is comparable to a 20 percent HMA modulus change), and incorporating an additional correlation step is likely introducing additional uncertainty. However, for assessing the contribution of the existing HMA overlay for a composite pavement, this is a needed step. In the design procedure, the thickness of the existing HMA overlay is subtracted from the calculated required thickness. However, the condition of the existing HMA overlay needs to be considered in selecting the thickness to subtract. One possible estimate for an effective thickness (h_e) is based on Boussinesq's equation, as follows:

$$h_e = h_{existing} * \sqrt[3]{\frac{E_{existing}}{E_{new}}} \qquad \text{(Eq. 24)}$$

Table 19. Typical modulus values and ranges for paving materials.

Material	Backcalculation		FAARFIELD Input[2]	
	Layer Type	Typical Modulus Range, psi[1]	Layer Type	Modulus Range, psi
Flexible surface	HMA (dense-graded) surface	101,500 to 3,625,000	P-401	200,000
PCC surface	PCC surface	1,450,000 to 10,150,000	P-501	4,000,000
Flexible stabilized bases	Asphalt treated base	101,500 to 3,625,000	P-401/P-403 Variable stabilized (flexible)	400,000 250,000 to 700,000
Rigid stabilized bases	Econocrete base Cement treated base Soil Cement	507,500 to 5,075,000 290,000 to 2,900,000 145,000 to 1,015,000	P-306 P-304 P-301 Variable stabilized (rigid)	700,000 500,000 250,000 250,000 to 700,000
Unbound aggregate	Granular (crushed) base Granular (uncrushed) subbase	14,500 to 217,500 7,250 to 108,750	P-209 P-154	75,000[3] 40,000[3]
Undefined	N/A	N/A	Undefined	1,000 to 4,000,000

[1] Stubstad et al. 2006
[2] FAA 2009.
[3] Initial values; final value used in design is a function of thickness and underlying layer modulus using the "Modulus" procedure.

Table 20. Typical modulus values and ranges for paving materials (FAA 2004).

Material	Backcalculated Value, psi	FAARFIELD Input, psi
Stabilized base/subbase under HMA	> 400,000	400,000
	150,000 to 400,000	Backcalculated value
	< 150,000	150,000
Cement stabilized base/subbase under PCC	> 700,000	700,000
	250,000 to 700,000	Backcalculated value
	< 250,000	250,000
Granular base and subbase	> 40,000	Use P-209
	< 40,000	Use P-154

Or another approach would be to use a weighted value, as follows:

$$h_e = h_{existing} * \frac{E_{existing}}{E_{new}} \qquad \text{(Eq. 25)}$$

where:

h_e = Effective thickness of existing HMA layer, in.
$h_{existing}$ = Thickness of existing HMA layer, in.
$E_{existing}$ = Elastic modulus of existing HMA layer, psi.
E_{new} = Elastic modulus of new HMA layer, psi.

Characterizing the underlying PCC strength in composite pavements is even more critical due to its extreme influence on the resulting overlay thickness. Although the elastic modulus for a PCC layer cannot be altered, the flexural strength can be adjusted based on backcalculation or laboratory testing results. SCI has a greater influence on design results than CDFU. Advisory Circular 150/5320-6E suggests SCI can be related to the historical C_b factor (see equation 21).

However, because the overall condition of the underlying PCC is difficult to assess, establishing either of these inputs can be difficult. History of the pavement, condition of the existing HMA overlay related to underlying conditions, and strength evaluation of the underlying PCC should all be considered in establishing SCI and CDFU. It appears to be reasonable to assume a CDFU of 100 percent and concentrate on establishing an appropriate SCI.

Determination of the appropriate input value for the granular layers depends on the layer type and thickness. Based on the sensitivity analyses, a thicker P-154 layer is about as influential as the modulus of the existing HMA (a 20 percent change in modulus is equivalent to about a 0.5-inch change in required thickness). For thinner P-209 layers, the HMA overlay thickness is not as sensitive: a 20 percent change in modulus results in approximately a ¼-inch change in required thickness. It is likely that using the default P-209 and P-154 layer types will provide adequate results. However, if layers are greater than around 12 inches thick, the modulus determined within FAARFIELD should be closely compared to the results from any field evaluations.

While FAARFIELD does not account specifically for the HMA overlay performance (such as reflective cracking or rutting within the new HMA layer), accurately characterizing the existing pavement is required to produce a structurally-sound cross section. In general, the default layer values should not be accepted without some knowledge of the in-place values, but a proper evaluation needs to be conducted to determine whether adjustments need to be made. The following steps should be considered in selecting design inputs for the design of HMA overlays using FAARFIELD:

- Use subgrade support values determined through project-level investigations.
- Consider using an "undefined" layer type when the existing HMA modulus (such as determined through NDT and/or laboratory testing) varies by more than 20 percent from the default layer modulus; differences less than 20 percent can be used but the effect on the required overlay thickness will be minimal.
- Assess the stiffness of the anticipated HMA overlay mixture, as the analyses suggest fatigue should be checked when the modulus is greater than approximately 300,000 psi.
- Correlate underlying PCC flexural strength with results from NDT and/or laboratory testing.
- Review aggregate design moduli to determine suitability of using the default layer types; evaluate changing layer type if moduli are greater than 20 percent different than those determined through evaluation.
- Assess the effective thickness of existing HMA overlay in composite pavements.
- Assuming a CDFU of 100 percent and establishing an appropriate SCI is likely a reasonable approach for composite pavements.

The design analysis within FAARFIELD assumes the stabilized layers are fully bonded. If areas of delamination are identified through evaluation of the pavement, steps need to be taken during construction to correct the problem or a much thicker HMA overlay will be required. The design also does not directly consider the performance of the HMA overlay material or consider reflection cracking.

The above discussion is based on the structural sensitivity analysis within FAARFIELD. Adjustment of inputs based on functional performance of existing pavement may also be warranted, and is discussed in Chapter 4. Additionally, adjustments should consider pre-overlay repair activities and material specifications. For example, the modification of the material used in the new HMA overlay is one method of mitigating reflective cracking, and the modification may alter the modulus of the material.

SUMMARY

HMA overlay design sensitivities were performed using FAARFIELD for flexible and composite pavement cross sections to evaluate the impact of select design inputs on the required overlay thickness. Specifically, the modulus of all pavement layers and thickness of existing HMA layers were assessed to determine the relative sensitivity of the thickness requirement to relative change in the design input. These analyses were used to assess recommendations on how to use available FAARFIELD inputs to characterize the existing pavement layers for HMA overlay design. Recommendations on other aspects to consider are also discussed.

The key findings from the sensitivity analysis are:

- Subgrade support has the greatest effect on the HMA overlay thickness for flexible pavements and, the strength of the PCC materials has the greatest influence on the HMA overlay thickness for composite pavements.
- Significance of changes to design inputs is greater as aircraft weights increase. That is, change in input has a greater effect for commercial aircraft mixes than general aviation aircraft mixes.

A summary of the key recommendations are listed below:

- The subgrade CBR (or elastic modulus) values determined using field evaluation techniques or other analytical methods can be used as the design value as opposed to using default values in FAARFIELD. Care should be taken in choosing the subgrade input because the influence on overlay thickness results.
- The use of the “undefined” layer is likely appropriate for the HMA surface layer when the existing modulus is greater than 20 percent different than the default P-401 modulus.
- The fatigue performance should be checked if the stiffness of the anticipated HMA overlay mixture is greater than 300,000 psi.
- The flexural strength of the underlying PCC in composite pavements needs to be determined accurately either by laboratory testing of retrieved samples or backcalculation.
- The effective thickness of existing HMA overlay in composite pavements should be considered.
- Assuming a CDFU of 100 percent and establishing an appropriate SCI appears to be a reasonable approach for determining HMA overlay thickness of composite pavements.

CHAPTER 4. EXISTING PAVEMENT LAYER CHARACTERIZATION

INTRODUCTION

While the implementation of FAARFIELD and Advisory Circular 150/5320-6E (FAA 2009) is a departure from the empirical design methods in previous FAA pavement design Advisory Circulars, the HMA overlay design procedure is still based on a structural deficiency approach. Determining the condition and properties of the existing pavement structure is therefore critical, but guidance for incorporating existing pavement characteristics into the design procedure is not clearly stated in the design procedure.

One of the primary objectives of this research study is to provide guidance on methods to evaluate existing pavement structures for assessing the design parameters to be used within FAARFIELD. This chapter presents a review of common evaluation techniques, including visual, destructive, and non-destructive methods, and provides recommendations on how results can be incorporated into HMA overlay design.

VISUAL PAVEMENT ASSESSMENT METHODS

The pavement condition index (PCI) procedure is the most commonly used technique for visual assessment of pavement distress for airfield pavements. A subset of the PCI data can be used to determine the structural condition index (SCI), which is generally a measure of the load-related deterioration. PCI is not directly considered in the FAA overlay design procedure, but SCI is used in the overlay design of rigid pavement structures. Although not currently included in flexible pavement overlay design, SCI might have applicability in how flexible pavements are characterized for overlay design.

Pavement Condition Index (PCI)

The PCI procedure is used to identify distress types, rate severity levels, and quantify extents of distress. A sampling procedure is used, as it is often not practical or economical to evaluate 100 percent of the pavement surface. During a PCI survey, visible signs of deterioration (i.e. distress type, severity, and quantity) within each selected sample unit are identified, recorded, and analyzed. Details of the PCI procedure are provided in the following publications:

- ASTM D 5340-10, *Standard Test Method for Airport Pavement Condition Index Surveys*.
- FAA Advisory Circular 150/5380-6B, *Guidelines and Procedures for Maintenance of Airport Pavements*.
- Unified Facilities Criteria (UFC) 3-270-05, *PAVER Concrete Surfaced Airfields Pavement Condition Index*.
- UFC 3-270-06, *PAVER Asphalt Surfaced Airfields Pavement Condition Index*.

There are many benefits to performing PCI inspections. For one, the PCI procedure is the industry standard for visually rating pavement surface condition on airfields. It provides a consistent and systematic methodology to identify and rate distresses on the pavement surface and to report general pavement condition. The type, severity, and extent of individual distresses provide an indication of maintenance and repair needs as well as a basis for establishing rehabilitation priorities in the face of constrained resources.

Furthermore, the results of repeated PCI monitoring over time can be used to determine the rate of deterioration. These data can be used to estimate the time at which certain maintenance or rehabilitation measures should be implemented. Pavements that exhibit significantly higher than average deterioration rates can also be examined to determine the reason for the accelerated deterioration; in this manner significant performance problems can be identified and addressed early in the life of the pavement, potentially avoiding the need for complete reconstruction.

The PCI scale ranges from 100 (representing a pavement in excellent condition) to 0 (representing a pavement in failed condition). The PCI scale is shown in figure 58. In broad terms, pavements with a PCI of 71 to 100 that are not exhibiting significant load-related distress will benefit from preventive maintenance actions, such as crack sealing and patching. Pavements with a PCI of 41 to 70 typically require major rehabilitation, such as an overlay. Often, when the PCI is less than 40, reconstruction is the most viable alternative due to the substantial deterioration of the pavement.

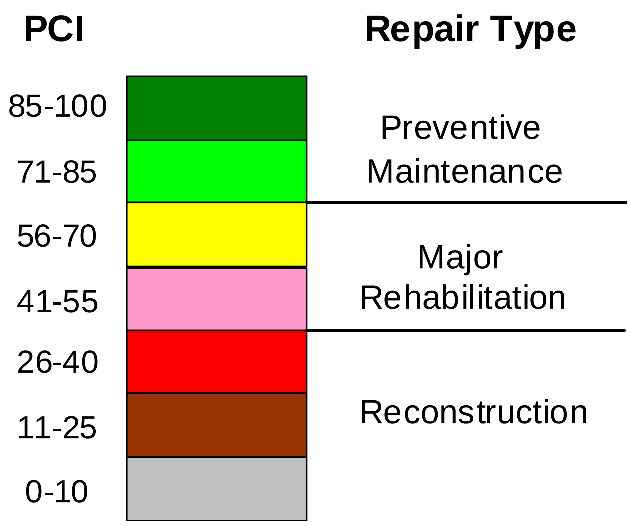


Figure 58. PCI scale with general repair categories.

As figure 58 shows, PCI ratings can be used as a general guideline in identifying the appropriate repair type, and they are also useful in assessing what distress categories (or types) are causing a reduction in the condition rating. The PCI procedure divides distress types into three categories based on the expected cause of the distress: load-related, climate-related, and other causes. By knowing the cause(s) of pavement deterioration, appropriate repair and rehabilitation alternatives can be identified.

Load-related distresses are caused by aircraft loadings and their presence usually indicates a structural deficiency; these distresses are of concern when evaluating the structural capacity of an existing HMA pavement and often indicate the need for additional pavement structure. Climate-related distresses often signify the presence of aged and/or environmentally susceptible materials. The "other" category includes distresses that do not have load or climate as the primary cause for deterioration, such as material- or construction-related distress and patches. Each distress type is listed in table 21, where they are grouped by the typical causative factor.

Table 21. Distress types and primary distress categories.

Pavement Type	**Pavement Distress Category**		
	Load-Related	**Climate-Related**	**Other**
HMA-Surfaced Pavements	• Fatigue (Alligator) Cracking • Rutting*	• Block Cracking • Joint Reflection Cracking • Longitudinal and Transverse Cracking • Patching • Raveling • Weathering	• Bleeding • Corrugation • Depression • Jet Blast • Oil Spillage • Polished Aggregate • Shoving • Slippage • Swelling
PCC Pavements	• Corner Break • Linear Cracking • Shattered Slab	• Blow-up • Durability Cracking • Joint Seal Damage	• Small/Large Patch • Popouts • Pumping • Scaling, Map Cracking, and Crazing • Faulting/Settlement • Shrinkage Cracking • Spalling, Joint/Corner • Alkali-Silica Reactivity (ASR)

* Rutting may occur due to "other" factors, such as from material or construction problems. While rutting requires an applied load, it can occur in the HMA or aggregate layers form material problems (such as unstable HMA, which should be corrected in the mix design) or construction problems (such as poor compaction of any pavement layer).

While the PCI rating is a numerical measure of the existing condition of the pavement based on the distresses observed on the pavement surface, it has limitations regarding the assessment of material and structural integrity of a pavement structure. The procedure only accounts for visible signs of deterioration on the pavement surface, although many distresses initiate, and are often worse, below the surface. Furthermore, it ultimately uses a single value to represent the contribution to pavement performance of a combination of factors, which may include different causative mechanisms. Also, this procedure is somewhat subjective, as it depends on the inspector's judgment.

While one of the primary benefits of the PCI procedure is an estimate of when maintenance or rehabilitation work should occur based on surface condition, it does not necessarily address what specific rehabilitation activity actually needs to be done. For example, the presence of load-related distress often indicates that a structural improvement is needed, but it does not quantify the structural deficiency.

Additionally, the PCI procedure does not provide enough information to pinpoint specific areas for repair. Often times, a detailed mapping of the distresses in conjunction with a PCI survey will provide better information for planning HMA overlays and associated pre-overlay repairs. A detailed distress map can also more effectively illustrate where distresses may be occurring in greater densities, indicating areas that may need to be evaluated in more detail.

Structural Condition Index (SCI)

The SCI uses the structural distress information collected from a PCI inspection to provide a structural rating of the existing pavement—essentially performing a PCI analysis excluding any non-structural distress. Similar to PCI, SCI is rated on a 100-point scale, with 100 indicating no structural distress present. Because it only uses the load-related distresses, the SCI will always be equal to or greater than the PCI.

The distresses included in SCI for flexible pavements are not generally agreed upon. The U.S. Army Corp of Engineers (USCOE) uses the HMA distresses listed in table 22 in the calculation of SCI. However, in the PCI procedure the load-related distresses are assumed to only include fatigue cracking and rutting. Additionally, only fatigue cracking and rutting are used to calculate SCI in the FAA's *Operational Life of Airfield Pavements*. Even though rutting is associated with a load-related distress, it may be caused by material instability in the HMA layer (opposed to load-related deformation of the subgrade).

Table 22. HMA pavement distress types used to calculate the SCI.

Distress	**Severity Level**
Alligator (Fatigue) Cracking	Low, Medium, High
Depression	Low, Medium, High
Longitudinal and Transverse Cracking	High
Patching	Medium, High
Rutting	Low, Medium, High
Slippage Cracking	(degrees of severity are not defined)

Because SCI is based on the PCI, it has some of its same advantages, such as systematically quantifying deterioration, monitoring deterioration and deterioration rates over time, and providing an indication of when rehabilitation is needed. However, it also means the SCI has some of the same limitations as the PCI, including being somewhat of a subjective measure of condition that relies on the inspector's judgment and it only accounts for visible signs of damage on the pavement surface. Additionally, the SCI does not quantify other HMA performance factors due to construction or material issues, including but not limited to stripping, slippage cracking, or debonding.

Use of Condition Indices in Design Procedure

The PCI is not directly accounted for in the overlay design procedure. However, as previously mentioned, the FAA incorporates the SCI of the underlying PCC pavement into the design of HMA overlays over rigid pavements. The design procedure does not account for SCI for the design of HMA overlays of existing flexible pavements.

For existing composite pavements, the FAA defines an SCI of 80 of the underlying PCC as structural failure (FAA Advisory Circular 150/5320-6E), which is consistent with 50 percent of slabs in the traffic area containing a low-severity structural crack. FAARFIELD allows the SCI input to range from 100 to 67. Chapter 2 also discusses how SCI is applied for rigid pavements in the updated FAA design procedure.

Advisory Circular 150/5320-6E states that the SCI provides a more precise and reproducible rating of the pavement condition than the condition factors (C_b and C_r) in the previous thickness design procedure (Advisory Circular 150/5320-6D). However, if SCI is not available, the FAA provides an estimate for SCI from C_b and C_r factors using the following correlations:

$$SCI = 100 * C_b - 25 \qquad \text{(Eq. 26)}$$

$$SCI = 93.2 * C_r + 7.1 \qquad \text{(Eq. 27)}$$

where:

SCI = Structural condition index computed from PCI data.
C_b = Condition factor of existing PCC for HMA overlay design ($0.75 \leq Cb \leq 1.0$).
C_r = Condition factor of existing PCC for PCC overlay design.

The SCI is used to establish increments of deterioration of the underlying PCC slab for adjusting the PCC modulus, as illustrated in figure 59 (Kawa, Brill, and Hayhoe 2007). The calculated modulus for each increment is used for determining the stresses in the pavement system. The terminal SCI is either 40 or 57, depending on the base/subbase structure, and the design stops when the terminal SCI is reached (FAA 2009).

At this time, there is no established correlation of HMA pavement failure to SCI. This topic is discussed in some detail in *Operational Life of Airport Pavements* (Garg, Guo, and McQueen 2004), which concludes that additional research of SCI regarding HMA pavement evaluation is needed. However, one possible option is to define “failure” for HMA pavements at an SCI of 80, as is done with PCC pavements.

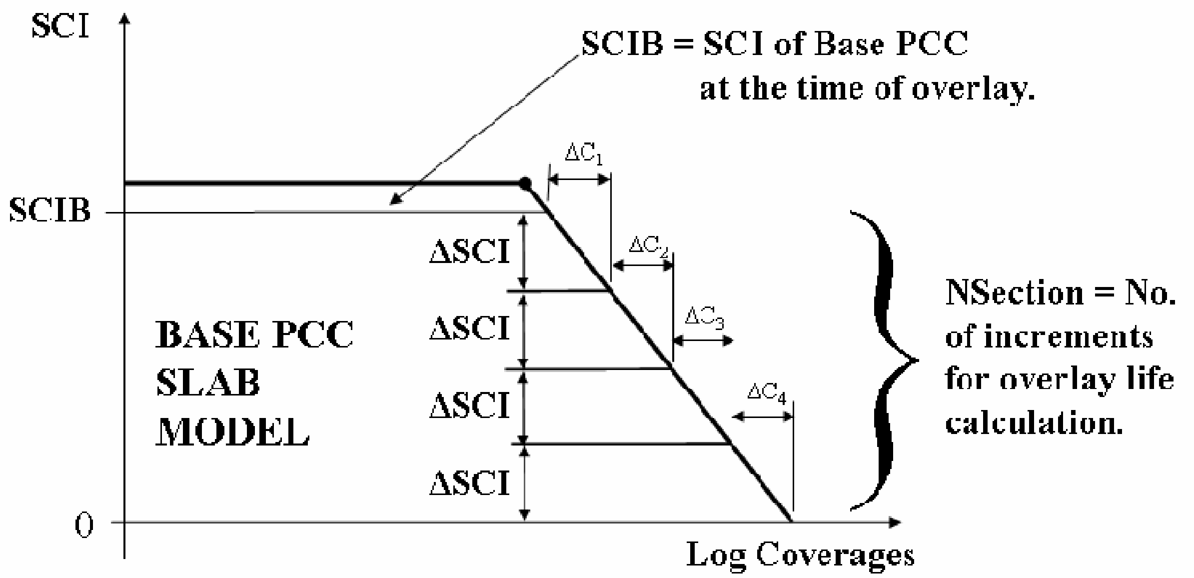


Figure 59. Illustration of incremental PCC deterioration (Kawa, Brill, and Hayhoe 2007).

To correlate an SCI for HMA pavements, the PCI distress deterioration curves can be considered. The deduct curves used in the PCI calculation process are presented in figures 60 and 61 for fatigue cracking and rutting, respectively. A pavement with an SCI of 80 has a 20-point deduct applied. While this is a simplified example (when several distresses are present, the PCI and SCI calculations are a bit more complicated due to correction factors being applied), an SCI of 80 equates approximately to any one of the following conditions, as further illustrated in these figures:

- 0.15% of the area with high-severity fatigue cracking.
- 0.35% of the area with medium-severity fatigue cracking.
- 0.95% of the area with low-severity fatigue cracking.
- 0.10% of the area with high-severity rutting.
- 0.45% of the area with medium-severity rutting.
- 2.5% of the area with low-severity rutting.

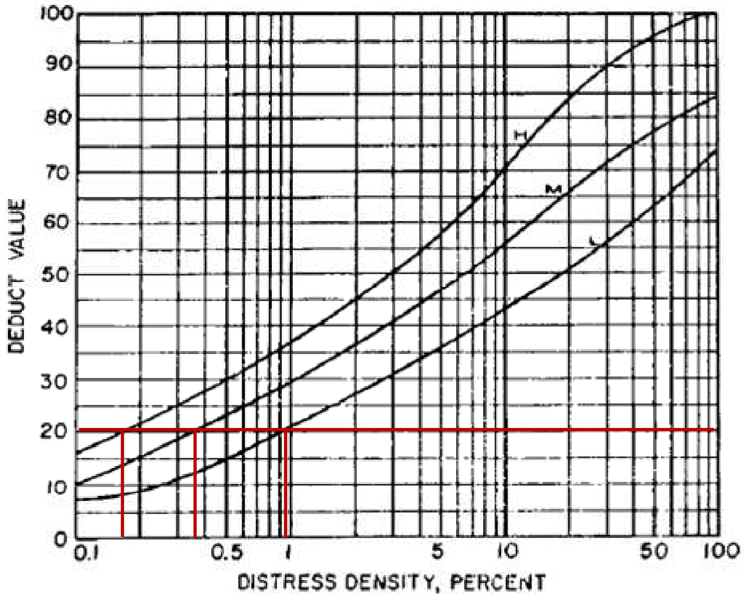


Figure 60. PCI fatigue cracking deduct curve (ASTM D5320-10).

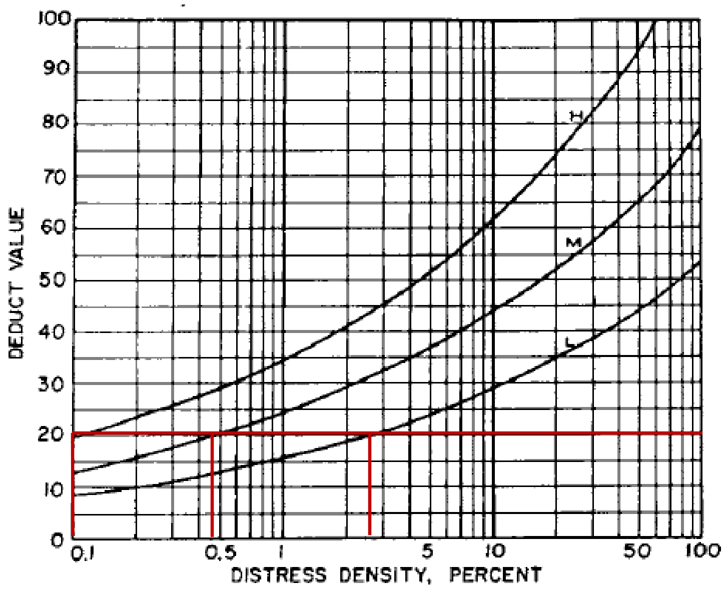


Figure 61. PCI rutting deduct curve (ASTM D5320-10).

For an HMA-surfaced pavement to have an SCI of 80 or above, less than 2.5 percent of the pavement surface can exhibit load-related distress, which becomes less than 1 percent for all distresses except low-severity rutting. Alternately, when fatigue cracking exists in 50 percent of the wheel path, which represents approximately 2 percent of pavement area (assuming the keel is 50 feet wide and the fatigue cracking spans a 1-foot width) has low-severity fatigue cracking there is approximately a 27-point deduct, or an SCI of 73. If rutting were defined the same way, 2 percent of the area having low-severity rutting corresponds to an SCI just above 80.

Visual Inspection Case Study

To demonstrate how this concept might apply with actual airport pavement data, table 23 presents PCI, SCI, and the results of a structural analysis (including estimated remaining life and HMA overlay needs) from an airport study. In this example, some sections have subsections noted by “a”, “b”, or “c” following their section identifier; these sections had notable differences in structural condition but were not distinguished as separate sections when the PCI inspection occurred.

In many cases, sections with 20 years of calculated remaining life (based on deflection testing and FAA thickness design) also have no structural distress, but this correlation is not true in all instances. There are some cases where the calculated remaining life is less than 5 or 10 years, but no load-related distress is observed; there are also instances where structural distress is present, but the calculated remaining life is at least 20 years. There are several possible causes for this apparent inconsistency, including design parameters (such as traffic) that are not entirely appropriate, repairs/recent rehabilitation that mask structural distresses, or simply variations in pavement performance. Based on the literature review and the cursory case study, SCI does not have an apparent consistent correlation with overall performance.

Table 23 also illustrates differences in estimated functional and structural lives. Non-structural distresses, such as extensive block cracking, weathering/raveling, and so on, have an effect on the overall exiting condition (and determining the need for a non-structural overlay), and these distresses will influence the future performance of the overlay (such as reflection cracking of underlying thermal cracks).

Table 23. Example of PCI, SCI and percent distress due to load, in conjunction with structural analysis results.

Airport	Branch	Section	Surface Type	Last Const. Date	PCI	SCI	Remaining Life (PCI), years[1]	Structural Remaining Life, years	HMA Thickness Deficiency, in
A	Runway	10	AAC	2001	97	100	11 - 20	6 - 10	0.3
	Taxiway	10a	AAC	2004	96	100	11 - 20	0 - 5	0.8
	Taxiway	10b	AAC					0 - 5	5.4
B	Runway	10	AAC	1992	85	95	11 - 20	0 - 5	1.8
	Taxiway	10	AC	1990	69	75	0 - 5	0 - 5	0.8
C	Runway	10	AAC	1999	69	80	0 - 5	0 - 5	2.1
	Taxiway	10	AC	1999	60	69	0 - 5	0 - 5	2.1
	Apron	10	AAC	1999	49	59	0 - 5	0 - 5	2.1
D	Runway	10a	AAC	2006	100	100	11 - 20	20+	0
	Runway	10b	AC					20+	0
E	Runway	10	AAC	1995	85	100	11 - 20	20+	0
	Runway	20	AC	2003	100	100	11 - 20	20+	0
	Taxiway	10a	AAC	2003	100	100	11 - 20	20+	0
	Taxiway	10b	AAC					20+	0
	Taxiway	20	AC	2003	75	75	0 - 5	0 - 5	1.0
F	Runway	10	AAC	2004	97	100	11 - 20	20+	0
	Taxiway	10	AAC	2004	100	100	11 - 20	20+	0
	Taxiway	20	AAC	2004	100	100	11 - 20	0 - 5	2.0
	Apron	10	AAC	1991	68	80	0 - 5	0 - 5	0.5
G	Runway	10	AAC	2002	96	97	11 - 20	0 - 5	1.4
	Taxiway	10	AAC	2002	79	85	0 - 5	0 - 5	2.2
	Apron	10	AAC	2002	89	96	11 - 20	0 - 5	3.8
H	Runway	10	AAC	1997	62	94	0 - 5	20+	0
	Taxiway	10	AAC	1997	67	79	0 - 5	20+	0
	Apron	10	AAC	1997	62	67	0 - 5	20+	0
I	Runway	10a	AAC	1996	89	100	11 - 20	20+	0
	Runway	10b	AAC					20+	0
	Runway	10c	AAC					20+	0
	Taxiway	10	AAC	2005	95	100	11 - 20	20+	0
	Apron	10	AAC	1996	81	98	6 - 10	20+	0
J	Runway	10	AAC	1996	91	99	11 - 20	20+	0
	Taxiway	10	AAC	1996	82	96	6 - 10	20+	0
	Apron	10	AAC	1996	89	99	11 - 20	20+	0
K	Runway	10a	AAC	1990	80	100	6 - 10	20+	0
	Runway	10b	AAC					20+	0
	Taxiway	10	AAC	1990	58	64	0 - 5	20+	0

[1] Projected time when major rehabilitation is needed (i.e., time until section reaches the critical PCI).

DESTRUCTIVE TESTING METHODS

Destructive testing on existing airfield pavements often includes coring or boring activities (or sample retrieval). While destructive testing methods can provide valuable information and are often needed to validate or calibrate non-destructive test results (discussed later), the biggest concern is typically the impact on airfield operations, especially on high-use runways or single-runway airports. Additionally, core data only provide a glimpse of the pavement structure at the specific location where the core is taken, and there are often a very limited number of cores retrieved from each site due to the required time and expense.

Pavement Coring and Soil Boring

It is generally recommended to retrieve cores as part of a pavement evaluation project. The number of cores may vary, depending on historical cross section information, results of non-destructive evaluation (discussed later), and observed conditions throughout the project area. As outlined in FAA Advisory Circular 150/5320-6E, general boring guidelines are to bore at 200-ft intervals along the length of runways and taxiways, with varying offsets; bores are recommended at a rate of one per every 10,000 square feet on aprons. Existing pavement thickness is a critical input for analysis of data collected using non-destructive testing methods, and for overlay designs. Coring data provides (or confirms) layer thicknesses of bound materials.

Core data can also provide information regarding the integrity of the existing materials by offering a glimpse of subsurface conditions. Bonding conditions between pavement layers can be observed, in addition to potential problems such as slippage or stripping. Figure 62 shows three examples of subsurface deterioration revealed by cores. The degree of bonding between the layers, especially the asphalt-stabilized layers, can greatly affect the pavement behavior and performance, and the manner in which the existing pavement should be analyzed for overlay design needs to be considered. FAARFIELD does not currently allow for adjustments to the existing pavement layers or bonding conditions based on such findings. While not routine, the pavement cores retrieved during sampling can be used in laboratory material testing (such as dynamic modulus testing).

If borings are performed, additional information on unbound base, subbase, and subgrade thicknesses and material properties can be collected. Subgrade assessment, through in-place testing or sampling and laboratory testing, is commonly conducted to determine general subgrade classification and CBR, the latter of which can be correlated to modulus. Variations in subgrade conditions and areas with fill can also be identified. Even if borings are not performed, additional information can be collected from the location where cores have been removed, such as by performing dynamic cone penetrometer (DCP) testing to determine in-place CBRs.

While not commonly done because of the significant disruption that would occur for airfield traffic, conducting a trench excavation also provides valuable information, such as the amount of rutting contributed by each pavement layer.

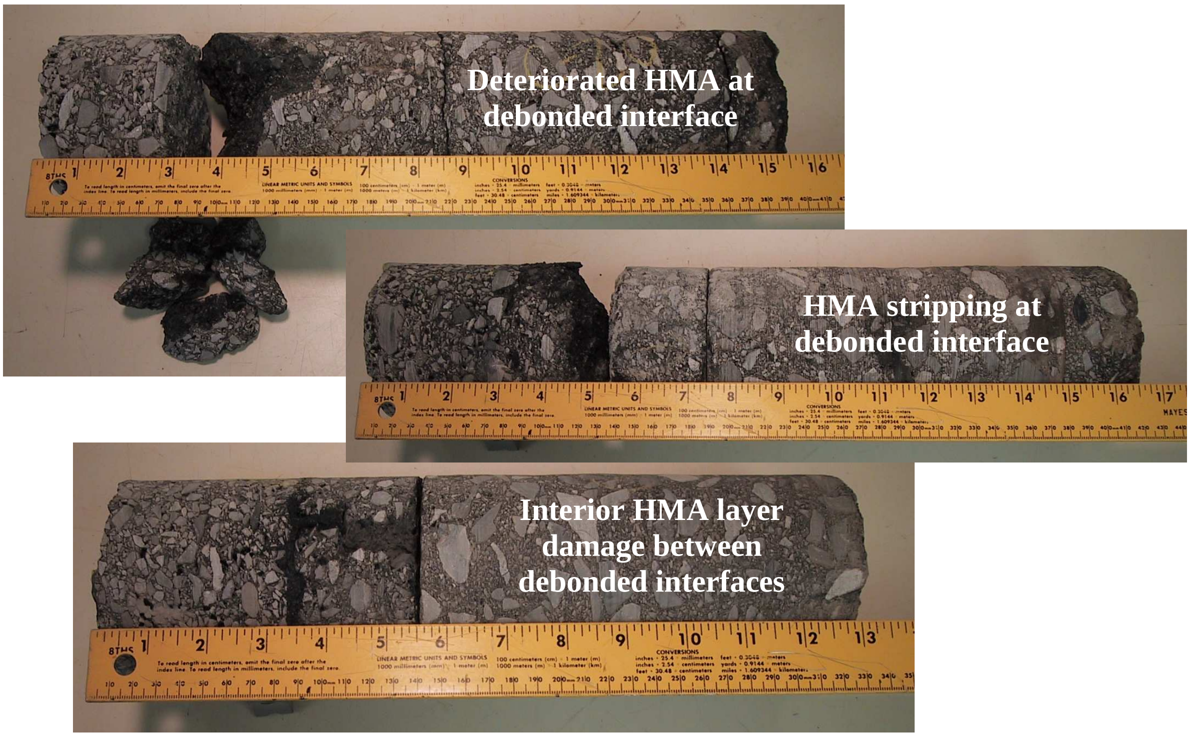


Figure 62. Observed subsurface HMA deterioration revealed from coring.

Dynamic Cone Penetrometer (DCP) Testing

A DCP is a simple testing device consisting of a long steel rod with a steel cone (or point) attached to the end. A weight is dropped a given height to drive the cone into the ground. By measuring the penetration of the cone against the number of drops of the weight (the DCP Index), it is possible to plot resistance to penetration, which provides an indirect measure of the strength/compaction of the layer. The DCP Index can then be correlated to a CBR using manufacturer recommendations and Air Force Engineering Technical Letter 02-19: *Airfield Pavement Evaluation Standards and Procedures* (AFCESA 2002).

DCP testing can be used to supplement nondestructive testing data and is especially useful to test unbound material where cores have been removed. Not only can these data be used to determine layer stiffness, but changes in resistance can be used to distinguish unbound layer thicknesses. This test is less costly than taking bores and performing laboratory testing because it can be done quickly in the field with relatively inexpensive equipment; however, this is still a destructive testing method and is often used in conjunction with laboratory testing and/or nondestructive testing. This test method does not work well when large aggregate (greater than approximately 2 inches) are present in the layer or as the CBR of the unbound material approaches 100.

Laboratory Testing

For airfield pavement evaluation projects, laboratory testing is most often performed on unbound materials collected from bores. These data are often analyzed for a subset of the retrieved materials for use in confirming field observations and field testing results (both destructive and non-destructive). One disadvantage is that destructive methods must be employed to retrieve the materials. Also, the cost of performing this testing typically limits the amount of data analyzed.

CBR and resilient modulus testing are the most common laboratory subgrade tests performed as part of a geotechnical study. FAARFIELD allows the input of either value, and assumes the resilient modulus is 1500 times the value of the subgrade CBR. One disadvantage of laboratory testing of subgrade material is that the results are for stress-state conditions during testing, and these are often not the same as those for the in-place material and pavement cross section.

When PCC cores are retrieved, it is not uncommon for compressive strength testing to be completed, or for select cores to be analyzed for material problems (such as petrographic analysis to detect alkali-silica reactivity [ASR]). However, similar testing on HMA cores is less common. HMA material testing has been incorporated into design procedures for roadway evaluations (such as in AASHTO's MEPDG), but there may be applications for these tests in airfield pavement evaluation and design. Although not part of FAA standards for evaluations, these laboratory tests (such as dynamic modulus testing on retrieved HMA cores) may hold merit for airfield pavement evaluation to help establish or validate moduli of the existing asphalt materials used in overlay design.

NONDESTRUCTIVE TESTING METHODS

NDT can be used for determining many properties of an existing HMA pavement structure, including layer moduli, layer thicknesses, and areas of subsurface deterioration, such as stripping and delamination. There are many types of devices available, generally falling into one of the following categories:

- Impulse – falling weight deflectometer (FWD), lightweight deflectometer (LWD).
- Electromagnetic – ground penetrating radar (GPR).
- Sonic/Ultrasonic/Seismic – impact echo, ultrasonic surface waves (USW), ultrasound.
- Thermal – infrared thermography.
- Vibratory – Dynaflect, Road Rater.

FWD testing is the most widely used NDT device, but the use of other methods, such as GPR and seismic analysis, are becoming more common. These methods are highly effective when used in concert, particularly on thick pavement structures. For example, the use of GPR, as well as seismic methods, such as the Portable Seismic Pavement Analyzer (PSPA), has been found to be effective in cases of near-surface deterioration. The GPR is able to pick up the presence of moisture, while the PSPA is able to determine the moduli of layers within 10 inches of the surface and is relatively unaffected by the conditions/thickness of the bottom layers.

Falling Weight Deflectometer (FWD) Testing

The FWD is a load impulse testing device capable of simulating in magnitude and duration the dynamic loads applied to a pavement by heavy aircraft wheel loads. It can also be adjusted to simulate the lighter loadings experienced on general aviation airports. A dynamic load is applied by dropping a set of masses onto a circular load plate, as illustrated in figure 63. A range of load magnitudes is achieved by varying the number of masses used and the heights from which they are dropped. The actual load magnitude is measured using a load cell within the load plate, and the resulting deflections are measured by transducers located at the center of the load plate and at preset distances away from the load plate.

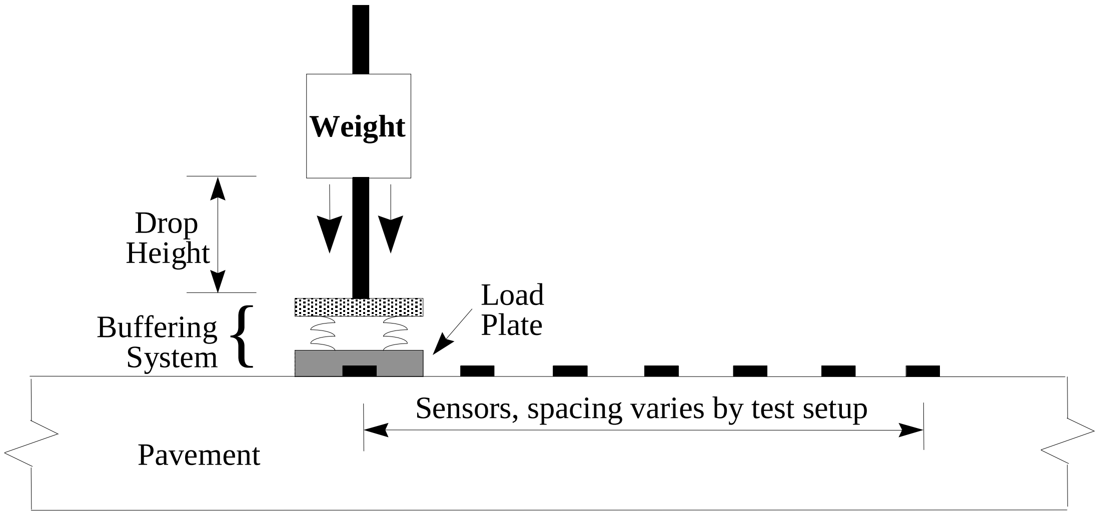


Figure 63. Schematic diagram of FWD testing equipment.

Testing and analysis procedures and guidance for using the results are provided in the following publications:

- FAA Advisory Circular 150/5370-11A, *Use of Nondestructive Testing in the Evaluation of Airport Pavements*.
- UFC-3-260-03, *Airfield Pavement Evaluation*.

FWD testing allows for a direct analysis of the structural integrity and load-carrying capacity of a pavement. Deflection testing data are used to backcalculate *in situ* material properties, to calculate pavement responses to load, and as inputs to other pavement analyses and designs. Deflection testing provides inputs for the structural analysis of the pavement sections without costly and time-consuming material testing required of destructive testing and analyses.

Normalized Deflections

Normalized deflections are used to analyze the load/deflection response of the pavement and to provide statistics to characterize the structural integrity of the entire pavement system. The measured FWD dynamic deflections are normalized to a chosen load to easily evaluate how response varies along the pavement. Because the FWD directly simulates the loading of a heavy aircraft, it is not necessary to introduce factors to correlate the pavement response to the typical wheel load.

The normalized deflection profile can be used to assess changes in subgrade support, variations within a pavement section, and the relative structural condition of a pavement. While the deflections vary mainly due to differences in pavement cross section, areas with comparatively high deflections or high standard deviations (typically greater than 25 percent of the average) can indicate deterioration within the pavement structure, either globally or locally. Alternatively, Impulse Stiffness Modulus (ISM) can be plotted to provide the same general information as the normalized deflection data. ISM is defined as the applied load divided by the deflection at the center of the load plate.

Two example normalized deflection plots from flexible pavement structures are presented: figure 64 presents data from the runway of a commercial airport, and figure 65 presents data from a runway at a general aviation airport (corresponding to the airport indicated as “I” in table 23). In the first figure, there is one obvious pavement change 2,000 feet from the runway end and another, less distinct change at about 7,000 feet. These observations were consistent with known cross section information. Another observation is that the outer test lanes (50 feet right and left of the centerline) have a different pavement response than those closer to the centerline, which was also consistent with the outer portions of the runway having a known thinner pavement structure than the keel. This is a good example of how changes in pavement response can be identified by FWD testing.

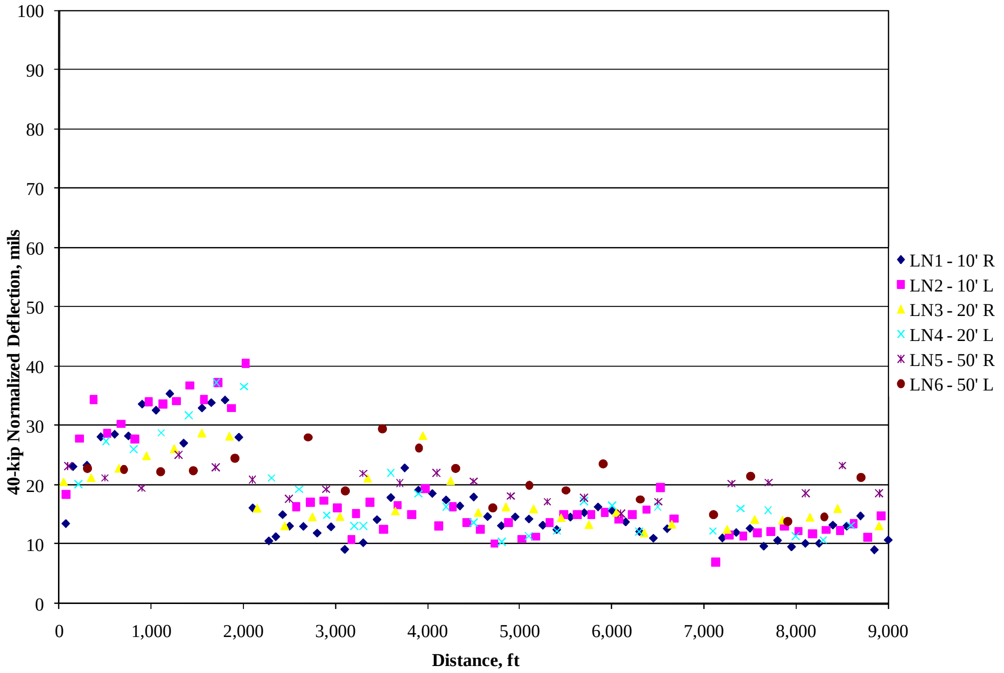


Figure 64. Example normalized deflection profile plot from a commercial airport.

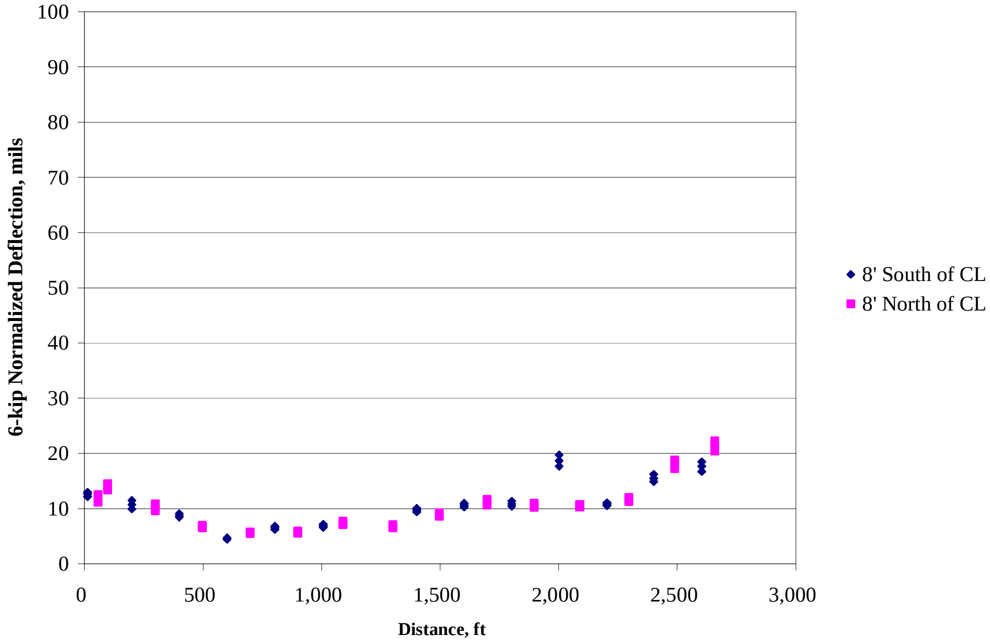


Figure 65. Example normalized deflection profile plot from a general aviation airport.

The runway represented in figure 65 consists of only one pavement section identified for pavement management purposes (based on observations during the PCI inspection, the pavement condition was fairly consistent along the length of the runway, and construction plans were not available). As evident in figure 65, there is some gradual deflection variation along the length of the facility, especially within 500 feet of each runway end. Based on data from three cores, the HMA thickness ranges from 4 to 8 inches without a granular base, which is the most likely contributor to this variation.

Subgrade Support

In addition to normalized deflections, pavement layer moduli and subgrade support conditions can also be determined from FWD data using appropriate backcalculation analysis methods: layered elastic methods are generally used for HMA pavements and plate theory is generally used for composite (or PCC) pavements. A subject for debate is whether the backcalculated subgrade support values (as well as other pavement layer moduli) need to be adjusted. The backcalculated values represent a field value and do not necessarily correspond with laboratory determined properties, which are the basis of most thickness design procedures, due to differences in loading conditions and stress states. The backcalculated values are often corrected to obtain a value considered consistent with laboratory testing results. While the FAA does not recommend a specific correction factor, the *AASHTO Mechanistic-Empirical Pavement Design Guide* (NCHRP 2004) can be referenced for guidance. There is also on-going FHWA research to re-assess the need for or magnitude of the corrections.

Either CBR or the subgrade resilient modulus can be directly entered into FAARFIELD, and the program will calculate the other variable. The FAA uses the following relationship to convert resilient modulus, determined from backcalculation, to CBR:

$$E_{subgrade} = 1{,}500 \times CBR \qquad \text{(Eq. 28)}$$

For a composite pavement (HMA overlay over PCC), rigid backcalculation procedures are used (such as the outer-AREA concept), and a subgrade k-value is determined rather than a resilient modulus. This backcalculated k-value is a dynamic value, which is commonly multiplied by 0.5 to represent a static value for use in design. The k-values determined from FWD testing represent not only the natural subgrade, but also the strength of all underlying unbound (granular) pavement layers. This strength, once corrected to a static value for design use, is referred to as the top-of-base k-value. Correlation between k-value and subgrade modulus is performed using the relationship provided in FAA Advisory Circular 150/5370-11A, *Use of Nondestructive Testing in the Evaluation of Airport Pavements*, as shown below:

$$E_{subgrade} = 26\,(k)^{1.284} \qquad \text{(Eq. 29)}$$

where:

$E_{subgrade}$ = Backcalculated static subgrade elastic modulus, psi.
k = Static modulus of subgrade reaction, psi/in.

Pavement Layer Moduli

Pavement layer moduli are also obtained from backcalculation. Because HMA moduli are highly temperature dependent, values obtained during testing should be corrected to moduli at a standard temperature for thickness design. The Federal Highway Administration's *Temperature Predictions and Adjustment Factors for Asphalt Pavement* (FHWA 2000) provides guidance for making this conversion. The FAA design procedure assumes a standard temperature of 90 ºF.

HMA moduli that are much greater than 800,000 psi (at a standard temperature of 90 ºF) may represent oxidized or brittle pavements. While such pavements may provide adequate structural support, they may be more susceptible to cracking and other environmental deterioration, which eventually weaken the overall structure. Additionally, HMA moduli less than about 100,000 psi (at a standard temperature of 90 ºF) likely indicate structural failure or deterioration of the layer. The standard deviations for the HMA moduli results generally are higher than the other layer calculations; while not ideal, standard deviations of 25 percent or greater for temperature corrected HMA moduli are not uncommon. The following factors can affect the calculated results:

- Variation in pavement cross section (i.e., layer type or thickness).
- Pavement deterioration.
- Varying support conditions.
- Errors in calculating any of the other layer moduli (i.e., the compensating layer effect).
- Variations in temperature-adjustment moduli, which is based on nationwide data.

Variations in pavement layer thickness have the greatest impact on the results, which can cause moduli to be overestimated or underestimated. Therefore, accurate layer information is required for this testing and analysis method to be meaningful.

Additionally, testing is typically not performed in areas that have apparent deterioration; therefore, the backcalculated layer moduli represent areas of pavement without substantial distress (and often pavements in need of rehabilitation have areas with deterioration). If testing is performed in areas of deterioration, the variability in the results generally increases. Additionally, an underlying assumption in backcalculation is that the layers are continuous, so cracking between the sensors violates the premise of the backcalculation analysis. However, the normalized deflection plots can reveal weak areas, even if the test data from these areas cannot be further analyzed.

FWD testing requires input from other evaluation methods. As mentioned, layer thicknesses can significantly impact the analysis of the deflection data. Bonding conditions between multiple layers (or lifts) also influences the results. The presence of a rigid layer (and the depth to the layer), such as bedrock or water table, has an impact on the determined values. Therefore, other evaluation methods (such as coring and boring or GPR) should be combined with FWD testing. Laboratory testing results of materials obtained through destructive sampling can also be useful in interpreting deflection testing results.

Ground Penetrating Radar (GPR)

GPR is an NDT method that uses electromagnetic pulses to map subsurface conditions. These pulses are transmitted from an antenna into the ground. When objects or interfaces with different dielectric constants are encountered, there are noticeable variations in the reflected signal, which is received by the device antenna. Elapsed time and dielectric properties are used to determine layer thicknesses. In this manner, changes in material types, debonding between pavement layers, voids, cracks, excessive moisture, and other abnormalities can often be detected. This testing method is acknowledged in FAA NDT documentation provided in Advisory Circular 150/5370-11A.

GPR testing can be collected at highway speeds (or those of slow-moving aircraft), which reduces the impact on airfield operations. This also allows for large coverage areas to be tested in a limited amount of time.

The accuracy of the data depends on the emitted wave frequency; resolution is improved with higher frequencies, while penetration depth is increased with lower wave frequencies. Also, the accuracy of the results is typically greater near the pavement surface and diminishes with depth. Therefore, there is an accuracy tradeoff that may be required, especially for thick airfield pavements.

As with FWD testing, destructive tests such as coring are beneficial to calibrate and validate the results. While this does not replace the need for coring, it can reduce the number of cores required on a project. GPR works well when collected in conjunction with FWD testing as pavement thickness is a critical input during analysis. The equipment can also be mounted on the same vehicles/equipment as used for FWD testing.

Portable Seismic Pavement Analyzer (PSPA)

A potential option to assess the in-place concrete strength is seismic testing, which, like the FWD, is a nondestructive method. Seismic methods can be used to estimate slab thickness (via impact-echo testing) and the concrete dynamic modulus (via lab and field seismic testing). One device in particular, the portable seismic pavement analyzer (PSPA), was developed specifically for these applications and employs a combination of the equipment/instrumentation used for both impact-echo and ultrasonic testing (see figure 66). Like most seismic equipment, the PSPA is rapid and nondestructive, requires no specimen preparation, and can be easily performed at numerous locations throughout a project. Preliminary calibration and periodic validation of strength-modulus relationships are necessary. Although the PSPA is a relatively new device, it has shown great promise in measuring pavement strength. The FAA recently tested this device at their pavement test track in Atlantic City (Garg and Hayhoe 2007).

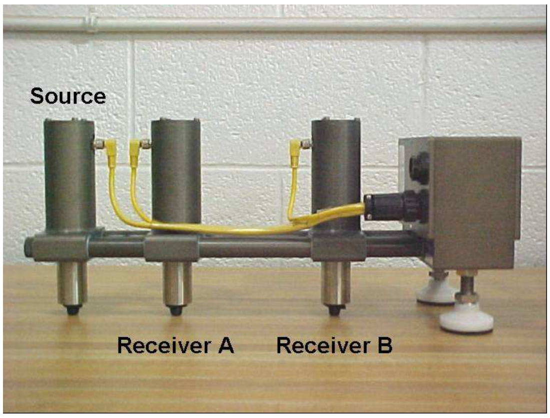



Figure 66. Portable seismic pavement analyzer (left) and its sensor unit (right) (Nazarian et al. 2006).

A project for the Innovative Pavement Research Foundation (IPRF), *Acceptance Criteria Based on Innovative Testing of Concrete Pavements*, evaluated maturity and seismic nondestructive testing technologies as a basis for developing new acceptance criteria for concrete airfield pavement construction (IPRF 2006). The study included extensive experimental work on concrete specimens and slabs of different mixes and cured under different conditions, as well as tests on an actual concrete pavement airfield construction project. The results indicated that concrete strength parameters can be estimated from either seismic modulus, maturity, or both with appropriate calibrations. Moreover, the results also showed that unlike a strength-maturity relationship that is restricted to a specific mix under a particular curing condition, a seismic modulus-based relationship is mainly affected by the nature of the coarse aggregate and, to a much lesser extent, by other parameters such as curing condition, admixtures, and water-cement ratio. Figure 67 illustrates a correlation of the seismic modulus with other strength measurements for a number of mix designs and curing conditions using the same siliceous river gravel (SRG) coarse aggregates.

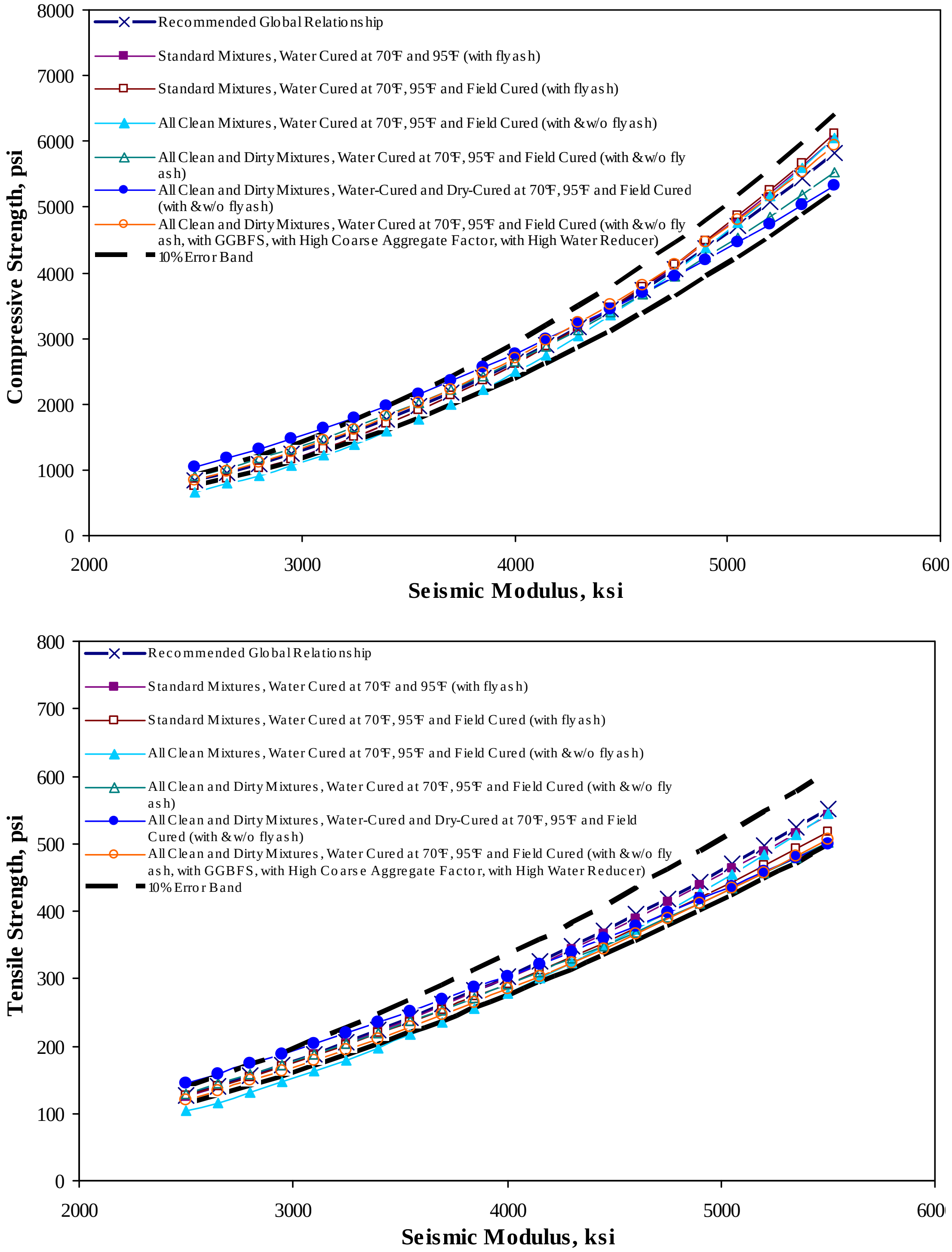


Figure 67. Comparison of relationships developed between strength and seismic modulus for different SRG mixes and different curing regimens (Nazarian et al. 2006).

Using this technology, some limited coring is still required to calibrate field testing results to laboratory results. The results are not impacted by pavement thickness, which is a substantial difference between this testing and FWD testing. The type of coarse aggregate has the biggest impact on the strength measurements obtained using the PSPA. PSPA testing, like many non-destructive options, is often more economical than destructive testing, can cover larger areas, and has a minimal impact on operations. This device offers a promising option for characterizing the existing HMA surface and stabilized base layers for overlay design but is not as effective for characterizing deeper unbound base/subbase layers or subgrade materials.

Other Uses of NDT

The previous discussion of common NDT methods concentrated on the determination of layer thicknesses and moduli, which can be used directly in the HMA overlay design procedure. NDT methods can also be used to evaluate other pavement conditions. Debonding (or delamination) between bound pavement layers can create many problems, including cracking and disintegration of surface layers and moisture-related deterioration (stripping). A recent study performed under AAPTP 06-04, *Evaluation of NDT Technologies to Assess Presence and Extent of Delamination of HMA Airfield Pavements*, assessed the ability of various NDT equipment to determine the presence of delamination (Celaya et al. 2010). The study indicates FWD, impulse response (IR), and USW provide the most promise for detecting delamination, but generally for areas larger than 4 feet (Celaya et al. 2010). Research has also been conducted regarding the use of NDT methods in detecting stripping in HMA pavements (Celaya and Nazarian 2007). The presence of such deterioration can significantly shorten the anticipated design life of an HMA overlay if it is not detected and addressed.

THEORETICAL METHODS

Miner's Hypothesis

Miner's hypothesis assumes that fatigue damage is cumulative and that each aircraft coverage consumes an infinitesimal amount of the total fatigue life of the pavement until the pavement fails structurally. The equation is based on the number of allowable loads to failure (N_{all}) and the number of loads that have been applied (n), as shown in the following equation:

$$Miner's\ Damage = \sum \frac{n}{N_{all}} \qquad \text{(Eq. 30)}$$

A pavement theoretically reaches failure when Miner's damage equals 1.0, or when the number of allowable applications equals the number of applied load applications. Thus, if Miner's damage is equal to or greater than 1.0, the pavement has theoretically consumed its fatigue life and has no remaining life. The coverages-to-failure-concept suggests that once a pavement has carried enough critical loads, either a fatigue crack or a critical deformation (rut) will develop. The failure is not a catastrophic failure, but rather a progressive process in which more and more of the pavement will develop distresses associated with pavement failure.

Miner's fatigue hypothesis is used to estimate the maximum allowable stresses and strains for the required pavement life, which ties in directly with the mechanistic-empirical design.

It should be noted that this methodology does not directly account for wheel loadings that have already taken place. It is possible to take previous loadings into account by including past traffic or using a parameter similar to that of the SCI or CDFU for PCC pavement layers. Traffic data are used in the analysis to estimate the total projected life of the pavement structure with the current layer properties. Previous load repetitions are likely to have already consumed a portion of the existing pavements' expected structural life—backcalculation results account for some, bu not all, of the internal damage (especially because the calculations rely on data from pavement areas in relatively good condition).

Cumulative Damage Factor Used (CDFU)

The FAA's LEDFAA computer program (outlined in Advisory Circular 150/5320-6D) and FAARFIELD program (outlined in Advisory Circular 150/5320-6E) incorporate the full traffic mix when determining remaining life and pavement design requirements. A cumulative damage factor (CDF) is used to determine the effect of each aircraft, accounting for factors such as gear location (distance from center of aircraft) and aircraft wander. The design is run until the sum of the CDF of each aircraft is equal to 1.0, as with Miner's hypothesis.

As discussed in previous chapters, CDFU can be entered into FAARFIELD for the design of HMA overlays over composite (and bare PCC) pavements to account for past traffic applications when the PCC layer has exhibited an "initial crack." This option is not available for the design of HMA overlays over flexible pavements. While backcalculated modulus for an HMA layer may take some of this into account, the same could be said of PCC backcalculated values in which the CDFU is still applicable. The CDFU concept could be an option for assessing rutting (in the surface) or fatigue cracking on existing HMA pavements.

Remaining Life

Remaining life is defined as the amount of life (expressed in terms of time or traffic loadings) left in an existing pavement before that pavement reaches failure. Pavement failure can be defined in any number of ways, such as reaching a critical roughness level (functional remaining life), reaching a critical PCI, or developing a critical level of a key structural distress (e.g., 50 percent of the wheel path with fatigue cracking). Whatever approach is used, the estimation of remaining pavement life is a useful component of a pavement evaluation. In a project-level evaluation, remaining life estimates provide insight into the structural capacity of the existing pavement and thereby indicate the need for an HMA overlay. Remaining life can also be used within a pavement management system. At the network level, remaining life predictions of families of pavements permits planning, scheduling, and budgeting of future rehabilitation activities, thereby contributing to the more effective use of limited resources.

Pavement remaining life can either be expressed as structural remaining life or remaining life before reaching a critical PCI. Structural remaining life considers the load-carrying capacity of the pavement, whereas the PCI remaining life evaluates the surface distress of the pavement and its ability to serve its intended function. In the former, a structural HMA overlay would need to be designed, while in the latter, a non-structural HMA overlay would be used to restore functionality.

Using FAARFIELD, the entire traffic mix is entered, along with the existing pavement structure (entering the layer properties determined by evaluation). The remaining life is calculated similarly to the thickness design except that the layer thicknesses are not changed, but rather the traffic volume (or years) is modified until a CDF of 1.0 is achieved. However, this has the same limitation as for HMA overlay design, in which past traffic is not directly being taken into account.

DISCUSSION AND SUMMARY

There are many traditional and emerging evaluation techniques that can be used to evaluate existing pavements, each with their advantages and disadvantages. Visual inspections are relatively rapid, easy to conduct, and inexpensive, but limit the data collection to distresses observed at the pavement surface. Coring and boring provide subsurface information but often disrupt airfield operations, are more expensive, and are typically only performed at limited locations. Non-destructive testing can provide large quantities of data locations but often need to be supplemented with some destructive testing, and the results are generally more variable.

Within the FAA's HMA overlay design procedure, visual condition data can be used to establish SCI for composite pavements (representing the underlying PCC condition). The assessment of SCI for composite pavements is made more difficult because of the overlying HMA, but a general assessment of the visible cracking, historical performance data, and data from other testing methods (such as deflection testing) can be used to establish appropriate SCIs. Sensitivity analyses indicate the HMA overlay requirement determined using FAARFIELD is only mildly influenced by SCI, for the conditions studied. Therefore, estimation of an SCI for a composite pavement in such a manner is likely acceptable in the majority of cases.

Visual condition data are not currently part of the HMA overlay design of a flexible pavement. Establishing an SCI for an HMA pavement is not as clearly defined as it is for PCC layers, and the continued deterioration of the underlying HMA pavement is not modeled using SCI as it is for the underlying PCC layers in a composite cross section. However, condition data can be used in several relevant ways, including:

- As a means of establishing needed pre-overlay repair methods and quantities. In addition to traditional PCI data, distress mapping can greatly help with planning repairs.
- Establishing a set of conditions where an HMA overlay is no longer a suitable rehabilitation option and should not be expected to perform adequately.
- Modification of results from evaluation testing. For example, deflection testing and backcalculation results can be adjusted based on overall condition, such as lowering the backcalculated layer moduli for deteriorated areas (note that deflection testing is typically conducted at intact locations and backcalculation assumes layer continuity). Laboratory testing would also generally be conducted on intact cores, which would not necessarily characterize the overall condition of the pavement.

In addition to visual inspections, commonly used evaluation techniques for assessing layer properties include destructive and non-destructive methods. Visual inspection needs to be supplemented with testing techniques to determine layer properties and subsurface deterioration. There is not necessarily a single, best alternative, but rather the best approach is often to use several methods in combination. The goals of a detailed testing evaluation should include:

- Estimating subgrade support. After aircraft weight, subgrade support is the most influential input in flexible pavement design based on the sensitivities conducted in Chapter 3. A 5-percent difference in subgrade support can result in nearly a 1-inch difference in overlay requirement.
- Determining pavement layer thicknesses. Pavement layer thickness has a significant influence in analyzing deflection testing (one of the more common NDT methods) and is relatively influential in the overall HMA overlay design. As shown by the sensitivity analyses (Chapter 3), a 5 percent change in thickness is equivalent to nearly a 20 percent change in HMA modulus, depending on the basis. An emphasis should be placed on accurately determining existing layer thicknesses during the evaluation.
- Establishing pavement layer characteristics. Existing HMA modulus and granular material moduli are less influential (based on the sensitivities presented in Chapter 3) but still result in noticeable changes in required HMA overlay thicknesses, particularly when moduli differ by more than 10 or 20 percent from default layer properties (for HMA and granular layers, respectively).

Destructive and NDT techniques are also used to identify areas where HMA overlays are not suitable or where repairs are required. Examples discussed in this chapter illustrate the use of deflection testing identifying sections of pavement with varying load response, which likely require different HMA overlay requirements. Another example illustrated the use of coring in the identification of stripping at subsurface boundaries, which requires repair prior to placing an HMA overlay (or possibly accounting for the underlying layer(s) as aggregate layers for design, even if they are stabilized).

Used independently, visual, destructive, and nondestructive evaluation methods may not provide a complete picture of the existing pavement condition, and, therefore, what design inputs should be selected for HMA overlay design. There is no single “do-everything” technique, and each HMA overlay design project needs to assess what techniques are best suited for that particular project. A hierarchy of evaluation methods may often be needed. For example, a project establishes visual, coring, and FWD testing as the initial evaluation methods. Coring identifies stripping in lower HMA layers, but the extents are inconclusive from the visual survey and FWD testing. Therefore, additional NDT (such as GPR or PSPA) are incorporated to identify the limits even though it was not initially planned. While each technique provides useful data, complementary methods assist with interpretation of the collected data and selection of existing pavement properties for HMA overlay design inputs. HMA design models do not generally account for past damage, so an accurate characterization of the pavement condition is even more important.

CHAPTER 5. IDENTIFICATION AND ASSESSMENT OF CORRECTIVE AND MITIGATIVE ACTIONS

INTRODUCTION

The performance of an HMA overlay is influenced in large part by the condition of the existing pavement or, more specifically, by the distresses exhibited on the existing pavement. The more the underlying pavement is distressed (particularly the more structural distress) and/or the greater the severity of the distress, the shorter the service life of the overlay. Conversely, the better the condition of the existing pavement, the longer the life of the overlay that can be expected, other things being equal.

Besides the expensive approach of increasing the thickness of the HMA overlay to compensate for deterioration in the existing pavement, there are two ways to address the adverse effects of existing pavement distress. One way is to improve the existing conditions prior to overlay by performing specific corrective actions, such as patching localized distressed areas (e.g., fatigue-cracked areas and deteriorated patches) or removing a delaminated or unstable (rutted and/or shoved) surface layer. Another way is to incorporate as part of the overlay one or more mitigative treatments that will prevent or slow the progression of underlying deterioration (e.g., cracking, material disintegration, and so on) through the overlay.

As discussed previously, the FAARFIELD overlay design procedure does not directly consider the use of corrective or mitigative actions in determining the required thickness of the overlay. The procedure entails defining the in-place thickness and modulus of individual material layers and calculating the required HMA overlay thickness to control the development of subgrade rutting in the pavement. The effects of corrective and/or mitigative actions can only be examined in FAARFIELD in terms of their influence on subgrade vertical strain, and not on the tensile strains in the HMA layers (although the overlay layer can be evaluated, it seldom will control the design) produced by loading and the tensile strains in the HMA overlay generated by movement in underlying cracks.

This chapter describes the various forms of corrective and mitigative activities that can be performed prior to or as part of the HMA overlay placed on an existing HMA-surfaced pavement. It discusses the expected impact of each activity on the performance of the overlay and provides an assessment of how the technique should be accounted for in the overlay design process. It should be noted that much of the performance information presented relates to highway applications, as this is where the bulk of the research has been concentrated; airport studies are referenced when available.

CORRECTIVE AND MITIGATIVE ACTION TECHNIQUES

To address existing pavement deficiencies, corrective and/or mitigative actions are often taken prior to overlay to help ensure the desired HMA overlay performance. These actions can be considered in several broad categories:

- Localized Maintenance and Repair.
  - Crack sealing.
  - Partial-depth patching.
  - Full-depth patching.
- Modification of Existing HMA Pavement.
  - Cold milling.
  - Surface recycling.
  - Surface leveling course.
- Stress/Strain Relieving Interlayers.
  - Stress-absorbing membrane interlayer (SAMI).
  - Proprietary stress-absorbing interlayers.
- HMA Overlay Reinforcement.
  - Steel or wire fabric.
  - Geogrids.
  - Geosynthetic fabric.
- Crack Arresting or Cushion Layers.
  - Aggregate.
  - Asphalt-aggregate.
- Bond Breakers.
- HMA Overlay Mixture Modification.
  - Rubber-modified asphalt binder.
  - Polymer-modified asphalt binder.
  - Soft (low-viscosity) asphalt binder.

It should be noted that some of these techniques have shown to be generally unsuccessful or may not be permitted for use on FAA-sponsored projects, and therefore are not discussed further in this chapter. For example, the use of bond breakers, which consist of stone dust, wax paper, roofing paper, or aluminum foil placed adjacent to pavement joints/cracks prior to overlay to prevent reflective cracking (by reducing stress concentration at the joints/cracks), has been found to provide slight to no benefit in terms of delaying the occurrence of reflective cracks (Von Quintus et al. 2009a). Additionally, a thin, unbonded layer will develop greater strains under loadings and will fail in fatigue. Also, although aggregate cushion layers are open and permeable (they use large-sized aggregate with low fines), they may be characterized as a "sandwich" layer (i.e., overlaid pavements containing granular separation course between the existing and planned impervious surface), which the FAA does not permit due to reduced stability of the overlay if the separation course is not adequately drained (FAA 2009). Lastly, the use of softer grade asphalt in the HMA overlay mix can effectively reduce stresses at the tip of underlying cracks; however, it is not generally recommended due to the potential for bleeding and rutting.

It should also be noted that many of these techniques can be, and often are, used in conjunction with other techniques. For instance, cold milling can be combined with crack sealing prior to the application of a modified overlay. Likewise, a surface leveling course can be applied, followed by a stress/strain relieving interlayer and modified HMA overlay.

The following sections discuss in detail the various techniques. Each section provides a description of the technique and its function or purpose and discusses the effect of the technique on HMA overlay performance.

Localized Maintenance and Repair

Localized maintenance and repair consists of small areas of patching and sealing, including the following: sealing of existing non-fatigue-related cracks to retard reflective cracking and to prevent water intrusion into the pavement base, and partial- and/or full-depth patching of small areas of deteriorated pavement to provide a sound foundation for the overlay. This work should be performed to address deterioration of the existing pavement and to provide uniform support for the HMA overlay. Specific discussions of each technique are provided below.

Crack Sealing

Crack sealing is often considered to be a preventive maintenance treatment involving thorough crack cleaning and preparation and placement of high-quality sealant materials into and/or over working cracks that are not severely deteriorated. Its main objective is to reduce the amount of moisture that can infiltrate a pavement structure, thereby reducing moisture-related distresses (e.g., stripping, pumping of fines, and fatigue cracking) and extending the life of the pavement (Grogg et al. 2001).

As a pre-overlay treatment, crack sealing has this same objective, but also the objective of retarding the reflective of cracks into the HMA overlay. The retarding effect is, in theory, a result of the reduction in horizontal stresses and strains that occur at the interface of the crack tip and the overlay as a result of thermal contractions and expansions and vertical deflections produced by loading. The reduced stresses and strains delay, and in some cases prevent, the onset of cracking that eventually propagates to the surface of the HMA overlay, thereby extending the life of the overlay. Crack sealing beneath an overlay provides an additional benefit, even once the crack has reflected through, in that it helps to slow down or keep water from infiltrating into subsurface layers. In addition, a properly performing seal potentially reduces the amount of water penetrating into the pavement system, thereby keeping the pavement dryer and at a higher strength for a longer period of time. This has the effect of a reduction in the development of load-related distresses, such as fatigue cracking, potholing, and rutting.

To work effectively, the crack seal treatment must be performed properly. In addition to following good practice in the placement of crack sealant, additional guidance for sealing beneath an overlay should be considered. Crack sealing is typically applied to cracks that are 1/8 to 3/4 inch wide. To avoid bleeding through the overlay and the creation of a hump, the sealant should be installed at least 6 to 12 months prior to the overlay or a sand blotter be placed atop the sealant prior to overlay. Oxidized sealant or sealant with a blotter will help minimize the potential for bleeding of the sealant into the overlay. Also, overband sealant configurations should not be used, especially with thin overlays (2 inches or less), as the overband application usually results in a bump in the overlay. This occurs because as the HMA overlay is compacted it is slightly pushed or crawls under the weight of the roller, while the stickiness of the crack seal interrupts the crawling effect causing a bump in the overlay (FPO 2000). To further limit the sealant bleeding through, it is recommended that the material be placed slightly recessed (1/8 to 1/4 inch) in a routed crack reservoir, which also provides space for the blotter material. Lastly, a somewhat stiffer crack sealant is sometimes recommended to reduce the deflection of the overlay at the crack under traffic loading.

Section 407.a.4 of Advisory Circular 150/5320-6E indicates that cracks 3/8-inch wide or greater should be filled with sand-asphalt material prior to overlay. Under this approach, when the cracks open, the material can sink down into crack, causing reduced support and localized loss of bond from loss of material. This guidance in the Advisory Circular should be reconsidered.

Little research has been done to quantify the specific effects of sealing prior to overlay. The FHWA’s Long-Term Pavement Performance (LTPP) Specific Pavement Studies (SPS-5) program examined the effects of “minimum” (patching and rut-filling) and “intensive” (patching, milling [1.5 to 2 inches], and crack sealing) pre-overlay repair techniques on the performance of 2- and 5-inch thick HMA overlays. The study found somewhat better performance (although statistically insignificant)—based on rutting and fatigue—for HMA overlays using “intensive” pre-overlay repair techniques (Hall et al. 2002). Although the results suggest possible benefits associated with crack sealing, it is believed the benefits were derived more from milling. In their report on techniques for mitigation of reflective cracking, Von Quintus et al. (2009a), suggest there is little evidence to show increased performance (reduced reflective cracking) with or without crack sealing.

Partial- and Full-Depth Patching

Pre-overlay repairs, in the form of partial- and/or full-depth patching, serve the dual purpose of removing or minimizing pavement distress and providing uniform support for the HMA overlay. Patching repairs in existing asphalt-surfaced pavements generally involve the use of HMA materials, although they could entail the use of PCC materials in an existing composite pavement (i.e., HMA overlay on PCC).

Patching repairs may either be partial-depth, used when the existing deterioration is confined to the surface layer (e.g., longitudinal cold-joint cracking) and thus removal and replacement involves all or a portion of the surface layer, or full-depth, used when the deterioration extends into subsurface layers (such as when there is fatigue cracking or a soft foundation) and removal and replacement captures the full-depth of the weakened structure. Because of the importance of producing high-quality patches, the perimeter and bottom of the repair must extend into sound portions of the pavement. Also, as described in Advisory Circular 150/5370-10E (Item P-101), partial-depth patches should be accomplished using a cold milling device, whereas full-depth patches should be accomplished via milling or sawcutting to the full depth of the HMA pavement.

The determination of which type of repair is appropriate will depend on the distress and the cause. While a condition survey will provide a general indication of the appropriate repair, the results of structural/materials testing can be more definitive. For instance, full-depth patching may be anticipated for fatigue cracking, but a coring investigation may show that the fatigue is associated with a delaminated surface layer, requiring only partial-depth patching.

As mentioned earlier, if localized deterioration is not corrected prior to overlay, thicker HMA overlays should be anticipated. However, some balance between the amount of distresses repaired (such as by severity level) and the thickness of the HMA overlay must be achieved. That balance is generally driven by the relative costs of patching and overlay (i.e., more pre-overlay repairs will result in a thinner overlay), as illustrated in figure 68 (Grogg et al. 2001).

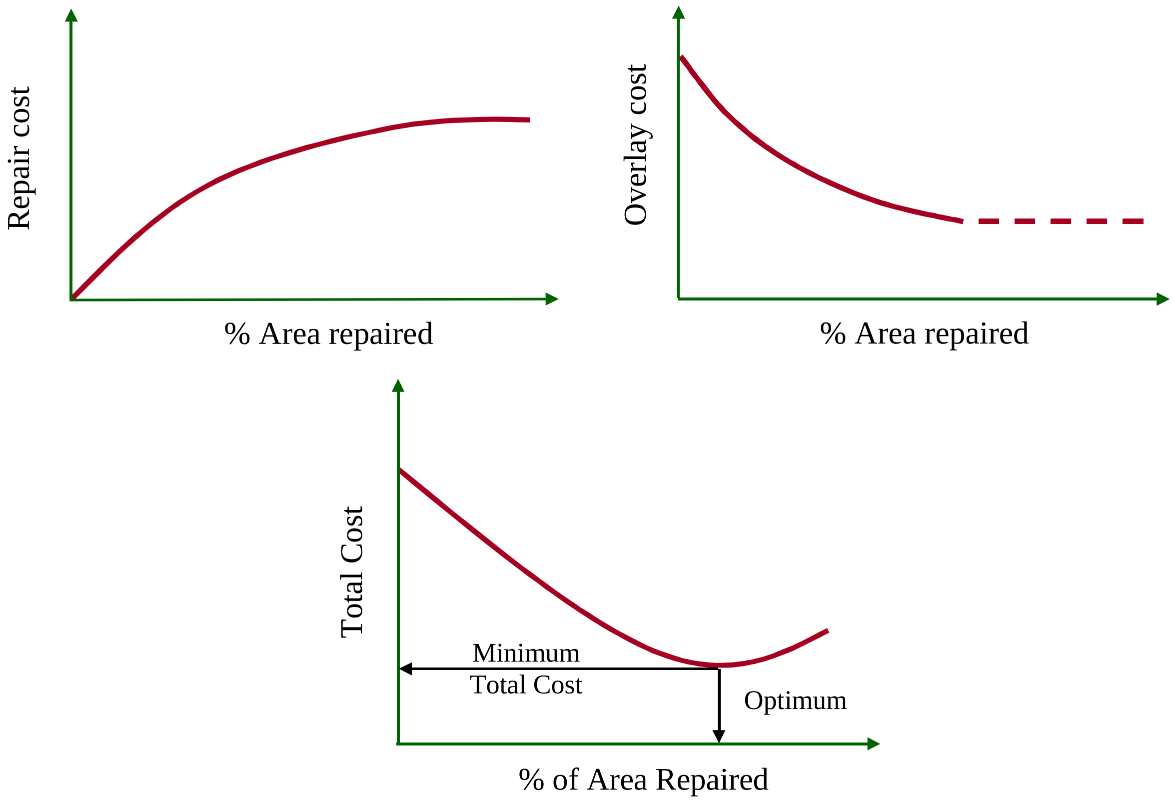


Figure 68. Patching, overlay, and total cost versus percent of area patched (Grogg et al. 2001).

Although the LTPP SPS-5 study did not compare the performance of overlays with pre-overlay patching versus those without patching, their effect on the performance of the HMA overlay can be significant. By repairing weak, deteriorated areas to their full limits, much greater support is provided for the overlay, resulting in better compaction during construction and greater endurance under loading. The ultimate effect is a reduction in the development of load-related distresses, such as fatigue cracking, potholing, and depressions, and an increase in the life of the overlay.

Modification of Existing HMA Pavement

This treatment strategy involves modifying the structural and/or surface profile properties of the existing HMA surface layer. It consists of three specific treatments—cold milling, hot in-place recycling (HIR) of the surface, and application of a leveling course—capable of eliminating or correcting surface distresses and/or lessening their influence on the performance of the HMA overlay. Such distresses typically include top-down cracks, bleeding, raveling, and surface irregularities (e.g., permanent deformation, shoving, and settlement).

Although more extensive treatments, such as mill-and-replace and full-depth recycling, could be included in this strategy, their costs would be comparable to the costs associated with increased HMA overlay thicknesses. Thus, they are not discussed here.

Cold Milling

Cold milling is the controlled removal of an existing HMA pavement to a desired depth, longitudinal profile, and cross-slope using a cold milling machine. The milling machine's large rotary cutting drum is equipped with many tungsten carbide cutting teeth that remove the existing HMA upon contact. The removed material, referred to as reclaimed asphalt pavement (RAP), can be reused as aggregate for base/subbase or as aggregate in recycled HMA mixes.

As a pre-overlay treatment, cold milling can serve three important functions. First, it can be used to remove surface distresses, such as top-down fatigue cracking, longitudinal cold-joint cracking, and unstable or highly oxidized surface mix. Second, it can be used to restore the transverse profile of a pavement with stable rutting (i.e., rutting due primarily to consolidation of one or more pavement layers, not instability in the layers), thereby enabling the placement of an HMA overlay with greater uniformity in thickness. Lastly, the milled surface provides a roughened and textured surface, which enhances the bond between the existing pavement and the HMA overlay. Another benefit of milling is potential of matching existing grades and not having to adjust lighting structures (depending on milling depth and required HMA overlay thickness).

As with partial- and full-depth repairs, an assessment must be made as to the optimum depth of milling. Although more surface distress may be able to be removed with increased depth, there is a point at which the overall structural capacity of the pavement is diminished enough such that the overlay thickness must be increased. Thus, not only is an increase in the cost of milling experienced (with increased milling depth), but also an increase in overlay cost. A combination treatment of cold milling and partial- and/or full-depth repairs requires an additional level of optimization to determine the most cost-effective milling depth and patching amounts for the HMA overlay. Visible, open cracks following milling should be sealed prior to overlay, as previously described.

The removal of delaminated surfaces via cold milling safeguards against premature failure of the HMA overlay. The combination of condition surveys and destructive and non-destructive testing will largely identify debonding issues and the FAARFIELD design procedure assumes that such issues are properly rectified.

As mentioned previously, LTPP SPS-5 study results found somewhat better (although statistically insignificant) performance in terms of rutting and fatigue cracking for HMA overlays using the intensive pre-overlay repair technique versus those using the minimum technique (Hall et al. 2002). The performance benefits are believed to be mostly the result of milling, although crack sealing may have been a contributing factor. An estimate of the impact of milling on HMA overlay performance was not provided in that study (Hall et al., 2002).

A 2008 study by the Ohio Department of Transportation examined the impact of milling on the Structural Number used by the Department in designing overlays of HMA and composite pavements (Mallela et al. 2008). The study evaluated the structural properties of 17 pavements (8 with HMA over HMA and 9 with HMA over a composite structure) obtained through FWD testing at three different stages: (1) prior to milling, (2) after milling, and (3) after HMA overlay placement. The condition of the pavements ranged from fair to poor. Results indicated that the effective structural number (SN_{eff}) for existing HMA pavements prior to milling was 17.4 percent higher than SN_{eff} following milling. Similarly, the effective structural thickness (D_{eff}) for existing composite pavements prior to milling was 13.1 percent higher than D_{eff} following milling. The differences were mainly attributed to the condition of existing pavement prior to rehabilitation and to the ratio of the milling thickness when compared to the original thickness. Greater differences were observed among the pavements in poorer condition and those with a higher ratio of milling depth to total thickness of existing HMA. The effect of lower structural number after milling indicates the need for a thicker overlay to compensate for reduced structural capacity.

Taken together, these studies suggest that the impact of milling on HMA overlay performance can vary from slightly negative (thicker overlay needed) to slightly positive (some extension in life). The specific effects depend on the makeup of the existing structure and the types, severities, and amounts (and locations within the structure) of distresses present. A recent survey of state practices summarized in *NCHRP Synthesis 388* (Tenison and Hanson, 2009) reports that about half of the states mill HMA surfaces prior to placing an HMA overlay to address shoving, rutting, and fatigue cracking issues. Based on its frequent use, the perceived benefits of milling appear to be significant. For the purposes of this report, 0 to 1 year of additional life is estimated, with the higher value representing a pavement in relatively good condition and with a lower ratio of milling depth to the existing HMA thickness, which would be the case for thicker airport pavements.

Hot In-Place Recycling (HIR) of the Surface

HIR surface recycling is one of three different forms of hot in-place recycling (the other forms include remixing and repaving) (Dunn and Cross 2001). Previously known as heater-scarification and heater-planing, it is the oldest HIR process, having been around since the 1930s and becoming popular in the 1960s.

The HIR process consists of heating, scarifying (using teeth or a rotary mill), mixing, leveling/re-profiling, and compacting the existing HMA mixture. Once heated, a recycling agent (i.e., rejuvenator) is applied and mixed in to rejuvenate the existing HMA, making it stronger and more pliable. The treatment depth is typically between 1 and 2 inches. Because no virgin aggregate or new HMA are added, the overall pavement thickness remains essentially the same (Dunn and Cross 2001).

As a pre-overlay treatment, HIR surface recycling can be particularly effective in addressing surface distresses, such as top-down cracking and bleeding, and restoring the longitudinal and transverse profile. The surface recycling process does not eliminate cracks that extend deeper than its application depth, but it can reduce the stress concentrations at the tip of the crack and usually fills the portion of the crack that remains below the recycling depth. The recycled surface also promotes bond with the HMA overlay.

Although HIR surface recycling is generally performed as a stand-alone treatment on pavements that primarily exhibit surface distress, it can and should be supplemented with partial- and/or full-depth patching to correct deeper deficiencies at isolated locations. These repairs are typically performed prior to the surface recycling.

Von Quintus et al. (2009a) reported that HIR surface recycling projects have shown mixed results, varying from none to good improvement in mitigating reflective cracks, as compared to control sections with no HIR surface recycling prior to HMA overlay. Like other forms of pre-overlay repair, the effects are governed by the types, severities, and amounts of distresses and the nature and extent of the recycling activity. A general estimate of the performance benefit provided by this activity is 0 to 2 years, with the higher value representing a pavement with key distresses largely confined to the pavement surface.

Surface Leveling Course

A leveling course is a lift of HMA placed directly on the existing pavement to fill low spots in the pavement, such as wheelpath ruts, depressions, and corrugations. Although application of a leveling course does not eliminate or correct existing surface distress, it does provide a short-term fix and a means of delaying the progression of distress into the HMA overlay.

Leveling courses are typically placed using pavers with an automatic screed control. The automatic screed control facilitates the placement of the HMA in a relatively smooth lift, one that is independent of the paver’s movements, which are tied to the basic contour of the existing surface. Although the smoothness of a leveling course is generally not tightly specified or controlled, it should be closely monitored to ensure that the HMA overlay can be placed to the required smoothness and that the overlay thickness is fairly uniform.

Leveling course lifts should be as thick as the deepest low spot, but not so thick that they are difficult to compact. This is particularly true in cases where the permanent deformation and/or deflection under loading are shown to be quite high. The inability to adequately compact the leveling course will not only result in insufficient support for constructing the HMA overlay, but also insufficient support for the traffic moving across the overlay.

In their report on techniques for mitigation of reflective cracking, Von Quintus et al. (2009a) stated there is general agreement within industry regarding the benefit of using leveling courses with appropriate HMA mixture properties. The extent of the benefit is not given, and would certainly depend on the specific structure and existing conditions of the pavement and the nature and extent (thickness) of the leveling course. A general estimate of the additional HMA overlay life provided by this activity is 0 to 2 years, with the higher value representing a thicker leveling course placed on a pavement with greater structural integrity.

Stress/Strain Relieving Interlayers

One strategy of reflective crack mitigation is to place a layer between the existing pavement and the HMA overlay that is capable of dissipating stresses and strains before they reach the overlay. Such interlayer systems are designed to deform horizontally or vertically and absorb much of the energy at cracks that is created by environmental and wheel loads.

The interlayer treatments that comprise this strategy are typically 1 inch thick or less and do not, or at least are not intended to, offer any additional structural capacity to the pavement. Moreover, while they are generally useful for reducing reflective cracking due to horizontal movement, they may be less effective at reducing reflective cracking due to vertical deformation.

Specific interlayer treatments include stress absorbing membrane interlayers (SAMI), the proprietary interlayer stress absorbing composite (ISAC) system, and the proprietary Strata® reflective crack relief system. Discussions of each are provided in the sections below.

SAMI

A SAMI is a layer of thick (typically 1/4 to 3/8 inch) asphalt binder (usually modified with rubber or other elastomers) and single-sized aggregate chips (3/8 to 1/2 inch), which are spread and rolled into the binder much like a chip seal (see figure 69). The SAMI layer is flexible and thus capable of absorbing a large amount of crack movement without initiating a crack in the HMA overlay. It is also serves as a waterproofing layer, reducing the potential for stripping within cracks and protecting the pavement structure and foundation from the weakening effects of moisture. However, Von Quintus et al. (2009a) note that SAMIs have minimal ability to distribute vertical shear stresses across cracks produced by traffic loadings.

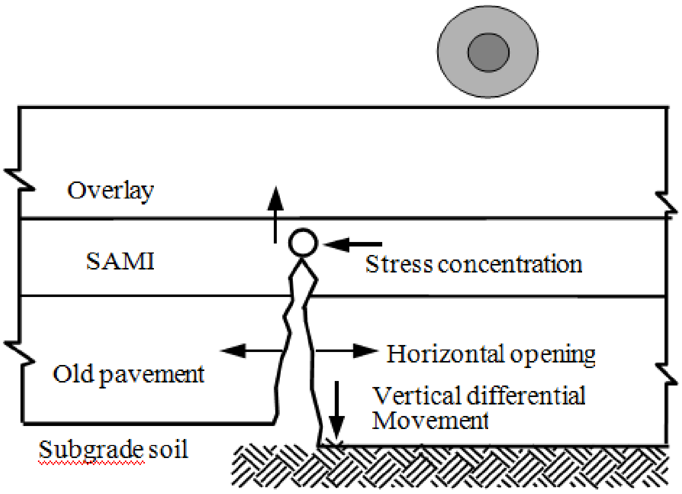


Figure 69. Placement of SAMI layer (Grogg et al. 2001).

SAMI layers are most suitable for use on pavements with moderate to extensive amounts of surface cracking and other surface distresses, but with a structurally sound base and adequate drainage. SAMIs can be applied globally over the entire existing pavement surface or locally at individual cracks. Localized application can greatly minimize material and installation costs (particularly if the cracks are distantly spaced), although some studies suggest that global application produces better results than localized applications (Von Quintus et al. 2009a).

Various reports address the performance of stress-strain relieving interlayers and HMA reinforcing techniques. Cleveland et al. (2002), for example, referenced past highway studies involving the use of SAMIs, including one in which a delay in reflective cracking of 3 to 5 years was observed and another in which bleeding and rutting issues were encountered with asphalt rubber SAMIs. Hajj et al. (2007) cited a study in which researchers found that the life of an HMA overlay may be extended by up to 200 percent as a result of using a SAMI. Subsequent evaluation of five Nevada pavements with stress relief layers indicated no distress for three pavements after 8, 4, and 2 years, and the beginning development of reflective cracking in the other two pavements after 5 years (Hajj et al. 2007). Von Quintus et al. (2009a) generalized that the performance of SAMIs in mitigating reflective cracks has been good, when used under the right conditions. Hanson et al. (2009) reported that SAMIs enable thinner HMA overlay lifts and retard reflective cracking, extending the serviceable life of the overlay.

Taken collectively, these findings suggest that SAMIs provide a range of life extension for HMA overlays by mitigating reflective cracking. The range is believed to be between 1 and 3 years, depending on the nature of the cracks addressed, the type of SAMI used, and the climatic conditions associated with the project.

ISAC

ISAC is a relatively new interlayer technique developed at the University of Illinois and marketed by PaveTech®. As illustrated in figure 70, the system consists of a low-modulus, low-stiffness geotextile as the bottom layer, a viscoelastic membrane layer (asphalt) as the core, and a very high stiffness geotextile for the upper layer (Von Quintus et al. 2009a). The non-woven geotextile at the bottom serves to (a) contain the rubber asphalt membrane, (b) fully bond with the existing pavement with the help of a tack coat, and (c) accommodate large strain at the crack so as to allow horizontal movement of the underlying pavement without breaking its bond.

Because ISAC can relieve stress at the crack tip and at the same time provide reinforcement to the HMA overlay, it is able to constrain the upward propagation of a crack and dissipate the stress at the tip of the crack. This delays the formation of reflective cracks and the subsequent deterioration of these cracks.

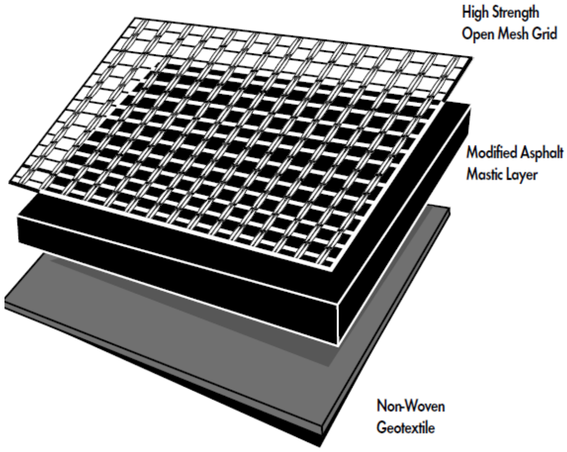


Figure 70. ISAC system (PaveTech 2003).

Like SAMI layers, ISAC can be placed globally or locally. However, its high cost makes localized application the preferred approach. Pavements most suitable for treatment with ISAC include those with low to moderate amounts of transverse or longitudinal cracking that can easily be bridged with strips of ISAC. Typical installation of ISAC is directly on the existing pavement, followed by the HMA overlay (2-inch minimum thickness). Partial- and full-depth patching of isolated deterioration can and should accompany the ISAC treatment, and the joints of such patches can be treated with ISAC.

Hajj et al. (2007) reported on two highway pavement studies that included ISAC. In one study, the system was found to delay reflective cracking by between 1 and 3 years compared to untreated and other crack control methods. In another, the system had to be removed shortly after HMA overlay placement due to problems with bump creation in the overlay. Von Quintus et al. (2009a) reported on the same study that showed good performance from ISAC and noted that limited performance data are available for this technique.

Assuming proper installation and use, it is believed that at least 1 to 2 years of additional HMA overlay life can be expected with ISAC, from the standpoint of reflective cracking.

Strata®

Strata® is a relatively new reflective crack relief interlayer system that claims to delay reflective cracking and to protect existing pavement structures from water damage (Von Quintus et al. 2009a). Developed by SemMaterials and marketed by Road Science, LLC, the system consists of an interlayer of a highly elastic polymer-modified asphalt binder and fine aggregate mixture that is capable of absorbing and redistributing stresses and strains internally.

The interlayer is placed using conventional hot-mix equipment to a thickness of approximately 1 inch and typically has a higher asphalt content. The interlayer can be opened to traffic for a few days prior to placement of an overlay, which aids in construction sequencing.

The manufacturer-recommended HMA overlay is Superpave or SMA using SBS polymer-modified asphalt binder.

Like SAMI layers, Strata® is most suitable for use on pavements with moderate to extensive amounts of surface cracking and other surface distresses, but with a structurally sound base and adequate drainage. The treatment is generally applied globally, either directly on the existing pavement or on a surface leveling course, if used. Partial- and full-depth patching of isolated deterioration can and should accompany the Strata® treatment.

Limited performance data are available for this technique. Hajj et al. (2007) cited a study in which Strata® has performed well over 2 years in retarding reflective cracks in HMA overlays of jointed PCC highway pavements. Von Quintus et al. (2009a) referenced an Iowa study in which Strata® showed reduced amounts of reflective cracking after 4 years in HMA overlays of highway PCC pavements. Hanson et al. (2009) also reported limited usage of Strata® but noted a recent installation at the Rock County Airport in Janesville, Wisconsin.

Based on the limited data available, it is believed that STRATA® is capable of providing at least 1 to 2 years of additional HMA overlay life from the standpoint of reflective cracking.

HMA Overlay Reinforcement

A second strategy of preventing reflective cracking is to reinforce the HMA overlay, thereby increasing its tensile strength and providing greater resistance to the propagation of cracks and other discontinuities in the underlying pavement. Three commonly used techniques for overlay reinforcement are steel reinforcing elements, geotextile fabrics, and geogrids. The latter two are part of a broad spectrum of geosynthetics, which also include composites (fabrics laminated onto a grid) and membranes (polypropylene or polyester mesh laminated on one or both sides of an impermeable rubber-asphalt membrane) (Von Quintus et al. 2009a). Because composites are considered a new generation product and membranes have had limited use, they are not discussed here.

Steel Reinforcement

Steel reinforcement is one of the oldest reinforcement systems used in HMA, dating back to the early 1950s. Both welded wire mesh and expanded metal reinforcement were used up through the 1970s, placed either in narrow strips over joints and cracks or continuously over the entire length of the HMA overlay. Although some success was observed in terms of delaying reflective cracks and holding cracks tightly together, difficulties with installation (e.g., elimination of folds, straightening of curled mesh as a result of being transported in rolls) and corrosion of the steel by water in the HMA brought a halt to its use.

Modifications to the original welded wire mesh in the form of a coated and woven mesh took place in the 1980s, leading to testing and evaluations of the next generation material in the late 1990s and early 2000s. The new products (e.g., PaveTrac™, Bitufor®, Road Mesh®) typically consist of twisted, hexagonal mesh with variable dimensions, and zinc-coated steel wires (circular or torsional flat-shaped) placed transversely at regular intervals (Von Quintus et al. 2009a). After placement on the existing pavement, the mesh is usually flattened using rollers and secured into position with nails and hook bolts. A slurry seal can be applied to facilitate placement of the HMA overlay by keeping the mesh from moving. The slurry seal also improves bonding between the reinforcing mesh and existing pavement and helps waterproof the pavement structure.

Von Quintus et al. (2009a), referencing studies of steel reinforcement netting in HMA overlays at the Virginia Smart Road and in Pennsylvania, noted good performance in terms of reducing reflective cracks. Also cited in this report was a finite element analysis study suggesting steel reinforcement application can improve overlay service life by a factor ranging from 50 to 90 percent, depending on the overlay thickness and the pavement structural capacity.

Although performance data for next generation steel reinforcing techniques are currently limited, an estimate of 1 to 2 years minimum of additional HMA overlay life would seem reasonable, based on crack reflection control.

Geotextile Fabrics

Geotextile fabrics are woven or non-woven fabrics typically composed of polypropylene, polyester, fiberglass, nylon, or various combinations of these materials. Some of the common geotextile fabrics currently being used are Paveprep, Petromat®, Mirafi® MPV and MTK, Bituthene®, Polyguard, Royston, Reepav, Amopave, and Trevira (Grogg et al. 2001; Von Quintus et al. 2009a). Much like steel reinforcement, the purpose of a geotextile fabric is to relieve the stress within the HMA layer by providing physical resistance to the formation or propagation of cracks in the underlying pavement.

Most fabric treatments are placed directly on the pavement, following the application of crack sealant and/or leveling course (if needed) and tack coat to the existing pavement surface (Grogg et al. 2001). Depending on the product, placement of the fabric may be limited to strips placed across cracks or rolls placed over the entire pavement surface. After the fabric is rolled or brushed into the tack coat, a second tack coat is generally applied, followed by the HMA overlay. Laboratory and field testing have indicated that better performance is achieved when the fabric is placed within the HMA overlay, as illustrated in figure 71. Typically, the recommended location is between the initial and final lift in a two-lift overlay or between the initial and second lift in a three-lift overlay. The downside of the fabric being located at the bottom of the overlay is that there is greater potential for inadequate bonding between the existing surface and the HMA overlay and for degradation of the reinforcement due to differential movement across the lower layer.

Early research (~1990) suggested that geotextile fabrics could provide reflective crack control equivalent to approximately 1.2 inches of additional HMA (or a delay of a little over 1 year), but did not prevent it from occurring (Grogg et al. 2001). The AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) includes a reflective cracking model based on an empirical relationship in which a properly installed fabric is equivalent to 2 inches of HMA overlay (ARA 2004). The basic effect of this supposition is that inclusion of geotextile fabric provides 2 years of additional HMA overlay life.

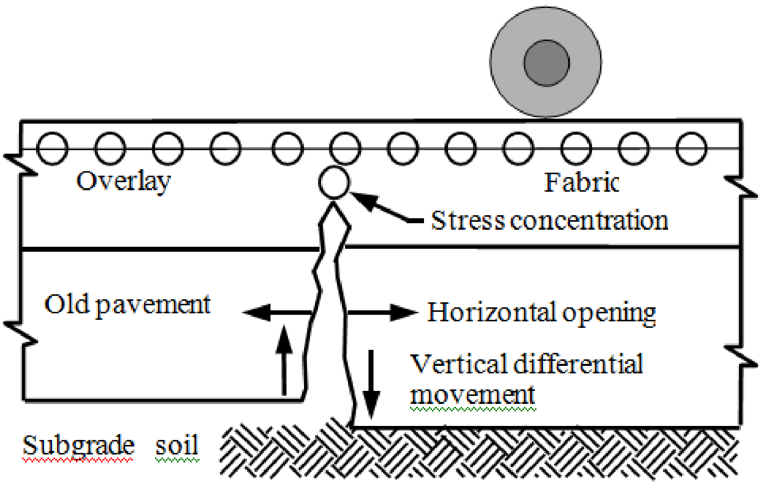


Figure 71. Proper placement of geotextile fabric with the HMA overlay.

Referencing a variety of past highway studies, Hajj et al. (2007) indicated varied performance with paving fabrics, ranging from no improvement in HMA overlay life to as much as 3.6 years of additional life. Subsequent evaluation of six Nevada highway pavements containing fiberglass yarn fabric showed mixed performance: three of the pavements experienced no distress after 4 to 6 years in service, while the other three experienced reflective cracking after 1 to 3 years of service (Hajj et al. 2007). Similar assessments of performance were provided in Von Quintus et al. (2009a), which also referenced a study indicating little benefit from fabrics placed with thin HMA overlays, but good success for fabrics placed with thicker overlays.

While geotextile fabrics have seen some success, they do not work well in placements where higher horizontal and vertical deflections are expected, or where large temperature swings might be experienced throughout the course of the year. Holtz et al. (1998) reports that geosynthetics can be effective in retarding reflective cracking from low- and medium severity alligator cracking. Button and Lytton (2007) indicate good performance when used in the proper application but also acknowledge that geosynthetics perform considerably better in warm and mild climates than in cold climates. Within a reasonable set of pavement conditions and as part of a relatively thick HMA overlay, it is believed that fabrics can generally provide between 2 and 3 years of additional life to the HMA overlay with respect to control of reflective cracking.

Geogrids

Geogrids are a relatively newer technology, having emerged in the 1980s. They may be woven or knitted from glass fibers or polymeric filaments, or may be cut or pressed from plastic sheets and then post-tensioned to maximize strength and modulus (Von Quintus et al. 2009a). Geogrids mitigate crack formation by providing tensile reinforcement to the HMA overlay, absorbing the stresses and strains from underlying cracks and distributing them over a wider area. Example geogrid products include Tensar Glasgrid®, Mirafi® PavingGrid, and TeleTextiles TeleRoad®.

Geogrids are strip products and typically have rectangular openings between 1/4 and 2 inches wide. To ensure proper bond and consolidation around the grid material, the opening size must be at least twice the size as the largest aggregate in the mix. Typical installation of the geogrid is after the initial lift of the HMA overlay and a subsequent tack coat. The geogrid system is placed over the entire surface and rolled into place without ripples. The remaining lift(s) of HMA are then placed on the geogrid.

Hajj et al. (2007) cited two studies involving the test and evaluation of geogrids in highway pavement HMA overlays. One study showed a geogrid product being ineffective in preventing or retarding reflective cracking. The other showed that fiberglass reinforcing grids can extend the life of the overlay by 2 to 3 times that of a non-reinforced overlay. Von Quintus et al. (2009a) noted a study suggesting good success with geogrids placed in HMA overlays on military airfields in the United Kingdom.

Properly applied, it is estimated that geogrid reinforcing systems can impart an additional 1 to 3 years of service life to the HMA overlay.

Mixture Modification

Another strategy to reduce reflective cracking is to modify the properties of the HMA overlay mixture so that it is less likely to crack. Specifically, this means improving the fracture resistance of the mix so that it can withstand the high stresses and strains above the cracks in the existing pavement (Von Quintus et al. 2009a).

The crack resistance of HMA depends on the asphalt grade, content, elastic characteristics, and temperature susceptibility (Von Quintus et al. 2009a). These properties affect the HMA mix's ability to absorb stresses generated at cracks and the self-healing properties, as well as its resistance to aging that causes the asphalt to become brittle with time. Improvements to these properties can be achieved by modifying the asphalt binder and/or increasing the film thickness of less viscous (softer) asphalts.

Two of the more effective modified binders are rubber-modified asphalt and polymer-modified asphalt. While these materials are generally unable to delay reflective cracks by themselves, they can be used to increase the probability of success when used with other crack mitigation strategies (e.g., stress/strain relieving interlayers and HMA overlay reinforcement).

Rubber-Modified Asphalt

Rubber-modified asphalt is a blend of hot asphalt cement, reclaimed tire rubber (15 to 20 percent by weight), and other additives. When mixed with asphalt at about 375 °F, the crumb rubber particles from the reclaimed tires swell to about twice their original volume and become softer and more elastic. This increase in flexibility enables the rubber-modified HMA mixture to withstand higher stresses and strains without breaking. Furthermore, because rubber-modified asphalt has improved age-hardening properties, it is able to resist reflective cracking longer than a mix with a conventional binder.

Rubber-modified HMA is placed using conventional asphalt paving equipment and techniques. The overlay is placed on a tacked and properly prepared (localized maintenance and repair) existing pavement surface. As noted, the benefits of a rubber-modified HMA are more pronounced when other crack mitigation techniques are used with it.

Although no specific performance benefits were given, Von Quintus et al. (2009a) reported that rubber-modified asphalt mixtures designed with proper strength and stability have delayed reflective cracking much longer than conventional neat mixes, polymer-modified mixes, and stone matrix asphalt (SMA) mixes.

Polymer-Modified Asphalt

Polymers are a large class of natural and synthetic materials. When added to asphalt cement, they modify the natural viscoelastic behavior of the asphalt cement, producing a material that is less temperature susceptible and has higher viscosity at ambient temperature compared to unmodified asphalt (see figure 72). And, like rubber-modified asphalt, polymer-modified asphalts have improved age-hardening characteristics.

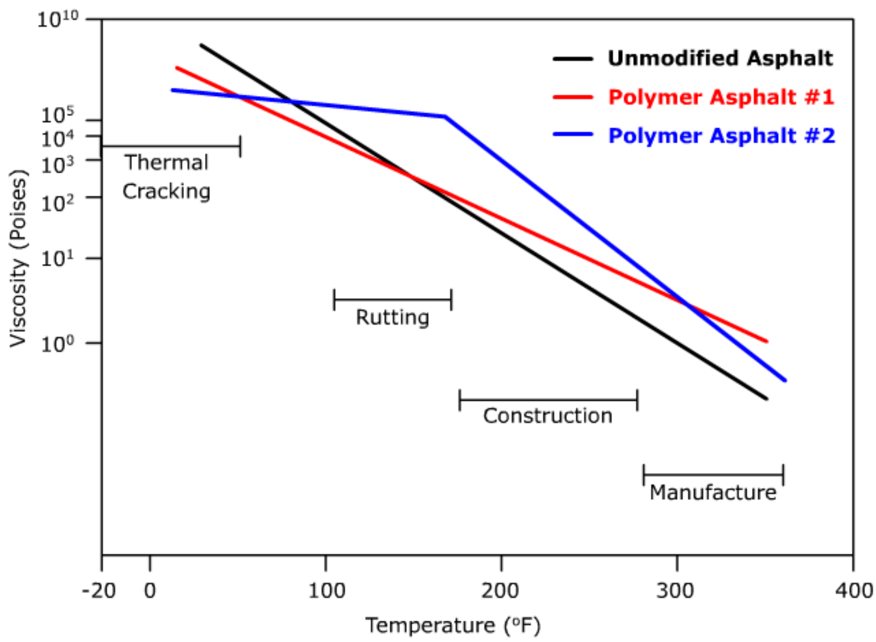


Figure 72. Effect of polymer modifiers on asphalt binder viscosity (www.pavementinteractive.com).

There are two main classes of polymers: elastomers, which enhance strength at high temperatures and elasticity at low temperatures, and plastomers, which enhance strength but not elasticity. The most commonly used polymers for asphalt are the elastomeric styrene-butadiene diblock (SB), ethylene-vinyl-acetate (EVA), styrenebutadiene triblock (SBS), and styrenebutadiene rubber (SBR). The addition of these types of polymers to asphalt cement results in a binder with higher viscosity, increased adhesion/stickiness, and increased elasticity. The higher viscosity yields thicker binder coatings, which prolongs the oxidation process. The increased adhesion translates into an improved ability to hold aggregate particles together. In addition, the increased elasticity results in greater ability to resist the development of cracking.

Polymer-modified HMA is also placed using conventional asphalt paving equipment and techniques. The overlay is placed on a tacked and properly prepared (localized maintenance and repairs) existing pavement surface. As previously noted, the benefits of a polymer-modified HMA are more pronounced when other crack mitigation techniques are used.

Von Quintus et al. (2009a) cited a recent study indicating a substantial benefit associated with the use of polymer-modified HMA used for overlays on existing flexible and rigid pavements. The study showed reduced levels of distress (transverse thermal cracking, fatigue cracking, and rutting) and better smoothness, but did not consider reflective cracking of the overlay. As a result, Von Quintus et al. (2009a) recommended polymer-modified HMA be used in combination with other crack mitigation techniques to increase the success of the other methods.

CONSIDERATION OF CORRECTIVE AND MITIGATIVE TECHNIQUES IN DESIGN

The corrective and mitigative techniques presented throughout this chapter generate changes in the physical and mechanical properties of the HMA overlay and/or existing pavement. These changes are intended to enhance specific aspects of overlay performance, such as reducing vertical strain on the subgrade so as to reduce rutting and reducing the tensile and shear strains in asphalt layers so as to reduce (or delay) the development of cracks. Table 24 summarizes the targeted engineering/performance properties of the different techniques.

Table 24. Engineering/performance properties targeted by corrective and mitigative techniques.

Corrective/Mitigative Technique	Control of Reflective Cracking[1]	Improve Existing Pavement Structural Integrity[2]	Increase HMA Overlay Strength[3]	Increase HMA Overlay Elasticity[4]
Localized Maintenance & Repair				
Crack Sealing	✓	✓[5]		
Partial-/Full-Depth Repair	✓	✓		
Modify Existing Pavement				
Cold Mill	✓	✓		
HIR Surface Recycle	✓	✓		
Surface Leveling Course	✓	✓		
Stress/Strain Relieving Interlayers				
SAMI	✓			
ISAC	✓			
Strata	✓			
HMA Overlay Reinforcement				
Steel Reinforcement	✓		✓	
Geotextile Fabrics	✓		✓	
Geogrids	✓		✓	
HMA Mix Modification				
Rubber-Modified HMA	✓			✓
Polymer-Modified HMA	✓			✓

[1] Reduced tensile and/or shear strains at crack tip-overlay interface.
[2] Reduced tensile and vertical strains in existing asphalt layers and vertical strain on subgrade.
[3] Increased allowable strain (to reduce fatigue, thermal, and reflective cracking).
[4] Increased recoverable strain (to reduce fatigue, thermal, and reflective cracking).
[5] By waterproofing the overall pavement system.

As indicated in this table, most of the techniques address reflective cracking issues, which is not a FAARFIELD design criterion. Localized maintenance and repair techniques as well as other techniques that modify the existing pavement can also address fatigue cracking and/or permanent deformation issues, which are relevant design criteria in FAARFIELD (the latter typically being the controlling parameter). And, in addition to addressing reflective cracking issues, HMA overlay reinforcement techniques and mix modification techniques address fatigue and thermal cracking issues within the overlay.

Increase of Existing Pavement Structural Integrity

The current FAARFIELD overlay design procedure “assumes that the HMA overlay is to be placed on a base pavement with significant structural integrity” (FAA 2009). What constitutes “significant structural integrity” is not explicitly defined, but the procedure recommends that severely distressed areas and subsurface drainage conditions be carefully evaluated and corrective actions be taken to restore deficiencies prior to overlay application. Failure to do so, the procedure warns, will result in an unsatisfactory overlay due to the underlying deficiencies being reflected into the overlay (FAA 2009). There is little guidance as to the type, severity, and quantity of distress that should warrant repair.

As mentioned in previous chapters, the structural integrity of an existing PCC pavement and composite pavement is represented in FAARFIELD by the SCI. However, there is no corresponding basis for representing the structural integrity of an existing HMA pavement. Furthermore, in the case of a composite pavement, SCI can only be obtained through an evaluation of the load-related distresses in the underlying PCC. Since the PCC is covered with asphalt, an assessment of the PCC distresses is, at the very least, difficult.

Given the limitations regarding developing a meaningful SCI for HMA-surfaced pavements, its use in characterizing the structural integrity of the existing pavement is not recommended. An alternative to be considered is an adjustment in the pavement layer moduli derived from nondestructive (e.g., falling weight deflectometer [FWD], portable seismic pavement analyzer [PSPA]) and destructive (coring and lab analysis) tests performed on the pavement during a structural evaluation (discussed in Chapter 4). Aside from localized distressed areas identified in the condition survey—which according to Advisory Circular 150/5320-6E should be repaired using appropriate techniques like partial- and full-depth patching, crack treatment, milling, and leveling—nondestructive and destructive testing can be used to identify additional weak areas to be repaired and to refine the pavement layer moduli used as inputs into FAARFIELD.

FAA Advisory Circular 150/5370-11A describes the process of using nondestructive deflection testing (NDT) results (e.g., deflections, backcalculated layer and subgrade moduli, lab-determined moduli and strengths, and layer thicknesses) in the selection of an evaluation or design input. It recommends using the statistical mean minus one standard deviation as a conservative approach for establishing design inputs. The selection of values under this approach ensures that the designed HMA overlay is adequate for some of the weakest locations throughout the project. If significant amounts of repair are identified via nondestructive and destructive testing and those repairs are properly made prior to overlay, then the frequency distributions of pavement moduli from NDT testing could be shifted considerably. This is because the repairs made to weak areas will increase the pavement moduli in those areas (assuming the repairs are made properly and cover the full extent of the distressed area).

An illustration of this effect is provided in figure 73. The frequency distribution of HMA moduli from NDT is represented by the curve farthest to the left. If it is determined that a small portion (2.5 percent) of the pavement is far too weak (corresponding to the left tail of the curve) and must be repaired (via patching, leveling, or other means), then the repairs would shift the modulus distribution so that the design input is increased from “X” to “Y.” Similarly, if an even greater amount of pavement (5 percent) is deemed too weak and must be repaired, then the repairs would shift the modulus distribution even further so that the design input is increased to “Z.”

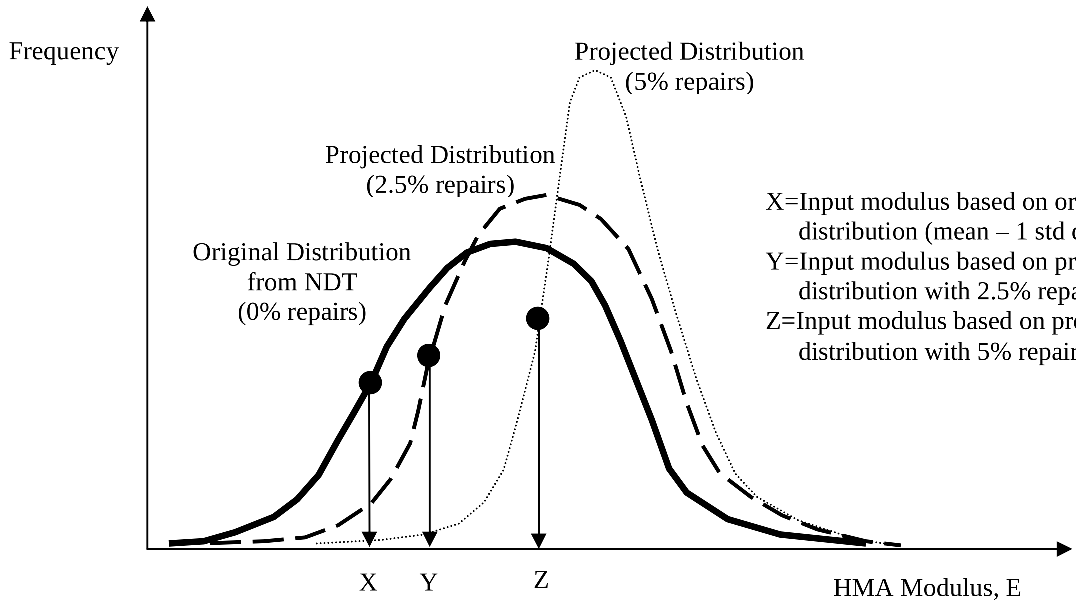


Figure 73. Effect of repairs on selection of pavement modulus.

This approach can also be used to assess the amount of risk for premature failure an airport owner is willing to assume. This degree of risk will depend on many factors, including the type of facility (runway, taxiway, or apron), the volume of traffic on the facility, the cost of the repairs, and the impact of premature failure on operations and costs. For a given condition, it might be determined that there is a 15 percent chance of premature failure. But by performing a limited amount of pre-overlay repairs for some set cost, that risk can be reduced, perhaps to as low as 5 percent (note: it may never be cost effective to completely eliminate the risk). Alternatively, the overlay thickness can be reduced to achieve the same level of risk at a lower initial cost. The level of risk and associated impacts must be weighed by the engineer, working closely with the airport owner, considering the factors previously discussed.

Reflective Crack Control

Reflective cracking is a major distress mode in HMA overlays of existing HMA, composite, and PCC pavements. The complex combination of tensile and shear strains at the bottom of the overlay-crack tip interface cause cracks to initiate at that point and propagate upward through the overlay. As the process continues, multiple reflective cracks form and eventually portions of the HMA overlay begin to spall and dislodge from the pavement surface. Although the cracks can be sealed to slow the rate of deterioration, the cracks can lead to a significant reduction in smoothness and shorten the life of the overlay.

Since reflective cracking is not a performance parameter currently evaluated in FAARFIELD, it is difficult to account for the purported performance benefits described earlier in this chapter for the various crack-mitigating techniques. These benefits primarily relate to the delay of reflective cracking (and probably fatigue cracking) development and not to the reduction in the development of permanent deformation, although some techniques (e.g., HMA overlay reinforcement and polymer-modified HMA) could influence this performance parameter.

Chapter 2 describes a process by which adjustments to the thickness and/or modulus of an existing cracked HMA pavement can be made to accommodate extra HMA overlay thickness required to prevent reflective cracking. The process is based on research performed by Sousa et al. (2005) and limited sensitivity analysis using FAARFIELD. It is premised on finite element analysis results indicating that reflective cracking development is driven in part by increased thickness and stiffness of the existing pavement and that to counter the rate of crack development, a thicker HMA overlay is required. This process entails the following four steps:

1. Determine the thickness ($t_{measured}$) and modulus of the existing cracked layer.
2. For every 145,000 psi above a modulus of 145,000 psi (referring to the existing layer modulus), reduce the thickness of the existing cracked layer by 2 inches ($t_1 = t_{measured} - 2*\delta m$, where "δm" is the multiple of 145,000 psi above 145,000 psi modulus). Use proportional values for other moduli.
3. For every 6 inches of thickness above 6 inches (referring to the existing layer thickness), reduce the thickness of the existing cracked layer determined in step 2 by 1 inch ($t_{input} = t_1 - 1*\delta t$, where "δt" is the multiple of 6 inches above 6 inches of thickness). Use proportional values for other thicknesses.
4. Use the thickness determined in step 3 as the input thickness of the existing cracked layer in FAARFIELD.

As an example, if testing showed the modulus of an 8-inch thick existing cracked pavement to be 435,000 psi, then t_1 would be computed to be 4 inches [8 – (2)*(2)] and t_{input} would be computed to be 3.33 inches [4 – (2/6)*(1)]. This value of 3.33 inches would then be used in FAARFIELD to determine the additional HMA overlay thickness needed to satisfy reflective cracking issues—the additional thickness being the difference between the HMA overlay thicknesses computed using a 3.33-inch existing layer thickness and an 8-inch existing layer thickness.

The reflective crack mitigation techniques described in this chapter have the ability to affect performance as if an additional overlay thickness had been placed, and thus slow the rate of crack development. In fact, geotextile fabrics were reported as being equivalent to between 1.2 and 2 inches of HMA and capable of providing between 2 and 3 years of additional HMA overlay life, under moderate circumstances. The performance of other crack mitigation techniques were found to be similar or slightly less than that of geotextile fabrics, and it would be reasonable to believe that "equivalent HMA thicknesses" could be applied to these techniques as well.

Table 25 provides suggested equivalencies for these techniques, based on the estimated overlay life extensions presented in this chapter. Depending on the results of a supplemental analysis of reflective cracking using the process outlined in Chapter 2, a crack mitigation technique could be selected using the equivalency values in table 25, which could offset the required additional overlay thickness.

Table 25. Summary of estimated performance of crack mitigation techniques and corresponding equivalent HMA thicknesses.

Crack Mitigation Technique	Estimated HMA Overlay Life Extension, years	Estimated HMA Thickness Equivalency, in
SAMI	1 to 3	1.0 to 2.0
ISAC	1 to 2 (minimum)	1.0 to 1.5 (minimum)
Strata	1 to 2 (minimum)	1.0 to 1.5 (minimum)
Steel Reinforcement	1 to 2 (minimum)	1.0 to 1.5 (minimum)
Geotextile Fabrics	2 to 3	1.5 to 2.0
Geogrids	1 to 3	1.0 to 2.0

SUMMARY

This chapter presents various corrective and mitigative techniques and methods that can be used to repair existing distress and improve HMA overlay performance. The discussion includes a description of the technique, its function or purpose, the expected impact of each activity on the performance of the overlay, and an assessment of how the technique should be accounted for in the HMA overlay design process. The following activities are discussed herein:

- Localized Maintenance and Repair.
  - Crack sealing.
  - Partial-depth patching.
  - Full-depth patching.
- Modification of Existing HMA Pavement.
  - Cold milling.
  - Surface recycling.
  - Surface leveling course.
- Stress/Strain Relieving Interlayers.
  - Stress-absorbing membrane interlayer (SAMI).

- Proprietary stress-absorbing interlayers (ISAC, STRATA®, etc.).

- HMA Overlay Reinforcement.
  - Steel or wire fabric.
  - Geogrids.
  - Geosynthetic fabric.

- HMA Overlay Mixture Modification.
  - Rubber-modified asphalt binder.
  - Polymer-modified asphalt binder.

The activities can be used by themselves or in combination with other techniques, as is often done.

Finally, possible approaches to account for these corrective or mitigative activities in the FAA's HMA overlay design procedure, both in terms of structural capacity and reflective cracking, are presented. The information presented in this chapter forms the basis for the recommendations presented in Chapter 6.

CHAPTER 6. RECOMMENDATIONS FOR REVISIONS TO FAA'S HMA OVERLAY DESIGN PROCEDURE AND FOR FUTURE RESEARCH

INTRODUCTION

The design of HMA overlays in the FAA's pavement design procedure is based on a thickness deficiency approach, with subgrade rutting as the primary failure criteria and HMA fatigue checked as a secondary criteria. However, HMA overlays deteriorate due to many factors other than subgrade rutting and HMA fatigue. Additionally, it is widely recognized that the condition of the existing pavement structure has a significant influence on the performance of an HMA overlay (not currently considered in the FAA's HMA overlay design procedure). Therefore, it is important to accurately characterize the existing pavement structure in the design process. Consideration of the improvement (or lack of improvement) to the existing pavement through pre-overlay repairs (or doing nothing) also needs to be considered.

The FAA's Advisory Circular on pavement design discusses the evaluation of existing pavements in the HMA overlay design procedure, but provides marginal direction on the use of the data collected. Similarly, while general guidelines are provided for the types of repairs that should be considered during rehabilitation, the design procedure does not account for any improvement in performance of the overlay (or reduced service life if corrective actions are not applied).

Previous chapters discuss the following aspects of HMA overlay design:

- Available design procedures and performance models.
- Sensitivity of FAARFIELD design inputs within the HMA overlay design procedure.
- Assessment of existing pavement evaluation methods.
- Assessment of corrective actions (or pre-overlay repairs).

This chapter summarizes the findings of the above tasks and presents recommendations for additional guidance and improvements to the HMA overlay design procedure.

DESIGN PARAMETERS AFFECTING HMA OVERLAY PERFORMANCE

The design parameters for the FAA's HMA overlay design procedure include the following:

- Aircraft traffic volumes and weights.
- Pavement layer types and thicknesses.
- PCC flexural strength (for composite pavement).
- CDFU and SCI (for composite pavement).
- Subgrade support.

These inputs are used within the FAARFIELD pavement models to predict HMA overlay performance until the design criteria are satisfied.

Performance Models

The FAA design procedure for HMA overlays over existing HMA pavement is primarily based on protecting the subgrade from rutting, just as with the new HMA pavement design procedure. While FAARFIELD allows the fatigue performance to be checked, it is not the primary design criteria and it seldom, if ever, controls the design. The design of HMA overlays of existing composite pavements is based on fatigue cracking of the underlying PCC pavement and HMA overlay performance is not directly considered. However, review of other aviation industry design procedures indicates these approaches are not uncommon.

The assessment of subgrade rutting and HMA fatigue do not directly account for other performance measures commonly associated with HMA overlays, including reflective cracking and in-layer rutting. One could argue that these performance criteria are related to materials and construction preparation and are covered elsewhere: the FAA design guidance provides discussion on several pre-overlay repair and reflective cracking mitigation techniques, and the FAA P-401 and P-403 specifications are generally assumed to provide rut resistant mixes.

However, models being developed can provide the designer useful performance estimates to assist with design selection. These include reflective cracking, thermal cracking, and in-layer rutting. In general, these models would need additional material inputs as part of the design procedure.

Additionally, research suggests that aged HMA does not necessarily perform in the same manner as a new HMA (Freeman et al. 2008). With current fatigue equations, the allowable load applications decreases as the modulus increases. However, testing of aged HMA samples suggests that the observed decrease in fatigue in the current equations is a result of the conditions under which the models were developed (Freeman et al. 2008). The research conducted under the study suggests fatigue performance increases as the modulus (or stiffness) increases.

Selection of Layer Inputs

Within the pavement models, the selection of layer types, moduli, and thicknesses can have a significant impact on the predicted performance as well as the actual performance observed in the field. The default layer types (such as P-401, P-403, and so on) have moduli that cannot be changed by the designer. While there are valid reasons for this for new HMA pavement design, these default layer types may not accurately characterize the existing material. However, there are “variable” and “undefined” layer types available that do allow the input of something other than the default moduli.

The sensitivity analyses summarized in Chapter 2 illustrate that in some cases the use of default layers can result in overly conservative designs or inadequate designs. Based on the analyses conducted, in terms of their effect on overlay thickness for flexible pavements, the existing layer inputs have the following influence:

- Subgrade support – a change of 5 percent in the subgrade support can result in nearly an inch (approximately 10 percent) difference in the determined HMA overlay thickness.
- Existing HMA thickness – a change in existing HMA thickness is essentially equivalent to the change in the required HMA overlay thickness, depending on selected modulus values (or layer types).
- Existing HMA modulus – a 20 percent change in existing HMA modulus results in about a 10 percent change in the required HMA overlay thickness; however, as the difference in the existing modulus from the default layer modulus becomes larger, so does the relative change in the required HMA overlay thickness. In other words, it is not a linear relationship.
- Existing PCC flexural strength – a change in the underlying PCC flexural strength in a composite pavement results in a significant change in HMA overlay thickness: a change of 5 percent can result in a 25 percent change in required overlay thickness.

An aspect not directly taken into account in the FAA's design procedure is the temperature dependency of HMA materials. This is especially relevant to the surface layer. The default HMA overlay modulus is 200,000 psi based on a design temperature of 90 ^{o}F. The analyses conducted suggest that HMA fatigue can become a significant factor when the overlay modulus exceeds around 300,000 psi, a value that could easily be exceeded in most climatic regions. To account for temperature sensitivity, the Asphalt Institute design procedure incorporates the mean annual air temperature (MAAT), and the military's layered elastic design software (Pavement-Transportation Computer Assisted Structural Engineering [PCASE]) allows the customization of "seasons" for the project site, as shown in figure 74.

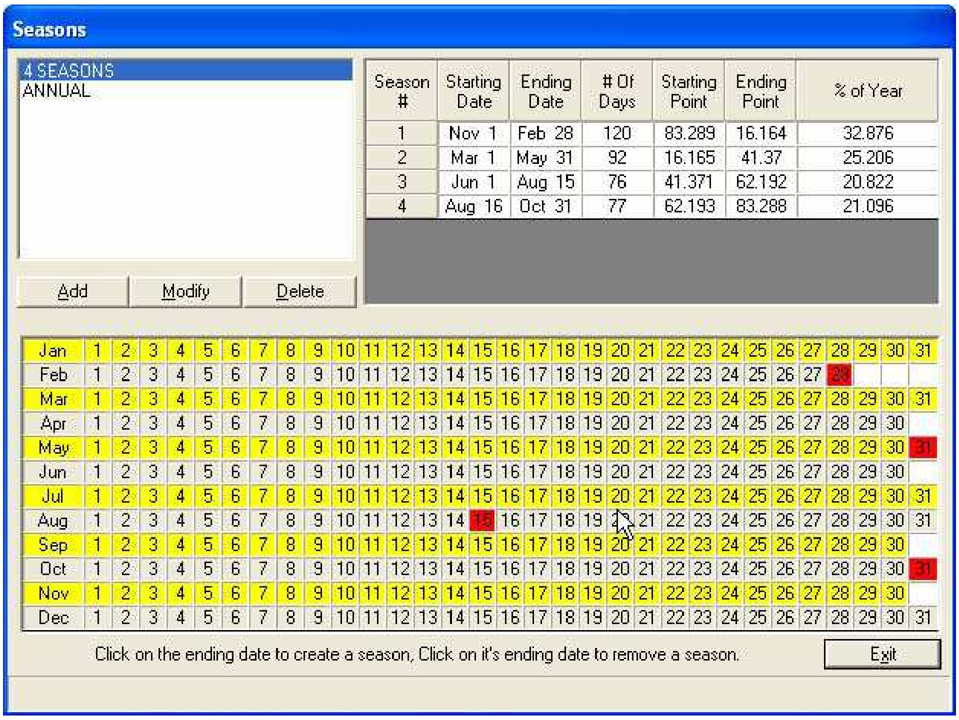


Figure 74. Illustration of seasonal capabilities of PCASE (Walker and Adolf 2005).

The number of seasons is user defined, and PCASE is capable of defining seasons from one annual season to each day being its own season. Each defined season then has appropriate layer moduli input, including subgrade, base, and HMA moduli. Obviously, the more seasons that are defined, the more data are required to be input.

Schemes such as MAAT or the use of seasons are an improvement in accounting for the temperature dependency of the HMA and the change in modulus throughout the year for specific climates. An additional advantage to the use of seasons in PCASE is the ability to account for seasonal fluctuation in the stiffness of the supporting layers (such as from moisture fluctuations, frost penetration, and so on). Neither of these approaches directly account for daily fluctuation in modulus, but at that level of input it becomes quite burdensome on the user and an automated process is needed.

In addition to the temperature dependency, the selection of asphalt cement grade and other mix properties during design can result in much different mixture behavior and moduli. Ongoing studies on the incorporation of Superpave mixture design into the P-401 specification have concentrated on the material specification and have not addressed the potential impacts to the design models. The HMA modulus also depends on the loading frequency.

Except in the composite pavement overlay design procedure, continued deterioration of the underlying pavement is not directly taken into account. In the overlay design of a composite pavement, the CDFU and SCI factors are used to characterize the structural condition of the underlying PCC. These descriptive factors are used to estimate incremental decreases in the PCC structural capacity over the design period. With the overlay of an existing HMA pavement, the pavement layers are assumed to have the same properties throughout the entire design period. This may not necessarily be true. Cracking of the existing HMA materials can reduce the underlying support properties and aging of the HMA can change the material’s modulus. However, it is not clear that SCI is an appropriate approach to account for these changes (discussed in more detail later).

ASSESSMENT AND CHARACTERIZATION OF EXISTING PAVEMENT

Characterization of the existing pavement layers is an essential step in the design of HMA overlays. While the FAA’s design procedure does emphasize the need to determine the existing pavement structural condition, little guidance is provided on how to incorporate such data into the design procedure. There are a number of evaluation techniques available that can be used to assess the condition of existing pavements and to determine appropriate design inputs, including visual, destructive, and non-destructive techniques.

As discussed in chapter 4, there are both advantages and disadvantages to traditional evaluation techniques; these are summarized in table 26. Therefore, some combination of available methods usually provides the best results.

Table 26. Summary of advantages and disadvantages of traditional evaluation techniques.

Evaluation Technique	Common Application	Advantage	Disadvantage
Visual Inspection	PCI, SCI	• Relatively rapid and easy to conduct • Can be less disruptive to airfield operations • Comparatively inexpensive • Entire pavement can be inspected	• Limited to condition of pavement surface • Subjectivity introduced by inspector's interpretation of distress • Does not definitively determine cause(s) of distresses
Destructive Testing	Coring, Boring, Laboratory Testing	• Provides subsurface data (material types, layer thickness, condition of layers, and so on) • Laboratory testing can provide material properties	• More disruptive to airfield operations • Relatively more expensive • Costs limit amount of data generated
Nondestructive Testing	FWD, GPR, PSPA	• Many locations can be tested • Used to determine overall pavement response to load and individual layer properties (FWD) • Evaluates surface properties (PSPA) • Generates layer thicknesses (GPR, PSPA surface layer) • Less disruptive to operations than destructive sampling	• Requires more experience to interpret data and results • Often needs to be supplemented with some destructive sampling • Results are often more variable (but can be offset by ability to collect more data)

Visual inspection is primarily based on the PCI procedure. While PCI results are not currently part of the design procedure, the observed distresses and severity levels can be used to determine limiting conditions and pre-overlay repair requirements and quantities.

Pavements with a PCI of 41 to 70 typically require major rehabilitation, such as an HMA overlay. Pavements with PCIs greater than 70 generally do not require overlays (unless to restore ride quality), and pavements with PCIs of 40 or below are often considered for reconstruction because of the severe extent of deterioration. While these divisions do not constitute hard-and-fast rules, they do provide general planning benchmarks.

As discussed in previous chapters, a subset of distresses within the PCI condition data can be used to establish the SCI for composite (or PCC) pavements, and is used in FAARFIELD as a condition index to characterize the existing PCC structure. The assessment of SCI for composite pavements is more difficult because the HMA overlay masks the PCC pavement distresses, but a general assessment of the visible cracking, historical performance data, and data from other testing methods (such as deflection testing) can be used to establish an appropriate SCI.

An SCI of 80 corresponds to the FAA definition of structural failure of a rigid pavement, which is consistent with 50 percent of slabs in the traffic area exhibiting structural cracking. An SCI of either 40 or 57 (depending on the base/subbase types) is used in FAARFIELD as the terminal value in the life computation for an HMA overlay of a composite pavement.

SCI is not currently used with HMA pavements. As presented in chapter 4, an SCI of 80 for an HMA pavement (under a greatly simplified analysis) roughly equates to any one of the following conditions:

- 0.15% of the pavement surface area with high-severity fatigue cracking.
- 0.35% of the area with medium-severity fatigue cracking.
- 0.95% of the area with low-severity fatigue cracking.
- 0.10% of the area with high-severity rutting.
- 0.45% of the area with medium-severity rutting.
- 2.5% of the area with low-severity rutting.

To illustrate further, for a 5,000 ft^2 sample unit (50 by 100 feet), 0.15 percent of high-severity fatigue cracking equates to approximately 7.5 ft^2 of the distress. Therefore, full-depth patching as a pre-overlay repair could still be an option even though the SCI is 80. Continuing with the fatigue cracking example, assuming the only deterioration is high-severity fatigue cracking, approximately 6 percent of the area (or 300 ft^2) would need to be deteriorated to bring the overall PCI to a level where reconstruction is generally considered (a PCI of 40), which would also equate to an SCI of 40. This would essentially be both wheelpaths—assuming one defined wheelpath to each side of centerline approximately 1 to 2 feet wide—at high-severity for the entire length of the sample unit. While it is unlikely that high-severity fatigue cracking would be allowed to develop to such an extent without being repaired, the example suggests 94 percent of the remaining pavement could be in good condition.

If one assumes that structural failure in fatigue is 50 percent of the wheelpath, approximately 3 percent of the sample unit area would be deteriorated for high-severity fatigue cracking, which correlates to an SCI of approximately 50. Similarly, 100 percent of the wheelpath with only high-severity rutting would result in an SCI of around 50. Therefore, an SCI lower than 80 may be more appropriate for an HMA pavement. While it could be a very useful way of characterizing support conditions, additional research would be needed to consider the use of SCI to characterize existing HMA.

There are many destructive and non-destructive evaluation techniques available for assessing layer properties. There is not necessarily a single, best alternative, but rather several methods should generally be used in combination. The goals of the evaluation methods should include the following:

- Estimating subgrade support. After aircraft weight, subgrade support is the most influential input in flexible pavement design, including HMA overlay design. Based on the sensitivity analyses conducted in chapter 3, a 5 percent difference in subgrade support can result in nearly a 1-inch difference in overlay thickness.
- Determining pavement layer thicknesses. Pavement layer thickness is a required input and has a significant influence in analyzing deflection testing data. The existing HMA layer thickness is also a relatively influential input for determining the required HMA overlay thickness.
- Establishing pavement layer characteristics. Existing HMA modulus and granular material moduli have less of an effect than subgrade support (based on the sensitivity analyses presented in chapter 3), but still result in noticeable changes in required HMA overlay thicknesses, particularly when moduli differ by more than 20 percent from the default layer properties.

Traditional destructive testing methods include pavement coring and soil boring. These methods are generally used to establish layer thicknesses, to classify unbound materials, and to characterize the general quality of materials. Subgrade samples are also often tested in the laboratory to determine support properties. Laboratory testing of bound layers is less common, with PCC compressive or splitting tensile strength testing being most common. HMA material testing is more common in roadway evaluations and several properties have been incorporated into the MEPDG Design Guide. These material properties are used in thermal cracking and in-layer rutting models, which are not currently part of the FAA design procedure.

While other NDT methods are available, deflection testing (followed by backcalculation of the layer moduli) is the most common technique for determining pavement layer properties. With thickness data obtained from records, coring, and/or GPR, deflection data are used in the backcalculation process to determine layer properties for the pavement structure. Deflection testing can be conducted over a wide range of temperatures, and the backcalculation results for the HMA layer will represent the material properties at the temperature during testing. Therefore, results of the HMA modulus backcalculation must be adjusted to the currently assumed design temperature for comparison with the default layer types or for use as design inputs.

Typical moduli for pavement layers are provided in table 19, and the FAA's guidance on selecting design inputs based on backcalculation (from Advisory Circular 150/5370-11A) is summarized in table 20. Backcalculated moduli can vary considerably from the default layer type moduli in FAARFIELD, and the provided guidance can result in some significant differences between the determined properties and those used in design. Based on the sensitivity analyses conducted as part of this project, differences of more than approximately 20 percent in the existing HMA should be evaluated when determining HMA overlay thickness. Moduli below 100,000 psi likely indicate the pavement should be reconstructed (or at least the entire surface layer replaced) instead of simply placing an overlay. Additionally, determined surface HMA moduli need to be correlated to the assumed design temperature.

One consideration with NDT is that it is typically performed at intact locations. Therefore, the determined moduli may not be accounting for cracking or other deterioration of the pavement (locally the material may appear sound, but globally the layer is not sound). An additional consideration is that some distresses may be repaired (discussed in more detail later) prior to overlay, which will also be a change to the overall condition of the pavement, but pre-overlay repairs often focus on areas of medium- to high-severity distress, leaving low-severity distress.

Previous FAA design methods (conventional or nomograph) used equivalent thickness factors to relate existing material to "new" material based on condition. The Asphalt Institute design method uses a similar approach and provides guidance based on the level of deterioration, as shown in table 5. These equivalency factors generally result in a structural reduction to the comparable "new" cross section. That approach can still be used, where the thickness determined from evaluation is adjusted based on the overall condition and the determined layer modulus.

A similar approach could be taken to adjust the determined modulus. The effect of changing the existing HMA pavement modulus, with the overlay as the default layer type, is shown in figure 75. The effect of changing the HMA overlay modulus, with the existing pavement set as default layer types, is shown in figure 76. There is difficulty in assessing the combined impact of existing HMA and HMA overlay moduli on HMA fatigue within FAARFIELD because one or the other layer needs to be a default layer type to obtain fatigue calculations for both layers.

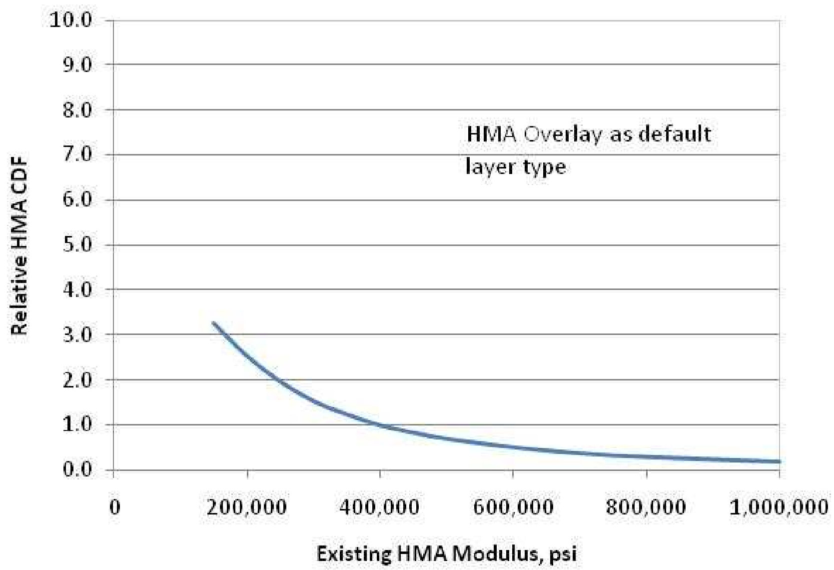


Figure 75. Influence of existing HMA modulus on HMA fatigue damage.

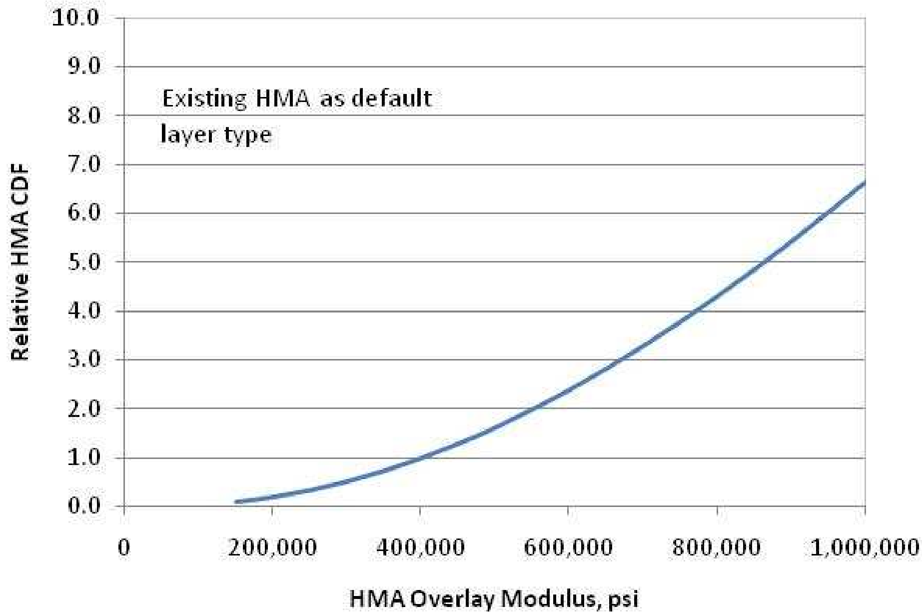


Figure 76. Influence of HMA overlay modulus on HMA fatigue damage.

As illustrated in figures 75 and 76 (and in chapter 3), there is an opposite effect for varying the two layers' moduli: increases in the HMA overlay modulus results in an increase in damage, but increases in the existing pavement modulus results in a decrease in damage. For both cases the HMA CDF is controlled by the existing pavement layer.

Based on the analyses and roughly correlating the Asphalt Institute thickness equivalencies to PCI, a possible adjustment scheme is presented in figure 77. The potential adjustment is based on the estimated (or effective) condition after pre-overlay repairs are performed. The effective PCI can be determined by modifying the collected PCI data to account for planned repairs. This possible adjustment assumes a modulus reduction based on overall condition. A reduction is based on the backcalculation assumption that testing is performed at intact locations; therefore, the backcalculated value is not entirely representative of the overall condition when accounting for distressed areas. Such a scheme can be applied to both existing HMA and PCC layers. This approach needs additional research, but provides one possible adjustment scheme for the existing pavement modulus.

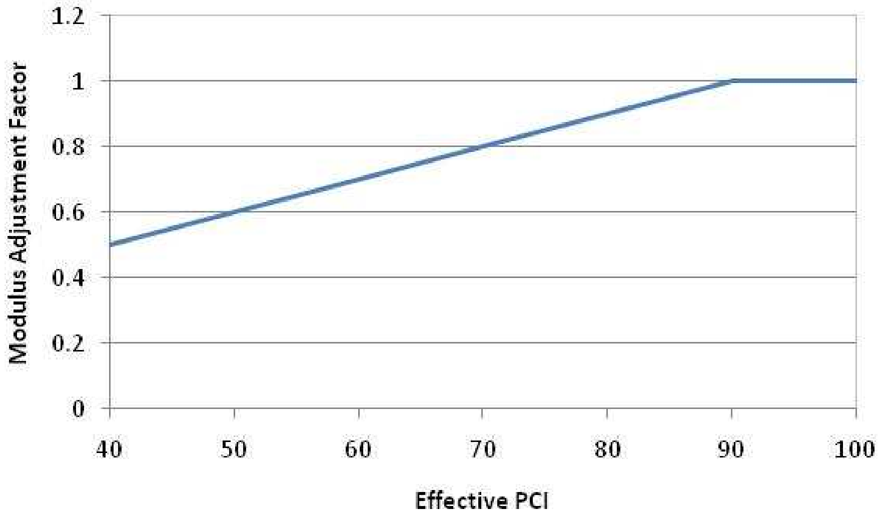


Figure 77. Potential existing pavement modulus adjustment based on effective condition.

Another possible approach is summarized in Chapter 5, where the determined HMA modulus is "improved" based on the level of repair effort. The disadvantage to the approach is that testing to determine the layer properties is likely not going to be performed in locations of high- or medium-severity distresses, which are the ones that would generally be repaired with corrective actions. Additionally, the localized repairs (such as patching) affect small areas and not necessarily the overall (or global) modulus.

Modulus adjustment is a relatively conservative approach, as shown in the sensitivity analyses in Chapter 3. While the layer thickness has a greater influence (essentially 1:1), any incorrect assumption about the layer thickness adjustment will have a more profound result in an under-designed or over-designed pavement section. Additionally, layer thickness adjustments would likely be based on a relationship of the existing layer modulus compared to the default (or standard) layer modulus. The relationship between modulus and effective thickness would need to be validated.

For assessing the contribution of the existing HMA overlay for a composite pavement, the thickness of the existing HMA overlay needs to be taken into consideration. In this case, once an effective thickness of the existing HMA overlay is determined, it is subtracted from the calculated required thickness of an HMA overlay over a bare PCC pavement. One possible estimate for an effective thickness (h_e) is as follows:

$$h_e = h_{existing} * \sqrt[3]{\frac{E_{existing}}{E_{new}}} \qquad \text{for } E_{existing} < E_{new} \qquad \text{(Eq. 31)}$$

or

$$h_e = h_{existing} \qquad \text{for } E_{existing} \geq E_{new} \qquad \text{(Eq. 32)}$$

where:

h_e = Effective thickness of existing HMA layer, in.
$h_{existing}$ = Thickness of existing HMA layer, in.
$E_{existing}$ = Elastic modulus of existing HMA layer, psi.
E_{new} = Elastic modulus of new HMA layer, psi.

EFFECTIVENESS AND CHARACTERIZATION OF CORRECTIVE ACTIONS

The previous discussion focused on how to assess the pavement layer conditions and adjust input data based on the analysis results. During rehabilitation design and construction, certain pavement distresses will be repaired, improving the general condition of the existing pavement. Chapter 5 provides a detailed description of the various pre-overlay repair methods, many of which are also briefly discussed in the FAA Advisory Circular.

Many of the pre-overlay activities are performed to address performance issues that are not part of the overlay design procedure, such as reflective cracking, thermal cracking, and so on. Repairs that alter the condition of the existing pavement layers are primarily patching (replacement of deteriorated materials with new, sound materials) and surface recycling, which would entirely change the properties of the surface but is not commonly used on airfields. Milling of the pavement generally will remove the majority of surface deterioration (such as weathering and raveling) and needs to extend deep enough to address distresses such as slippage cracking, if present.

Other repairs do not directly alter the condition of the existing pavement, but are used to improve the overall overlay performance. These include interlayers, cushion layers, overlay reinforcement, and HMA overlay mixture modifications. Such treatments alter the pavement response to loading, but primarily address mechanisms of distresses other than subgrade rutting and fatigue cracking.

As a minimum, the high-severity distresses and select medium-severity distresses, such as areas of alligator (fatigue) cracking or material instability, need to be repaired prior to overlay placement to ensure adequate performance. Without addressing severe distresses, an overlay cannot be expected to perform for an appreciable time. While pre-overlay repairs are generally expected to repair some level of distress, HMA overlays should not be considered if the results of an evaluation indicate the presence of any of the following conditions:

- Fatigue cracking is widespread and dictates full-depth pavement removal.
- HMA material deterioration, such as stripping, is widespread and dictates removal.
- Rutting is attributed to instability of the existing HMA, and the instability will result in the recurrence of rutting.
- Degradation or instability of the base/subbase material dictates removal and replacement.

Based on the review and discussion provided in Chapter 5, the suggested consideration of each corrective action is provided in table 27. Additionally, based on the sensitivity analyses conducted and the FAA's fatigue model currently incorporated into FAARFIELD, adjustment of the HMA overlay modulus is not recommended, even if the HMA material is designed specifically to improve pavement performance.

Table 27. Summary of corrective action design adjustments.

Category	Corrective Action	Proposed Adjustment
Localized Maintenance and Repair	Crack sealing	Adjustment based on estimated condition after repair.
	Partial-depth patching	Adjustment based on estimated condition after repair
	Full-depth patching	Adjustment based on estimated condition after repair
Modification of Existing HMA Pavement	Cold milling	Milling depth should remove as much surface distress as possible; reduce existing pavement thickness in design accordingly; adjustment based on estimated condition after milling
	Surface recycling	Not commonly used; would modify existing layer modulus and potentially thickness
	Surface leveling course	Include with the determined overlay thickness
Stress/Strain Relieving Interlayers	Stress-absorbing membrane interlayer (SAMI)	Equivalent thickness in external reflective cracking check
	Proprietary stress-absorbing interlayers (ISAC, STRATA)	Equivalent thickness in external reflective cracking check
HMA Overlay Reinforcement	Steel or wire fabric	Equivalent thickness in external reflective cracking check
	Geogrids	Equivalent thickness in external reflective cracking check
	Geosynthetic fabric	Equivalent thickness in external reflective cracking check
Crack Arresting or Cushion Layers	Aggregate	Not recommended
	Asphalt-aggregate	Model appropriate layer thickness and modulus
	Bond Breakers	Not recommended
HMA Overlay Mixture Modification	Rubber-modified asphalt binder	No adjustment
	Polymer-modified asphalt binder	No adjustment
	Soft (low-viscosity) asphalt binder	Not recommended

As suggested in table 27, an external design check for reflective cracking can be considered because reflective cracking is a significant performance factor for HMA overlays. As discussed in Chapters 2 and 5, one possible process for evaluating reflective cracking is through the approach developed by Sousa et al. (2005), which entails the following four steps:

1. Determine the thickness ($t_{measured}$) and modulus of the existing cracked layer.
2. For every 145,000 psi above a modulus of 145,000 psi (referring to the existing layer modulus), reduce the thickness of the existing cracked layer by 2 inches ($t_1 = t_{measured} - 2*\delta m$, where "δm" is the multiple of 145,000 psi above 145,000 psi modulus). Use proportional values for other moduli.
3. For every 6 inches of thickness above 6 inches (referring to the existing layer thickness), reduce the thickness of the existing cracked layer determined in step 2 by 1 inch ($t_{input} = t_1 - 1*\delta t$, where "δt" is the multiple of 6 inches above 6 inches of thickness). Use proportional values for other thicknesses.
4. Use the thickness determined in step 3 as the input thickness of the existing cracked layer in FAARFIELD.

The reflective cracking mitigation methods (or corrective actions) previously discussed can then be considered in terms of subtracting an equivalent thickness from the overlay thickness determined in step 4. Suggested equivalencies are provided in table 28.

Table 28. Summary of estimated performance of crack mitigation techniques and corresponding equivalent HMA thicknesses.

Crack Mitigation Technique	**Estimated HMA Thickness Equivalency, in**
SAMI	1.0 to 2.0
ISAC	1.0 to 1.5 (minimum)
Strata	1.0 to 1.5 (minimum)
Steel Reinforcement	1.0 to 1.5 (minimum)
Geotextile Fabrics	1.5 to 2.0
Geogrids	1.0 to 2.0

This procedure may or may not increase the recommended HMA overlay thickness, depending on the existing structural capacity and required overlay thickness determined based on subgrade rutting and HMA fatigue. These steps provide a relatively simple design check that can be used within the current version of FAARFIELD. More advanced models could be developed, but additional inputs would be needed. This procedure makes several assumptions from the work performed by Sousa et al. (2005) and needs further refinement and validation before being fully implemented into FAARFIELD.

HMA rutting should, for the time being, be considered during the HMA mix design and mixture specification. Asphalt binder selection and aggregate properties can be selected to address this failure mode for the specific project conditions until improved airfield performance-based models are developed.

RECOMMENDATIONS FOR REVISIONS TO DESIGN PROCEDURE

To expand on the discussion included in the FAA Advisory Circular, several recommendations are made from the research conducted during this study.

Overlay Design Revisions

Section 401 (Condition of Existing Pavement) indicates: *The overlay design procedures in this AC assume that the overlay is to be placed on a base pavement with significant structural integrity*. However, there is little discussion of what constitutes significant structural integrity (or lack of structural integrity). As such, additional discussion should be presented to provide guidance on when an HMA overlay would not be expected to perform as desired, either in section 401 or 404. Limiting conditions include the following, as discussed previously:

- Fatigue cracking is widespread and dictates full-depth pavement removal.
- HMA material deterioration, such as stripping, is widespread and dictates removal.
- Rutting is attributed to instability of the existing HMA, and the instability will result in the recurrence of rutting.
- Degradation or instability of the base/subbase material dictates removal and replacement.

Section 404 (Overlays of Existing Flexible Pavements) indicates: *The existing flexible pavement is characterized by assigning the appropriate thicknesses and moduli of the existing layers*. It should be added that the layer types and moduli determined through evaluation should be used in establishing the design section, and additional guidance on selecting layer properties is provided in chapter 6 of the Advisory Circular. The illustrative example should indicate the existing layers were found to be sufficiently close to the default layer types to warrant using them in the analysis.

The examples used in Section 405.c. (Hot Mix Asphalt Overlays of Existing Rigid Pavements) should indicate the existing PCC flexural strength was established by a detailed pavement evaluation.

Section 405.c.(3) (Previously Overlaid Rigid Pavement) does not provide guidance for assessing the existing HMA overlay. The modulus of the layer determined by evaluation can be used to estimate an effective thickness to subtract from the determined HMA overlay requirement, such as indicated in equations 6-1 and 6-2. Alternatively, assess the capability of including the existing HMA layer in the FAARFIELD modeling. The last sentence should be appended to: *The condition of the rigid pavement should be determined using* <u>evaluation methods and</u> *engineering judgment.*

Section 405.c.(5) (Reflection Cracking In Hot Mix Asphalt Overlays) provides discussion on addressing reflective cracking. The proposed reflective cracking design check approach discussed previously could be inserted within this section, although the details would need to be studied and developed.

Section 407.a.4 of Advisory Circular 150/5320-6E indicates that cracks 3/8-inch wide or greater should be filled with sand-asphalt material prior to overlay. Under this approach, when the cracks open, the material can sink down into crack, causing reduced support and localized loss of bond from loss of material. This guidance in the Advisory Circular should be reconsidered.

The temperature dependency of HMA materials is not entirely addressed. The flexible pavement design procedure indicates the surface and overlay moduli are conservatively selected to represent HMA at a design temperature of 90 ^{o}F. However, a pavement constructed in the northern climate is not going to perform the same as one constructed in the southern climate, as the HMA modulus fluctuations will be quite different. The use of performance-graded (PG) asphalt binders also influences the modulus with temperature. The inclusion of climatic modeling should be investigated, such as that provided in the PCASE design software.

Pavement Evaluation Revisions

Section 602.c. (Layer Properties) indicates that pavement constructed to FAA material specifications should be assigned corresponding layer types. The use of “undefined” or “variable” layers is discussed but primarily in the context of the aggregate layers. While the guidance indicates using these layers when the materials “differ significantly from the assumptions for FAA standard materials,” there is no guidance as to what “significant” means. The sentence goes on to say “or use lower quality material to model structure (e.g., P-154 or P-209),” implying that the representative modulus will always be lower than the default, which may not be the case. From the findings of this study, the existing pavement layer moduli should be modified if it differs by 20 percent or more from the default layer type being evaluated in the design analysis. The “undefined” layer type can be used to model the existing HMA layer with the determined design modulus.

This section should include additional guidance on assessing the results of evaluation efforts, including the effect of anticipated pre-overlay repairs. Backcalculated HMA layer moduli need to be adjusted to the design temperature. The HMA layer modulus should not be less than 100,000 psi. If it is determined to be lower, the pavement should be strongly considered for reconstruction (or at least removal of the HMA surface layer). A potential adjustment based on overall anticipated condition prior to overlay placement is illustrated in figure 77.

The examples included in Section 603. (Application of Flexible Pavement Evaluation Procedures) should either indicate the properties determined by evaluation are consistent with default layer types or the examples could be revised to illustrate the use of the “undefined” layer type for an HMA modulus significantly different from the default.

Economic Analysis

While not directly related to the design of an HMA overlay, the decisions made in design can influence the economic analysis. Assessing costs related to HMA overlays was not part of this project, but the following is provided for consideration during the design selection.

The discussion provided in Appendix 1 of the Advisory Circular covers several different alternatives and provides a detailed account of one life cycle cost analysis. In addition to the one life cycle cost analysis, the final costs are summarized—note that the cost summary table alternatives do not entirely match the original alternative list—which also includes assigning a "Chance for Success" to each alternative. There appears to be no discussion as to how the chance of success was assessed for each alternative. For HMA overlays, the chance of success can be assigned based on the level of pre-overlay repair effort, additional steps taken to mitigate distresses (such as reflective cracking), or overall condition of the pavement prior to overlay.

The summarized costs do not appear to account for corrective actions that may need to be made for each treatment, which may differ by treatment. For example, delaying the indicated overlay in lieu of the asphalt-rubber chip seal likely would require some repairs prior to the chip seal application and then again prior to the HMA overlay. It would be useful to have the remaining alternatives' life cycle cost analyses because without them it cannot be determined if corrective actions are considered in the other alternatives. It could also be anticipated that future maintenance needs should also increase with decreasing pre-overlay repair efforts (i.e., pay now or pay later). It is not known whether maintenance needs were adjusted for the various alternatives without having the other life cycle cost summaries.

INCORPORATION INTO FAARFIELD

It is recommended that calculation of the stresses and strains be performed for every HMA layer to evaluate fatigue damage and be included in FAARFIELD (note: the calculation of the fatigue damage for each layer was added in version 1.305; however, it is not the default design criteria). Based on the analyses run performed using the evaluation version of FAARFIELD v1.305, fatigue of the bottom layer can control the fatigue performance and should be included in the analysis. Additional features to consider in future versions of FAARFIELD include the following:

- The ability to adjust the HMA layer modulus without the need to use the "undefined" layer. In-place properties can be considerably different than the assumed new material property, and using the "undefined" layer type has limitations, particularly if the HMA overlay properties need to be evaluated.
- The ability to account for variation in the HMA modulus based on temperature. Climatic regions will have much different temperature spectrums, and it can be anticipated that HMA material will perform differently in different climates. Including the ability to consider seasonal variation in the modeling would simplify the assessment of temperature differences. The more refined temperature modeling becomes (such as daily fluctuations), the more pressing the need for an automated process. A simplification would be to use a modulus established similar to using MAAT, which at least accounts for differences in climatic regions.
- Characterization of existing HMA overlays in composite pavement sections. Directly modeling the existing layer in the FAARFIELD analysis may resolve the arbitrary assessment of its contribution in the required HMA overlay thickness.

RECOMMENDATIONS FOR FUTURE RESEARCH

The above discussions are based on assessing design calculations and available models. While it is useful to run sensitivity analyses, any revision needs to be validated with in-place pavement performance data and testing. Additionally, there are several materials and performance topics that are pertinent to HMA overlays. Additional research would be useful in the following areas:

- Continued work on reflective cracking performance predictions. One possible design check is based on work performed by Sousa et al. (2005); however, other models are available and research continues on this deterioration mechanism. Even if this method is accepted, additional work is needed to fine-tune the magnitude of the changes. With reflective cracking being a primary concern with HMA overlays, accounting for it in the design process is essential.
- Continue the development of assessing in-layer HMA rutting performance. While a simplified design model would be ideal, additional inputs to the design procedure are likely needed to provide an effective tool.
- Age-hardening effects on HMA modulus. Research conducted by Freeman et al. (2008) suggests that aged HMA does not follow the fatigue performance predicted by the current equations. In addition to review of the fatigue equation, accounting for age hardening in the design procedure may be applicable. One such method is described in the Global Aging System (Mirza and Witczak 1995), which is included in the AASHTO MEPDG.
- Influence of levels of pre-overlay repair effort on long-term pavement performance. Much of the research on repairs has been performed for roadways, and even that research does not provide a clear description of how repair efforts should be accounted for in design.
- Overlay designs with default and adjusted inputs. While the sensitivity analyses conducted using current design equations indicates varying influence of the design parameters, the actual in-place performance may or may not follow the findings. A testing program (such as at NAPTF) can assess these correlations.
- Feasibility of a structural index (such as SCI or some other concept) for HMA layers. Existing damage and continued damage of in-place HMA layers is not currently directly accounted for in the design procedure. While the layer modulus and thickness can be adjusted, continued fatigue damage may not be getting characterized appropriately. The design procedure historically considered fatigue in the surface layer, so this continued deterioration was not addressed. However, additional research would need to be performed to assess the appropriate means to characterize the continued deterioration as the use of SCI for HMA pavements may not be as clear as it is for PCC pavements.
- Top-down cracking. While different mechanisms have been proposed for the cause of top-down cracking, pavement studies suggest this distress warrants consideration. Whether the consideration is only needed in the mix design properties or in the structural design needs to be determined.

- Frequency/time of loading effects on HMA modulus. In addition to temperature effects, frequency of loading (or time of loading) influences the apparent HMA modulus. While the predominant research has been conducted for roadway design, this influence on airfield HMA overlays should be investigated.

SUMMARY

The performance of an HMA overlay is often measured by criteria other than subgrade rutting and HMA fatigue based on the current FAA pavement design procedure. Additionally, the condition of the existing pavement structure and the level of effort to correct deterioration have a significant influence on the performance of an HMA overlay. Therefore, it is important to consider other performance measures and to accurately characterize the existing pavement structure during the design process.

Two failure mechanisms typically associated with HMA overlay performance are reflective cracking and in-layer rutting. Discussion is provided in the design procedure for methods of mitigating reflective cracking, but it is not directly accounted for in the design of an HMA overlay. Similarly, the FAA's material specifications are intended to address the potential of in-layer rutting, but it is not directly accounted for in the design. A possible design check for reflective cracking is presented herein, which considers a simplified approach to determining an effective cracked section and then using FAARFIELD to determine HMA overlay requirements. Adjustments for treatments specifically for reflective cracking can be included in the approach. While there has been some aviation research on in-layer rutting, the developed models are generally mix specific. Therefore, no specific recommendation is made for a simplified approach to address this issue.

To develop the pavement design inputs for the existing pavement layers, a combination of available evaluation techniques—such as a PCI inspection, limited coring, and FWD testing, for example—typically provides the best results. The combination of techniques provides pavement layer thicknesses, material properties, general condition ratings, and estimates of pre-overlay repair needs. While the FAA's Advisory Circular discusses evaluation techniques, the guidance using the determined properties is limited. Guidance proposed in this chapter includes using determined properties when they vary by more than 20 percent of the default layer properties and an adjustment scheme for the HMA moduli based on pavement condition anticipated after pre-overlay repairs are performed. In addition to determining pavement properties at the time of testing, climatic conditions should be considered.

Based on the literature review and analyses performed as part of this study, recommendations for future research are also made, including in the following areas:

- Reflective cracking performance predictions.
- Assessing in-layer HMA rutting performance.
- Age-hardening effects on HMA modulus.
- Influence of levels of repair effort on long-term pavement performance.
- Validation of overlay designs with adjusted inputs compared to default inputs.

- Feasibility of using a structural index to characterize continued deterioration of the HMA.
- Assessment of top-down cracking significance in airfield HMA overlays.
- Frequency/time of loading effects on HMA modulus.

As this research is performed, further refinements and enhancements to the FAA's HMA overlay design procedure can be made.

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APPENDIX A - ANNOTATED BIBLIOGRAPHY

Abaza, K. 2000. "Performance-Based Models for Flexible Pavement Structural Overlay Design." Journal of Transportation Engineering, Vol. 131, No. 2, pp. 149-159 (February 2005). American Society of Civil Engineers, Reston, VA.

Performance of flexible pavement has long been recognized as an important parameter in the design of flexible pavements. Pavement surface condition evaluated using visual inspection is periodically done to assess pavement performance over time. A distinct performance curve is then constructed for each pavement structure that relates the pavement surface condition to service time or accumulated 80 kN equivalent single axle load applications. The presented flexible pavement overlay design models are constructed using performance curve parameters to provide an adequate overlay thickness at any given future time. The undertaken approach attempts to compensate an existing pavement structure for the loss in performance (strength) that it has endured over a specified service time. In essence, this approach is similar to the mechanistic methods of overlay design that make a compensation for the loss in a particular strength indicator such as the commonly used deflection method. Therefore, compensation is made for the loss in performance as represented by appropriately selected performance curve parameters. Performance parameters are then converted into equivalent relative strength indicators, which are in turn converted into equivalent overlay thicknesses. The relative strength indicators deployed in this paper are the structural No. and gravel equivalent used by the American Association of State Highway and Transportation Officials and the California Department of Transportation design methods of flexible pavement, respectively.

Anderson, D., Kosky, C. 1987. "Advances in Asphalt Overlay Design Procedures." *Proceedings*: International Conference on the Structural Design", University of Michigan, Ann Arbor, MI.

No Abstract Provided.

Barker, W., and Brabston, W. 1975. "Development of a Structural Design Procedure for Flexible Airport Pavements." FAA-RD-74-199; ADA-019-205. Army Engineer Waterway Experiment Station, Vicksburg, MS.

A design procedure is presented for three types of flexible pavement: conventional, bituminous concrete, and chemically stabilized. These represent nearly all flexible pavements being constructed at this time. Designs are based on analytically determined strain values and experimental and laboratory determined material fatigue strengths. Thus, the procedure can handle in a rational manner the possible variations in the properties of different pavement materials. An adaptation of the cumulative damage concept permits the consideration of cyclic variation in bituminous materials due to variations in temperatures and the variation in subgrade strength resulting from freeze-thaw cycles.

Bassem, A., Sadel, M., and Shahrour, I. 2008. "Elasto-viscoplastic Finite Element Analysis of the Long-term Behavior of Flexible Pavements: Application to Rutting." Road Materials and Pavement Design Vol. 9 No. 3, pp 463-379. Hermes Science Publications, France.

In this paper, the authors present a finite element model to be used in modeling long term pavement behavior, with focus on rutting. The model takes into account the nonlinear behavior of the different pavement layers and the influence of factors such as temperature, traffic load, traffic speed, and tire type on pavement rutting. The authors find that inelastic deformation from traffic is induced in both the asphalt and base layers. Results from numerical simulations indicate that pavement rutting increases with an increase in temperature for the asphalt concrete.

Bennert, T. and Maher, A. 2007. "Evaluation of the Current State of Flexible Overlay Design for Rigid and Composite Pavements in the United States." Transportation Research Record: Journal of the Transportation Research Board No. 1991. Transportation Research Board, Washington, DC.

Current composite pavement practices and key hot-mix asphalt (HMA) overlay design features, mainly to mitigate reflective cracking in the HMA overlay, used by state highway agencies in the United States are summarized. This information was obtained from a comprehensive survey conducted from January to May 2006 under a research effort for the New Jersey Department of Transportation to help minimize or retard the onset of reflective cracking in HMA overlays on portland cement concrete (PCC)/composite pavements. Information is provided on reflective cracking mitigation methods (successes and failures), current in-place composite/PCC pavement designs, treatments to prepare PCC pavement before HMA overlay and typical time until reflective cracking appears in the HMA overlay. Parameters that state agencies use to characterize pavement site conditions, including nondestructive testing, traffic conditions, and laboratory testing are also given. Further analysis is conducted by comparing the low temperature climatic conditions of the state to the low temperature performance grade of the asphalt binder commonly used in the HMA overlay. This examination and summary of survey findings on current pavement design practices and design features on the use of flexible overlays for rigid pavements in the United States will be of interest to state agencies, practitioners, and pavement researchers.

Blankenship, P. 2007. "Reflective Cracking Relief Interlayer for Composite Pavements." Asphalt Vol. 22 No. 2, pp 17-18. Asphalt Institute, Lexington, KY.

The author discusses reflective cracking in a composite pavement with a Portland cement concrete (PCC) primary composition with some hot mix asphalt thickness on top of the concrete. Reflective cracking, which results from vertical and horizontal movements, caused by temperature changes and traffic loadings, in underlying PCC joints and cracks, eventually weakens overall pavement structure and also reduces ride quality. Reflective cracking delay or elimination has as a key element asphalt overlay stress and strain production reduction or elimination. The author discusses relief strategies, technology implementation, and cracking reduction. Kentucky's 1-75 and 1-64 serve as examples.

Burns, C., Rone, C., Brabston, W., and Ulery Jr. 1974. "Comparative Performance of Structural Layers in Pavement Systems. Vol. I. Design, Construction, and Behavior under Traffic of Pavement Test Sections." FAA-RD-73-198 Vol. 1; AD-0785-024. Army Engineer Waterway Experiment Station, Vicksburg, MS.

Rigid and flexible pavement test sections were constructed to evaluate the performance of pavements incorporating a membrane-enveloped soil layer, insulating materials, chemically stabilized soil layers, and various types of surfacing including fibrous-reinforced concrete, plain portland cement concrete, and asphaltic concrete. These test sections were trafficked with 200- and 240-kip twin-tandem assemblies (Boeing 747 spacing) and a 50-kip single-wheel assembly. The design, construction, and behavior under traffic of the pavements are reported herein; the data will be used in further studies to determine the response of the pavement to both static and dynamic loads and to develop design and construction criteria. These studies will be reported in subsequent Vol.s.

Cao, Y.m Dai, S., Labuz, J F., and Pantelis, J. 2007. "Implementation of Ground Penetrating Radar.". Report No. MN/RC 2007-34. Minnesota Department of Transportation. Minneapolis, MN.

The objective of this project was to demonstrate the capabilities and limitations of ground penetrating radar (GPR) for use in local road applications. The effectiveness of a GPR survey is a function of site conditions, the equipment used, and experience of personnel interpreting the results. In addition, not all site conditions are appropriate for GPR applications. GPR is a nondestructive field test that can provide a continuous profile of existing road conditions. GPR utilizes high-speed data collection at speeds up to 50 mph, thus requiring less traffic control and resulting in greater safety. GPR has the potential to be used for a variety of pavement applications, including measuring the thickness of asphalt pavement, base and sub-grade; assisting in the analysis of rutting mechanisms; calculating and verifying material properties; locating subsurface objects; detecting stripping and/or layer separation; detecting subsurface moisture; and determining depth to near-surface bedrock and peat deposits. These applications are discussed in reference to 22 projects completed throughout the State of Minnesota.

Chatti, K., Haider, S., Buck, N., Pulipaa, A., Lyles, R., and Gilliland, D. 2006. "Factors Affecting Flexible Pavement Performance Based on the LTPP SPS-1 Experiment", ***Proceedings*****: 10th International Conference on Asphalt Pavements (August 2006). International Society of Asphalt Pavements. Quebec City, Canada.**

This paper is a summary of results from a study on the relative influence of design features on the performance of in-service flexible pavements. The data used in this study were from the SPS-1 experiment of the Long Term Pavement Performance program. This experiment was designed to investigate the effects of HMA surface layer thickness, base type, base thickness, and drainage on the performance of flexible pavements constructed under different site conditions (subgrade type and climate). Based on various statistical analyses conducted on the data, base type was found to be the most critical factor affecting fatigue cracking, roughness (IRI), and longitudinal cracking (in the wheel path). The best performance was shown by pavements with asphalt-treated bases. When combined with base type, drainage also plays an important role in improving pavement performance, especially in terms of fatigue and longitudinal cracking. Base thickness has only secondary effects, mainly for roughness and

rutting. In addition, climate was found to have significant effects. Also, in general, pavements built on fine-grained soils have shown the worst performance, especially in terms of roughness. The study provides an overview of the interactions between design and site factors as well as new insights on various design options.

Chen, D., and Scullion, T. 2008. "Forensic Investigations of Roadway Pavement Failures.". Journal of Performance of Constructed Facilities, Vol. 22 No. 1, pp 35-55. American Society of Civil Engineers. Reston, VA.

This paper presents three field projects to illustrate the integrated approach to forensic investigations of roadway pavement failures used widely in Texas. In each case, the combined ground-penetrating radar (GPR) and falling weight deflectometer (FWD) data were extremely useful for identifying contributing factors, such as stripping in the hot mix or localized areas of wet or weak base. Dynamic cone penetration (DCP) is used for validating problems with base and subbase layers. Laboratory tests are often required to complete the investigation, especially if the repair strategy calls for in-place recycling of the existing structure. The extent of stripping and high porosity that caused delamination for Projects 1 and 2 was detected by GPR and verified by core samples. GPR, FWD, DCP, and field soil samples all showed indications that the existing base on Project 1 was wet and the stiffness was only about one-third that of a typical flexible base in Texas. FWD data demonstrate that the pavement structures for Projects 1 and 2 were inadequate, so a rehabilitation strategy was selected that included structural strengthening. In Project 3, GPR, lab density, and permeability tests indicate that the dramatic pavement failures were attributable to moisture entering the base through a poorly compacted asphalt concrete layer and poor longitudinal joints. The base material was found to be highly susceptible to moisture and did not meet the Texas Department of Transportation's compressive strength requirements when subjected to capillary soaking. The repetitive triaxial test results revealed that the stiffness and load-carrying capability became inadequate when the base materials were exposed to moisture. The pavement structure had been totally replaced in 2004.

Cho, Y., Weissmann, J., and McCullough, B. 1995. "Initial Performance of Asphalt Overlays on Overlaid Jointed Concrete Pavement (JCP) and on Flexible Pavements in Field Test Sections in Lufkin, Texas. Interim Report.". Report No. TX-95+987-4. University of Texas, Austin, TX. Texas Department of Transportation, Austin, TX.

This report focuses on the performance of the various rehabilitation strategies implemented in the test sections placed on US 59 by the Lufkin District. Specific chapters discuss traffic, temperature, deflections, rutting, profiles, and the effectiveness of each rehabilitation strategy. The two sets of test sections constructed for this study have served as representative sections of rigid and flexible pavements in the Lufkin District. A Weigh in Motion (WIM) station was installed to collect such traffic information as axle classification, weight, speed, lateral distribution, and temperature -- all of which can contribute to pavement damage. The WIM records both air and pavement temperature within the control sections. The authors surveyed, measured, and recorded at set intervals the rutting, profiles, and various kinds of cracks. Deflection measurements were recorded during the three stages of construction: before milling of the existing overlay, after milling, and after construction of the new overlay. These data summarize the structural response of the different pavement systems. Backcalculating the stiffness for the different layers of each test section shows the stiffness variation under repeated traffic loads. The rut depth survey verified that a pavement that has had rutting problems in the original pavement continues to have rutting problems. It also showed that the flexible base layer does not prevent rutting, even though it can prevent some reflective cracking. After analyzing the condition surveys, the authors concluded that all transverse cracks in the rigid sections are reflective cracks, while the transverse cracks in the flexible sections are a combination of fatigue-related cracking and reflective cracking. The open grade mix (or Arkansas mix) performed better than the other rehabilitation methods. The pavement roughness was summarized using the International Roughness Index (IRI) and Pavement Serviceability Index (PSI). The crack and seat method showed the poorest serviceability after 2 years of traffic loading and exposure to the environment. This and other performance information will be used to identify optimal rehabilitation strategies for the rigid and flexible pavements throughout the Lufkin District.

Chou, Y. 1984. "Asphalt Overlay Design for Airfield Pavements", *Proceedings*: Association of Asphalt Paving Technologists.". Association of Asphalt Paving Technologists. St Paul, MN.

This paper describes how overlay pavements are designed to increase the load-carrying capacity (strength) of the existing pavement. The basis for design is to provide a layer or layers of material on the existing pavement that will result in a layered system, which yields the predicted performance of a new pavement if constructed on the same foundation as the existing pavement. Strengthening is required when heavier aircraft are introduced or when a pavement is no longer capable of supporting the aircraft loads for which it was designed. Two types of overlay

pavement are considered-rigid and nonrigid. The procedures described in this paper are limited to nonrigid overlays to strengthen existing rigid or flexible pavements. Nonrigid overlays include both flexible (nonstabilized base and asphaltic concrete (AC) wearing course) and all-AC for strengthening existing jointed concrete (JC) or jointed reinforced concrete (JRC) pavements. The paper discusses how specific procedures must be followed to determine if a pavement requires strengthening or other rehabilitation.

Clyne, T., Worel, B., and Marasteanu, M. 2006. "Low Temperature Cracking Performance at MnROAD.", *Proceedings*: Cold Regions Engineering 2006: Current Practice in Cold Regions Engineering (July 2006). American Society of Civil Engineers. Orono, ME.

The Minnesota Road Research Project (MnROAD) was constructed in 1990-1993 as a full-scale pavement testing facility. Several different cells were built with various materials, mix designs, and structural designs. Two different asphalt binders were used during the original construction: PG 58-28 and PG 64-22. The sections have all shown various degrees of low temperature cracking. In general the cells with stiffer binder (PG 64-22) experienced a higher No. and greater severity of thermal cracks than those with the softer binder. The ride quality of the pavements has been adversely affected by the deterioration of the low temperature cracks. In 1999 three cells were reconstructed on the Low Vol. Road as a study specifically examining low temperature cracking. These sections were designed using the exact same Superpave mix design except for the asphalt binder type, which differed at the low temperature performance grade. The performance grades for Cells 33, 34, and 35 were PG 58-28, 58-34, and 58-40 respectively. After several years in service these sections have begun to show marked differences in performance. Cell 35 has shown the most cracking, even though it has the softest grade at -40. The cracks on Cell 35 do not look like typical thermal cracks, while Cell 33 exhibits the expected typical thermal cracks. Cell 34 had virtually no distress after six years.

Colorado Department of Transportation. 2007. *2008 Pavement Design Manual*. Colorado Department of Transportation, Denver, CO.

The purpose of this manual is to provide the Colorado Department of Transportation (CDOT) and consultant pavement designers with a uniform and detailed procedure for designing pavements on CDOT projects. This manual should be used after July 1, 2007. The manual is organized in the following chapters: (1) Pavement Design Information; (2) Subgrade and Base Materials; (3) Principles of Design for Flexible Pavement; (4) Principles of Design for Rigid Pavement; (5) Principles of Design for Pavement Rehabilitation with Flexible Overlays; (6) Principles of Design for Pavement Rehabilitation with Rigid Overlay; (7) Rehabilitation of Portland Cement Concrete Pavement; (8) Pavement Intersections; (9) Principles of Design for Rigid Pavement Intersections; (10) Pavement Type Selection and Life Cycle Cost Analysis; and (11) Pavement Justification Report.

Cortez, E R., Chen, D., Yang, W and Petros, K. 2007. "Analysis of Permanent Deformation in Flexible Pavements Built with Various Subgrade Soils at Various Moisture Conditions.". *Proceedings*: Transportation Research Board 86th Annual Meeting (January 2007). Transportation Research Board. Washington, DC.

To develop subgrade failure criteria and performance prediction models that consider the subgrade soil type and moisture condition, 12 sets of full-scale flexible pavement test sections were built inside the Frost Effects Research Facility of the U.S. Army Corps of Engineers. The pavement materials and layer thickness were kept constant, but the subgrade soil type and moisture condition varied from test section to test section. The test sections were subjected to accelerate traffic by means of a heavy vehicle simulator. Sensors were embedded in the test sections to measure deformation and stress. Although sensors existed in the vertical, longitudinal, and transversal directions, only vertical results are included in this paper owing to space limitations. Surface rutting was monitored by means of a laser profilometer. The temperature was artificially kept constant at room temperature. The subgrade soil moisture was kept constant during the testing by means of a closed basin system. Although intuitively, one can expect moisture above the conventional ?optimum? to play a weakening role, the experimental data suggest that this is not always the case. The experimental data support the formulation of quantitative relationships between subgrade moisture content and performance for each of several subgrade soil types. Consideration of these moisture effects for various subgrade soils can be used to improve the permanent deformation models currently built into the NCHRP 1-37A Design Guide. This paper contains an analysis of permanent deformation based on experimental results. Other papers are planned to cover other aspects, such as stress?strain relationships, load-damage, and performance models.

Dai, S., Boerner, D and Isackson, C. 2007. "Failure Analysis of Flexible Pavement Section on Mn/Road.", *Proceedings*: Transportation Research Board 86th Annual Meeting (January 2007). Transportation Research Board. Washington, DC.

On July of 1997, one of the Minnesota Road Research project (Mn/Road) test sections (cell28) exhibited significant rutting after receiving approximately 59,000 ESALs. The rut depth was as deep as 25.4 mm (1 inch). A comprehensive forensic excavation of the section, laboratory study and analysis were therefore conducted to investigate the failure mechanism. Historical rutting measurements showed that rutting of this section began almost immediately after the section opened to traffic. Rut depth had been increasing since then and reach approximate 1 inch in the summer of 1997. From the forensic excavation, the cross-sections revealed that the primary rutting of the pavement was due to permanent deformation of the aggregate base, which indicated base failure. After the forensic excavation, laboratory tests were performed on the aggregate base material to obtain shear strength using conventional triaxial experiments. Mohr-Coulomb failure criteria were established. WESLEA was used to calculate stresses in the base and the calculated stresses were compared with the strength. The results indicated that the stresses in the middle of the base exceeded the material strength, which caused bearing capacity failure in the base. This analysis suggests that a minimum asphalt pavement thickness should be established in a mechanistic-empirical design method to prevent bearing capacity failure of subsurface layers. Furthermore, the strain measurement on the bottom of the pavement was examined.

Dai, S., Skok, G., Westover, T M., Labuz, J F., and Lukanen, E O. 2008. "Pavement Rehabilitation Selection.", Report No. MN/RC 2008-06. Minnesota Department of Transportation. Minneapolis, Minnesota.

The objective of the project was to outline best practices for the selection of asphalt pavement recycling techniques from the many choices that are available. The report specifically examines cold-in-place recycling (CIR), plain full depth reclamation (FDR), and mill & overlay (M&O). Interviews, surveys, and site visits were conducted at both Minnesota Department of Transportation (Mn/DOT) districts and counties, where relevant rehabilitation information was supplied on over 120 projects. A database was constructed to organize the details of these projects, and the parameters in the database included (1) cracking, (2) ride, (3) rutting, (4) age, and (5) traffic Vol.. From studying the existing rehabilitation projects in the State, Ride Quality Index (RQI) and Surface Rating (SR) were selected as the descriptors of pavement surface condition. A decision procedure based on the analysis of all available projects was developed. The decision procedure included (1) consideration of road geometrics; (2) pavement condition survey; and (3) structural adequacy evaluation. Furthermore, a step-by-step checklist was developed to provide local engineers with a simple and useful tool to follow the decision procedures. The procedure includes selection of rehabilitation method, pavement thickness design, materials mixture design, and construction.

Donovan, P., and Tutumluer, E. 2009. "Use of Falling-Weight Deflectometer Testing to Determine Relative Damage in Asphalt Pavement Unbound Aggregate Layers.". *Proceedings*: Transportation Research Board 88th Annual Meeting (January 2009). Transportation Research Board. Washington, DC.

Falling weight deflectometer (FWD) testing is a nondestructive pavement structural evaluation technique routinely performed on highway and airfield pavements to estimate pavement layer properties from measured deflection basins. This paper presents a methodology based on analyzing FWD test data between the trafficked and non-trafficked lanes to determine degradation and rutting potential of asphalt pavement unbound aggregate layers in comparison to the subgrade damage. The validity of the approach is demonstrated by analyzing the heavy weight deflectometer (HWD) data obtained from the Federal Aviation Administration's National Airport Pavement Test Facility (NAPTF) flexible airport pavement test sections built with substantially thick unbound aggregate base/subbase courses. The modified Base Damage Index and Base Curvature Index defined from HWD pavement deflection basins were used to determine relative base to subgrade damage, which clearly showed evidence of the increased base damage induced in the NAPTF airport pavement layers during trafficking partly due to the applied gear load wander. This was in accordance with both the individual pavement layer recovered and unrecovered (inelastic or residual) deformation trends identified from analyzing the multi-depth deflectometer data collected during trafficking and the post traffic forensic analysis results, which indicated that a majority of the permanent deformation occurred in the unbound aggregate layers and not in the subgrade. The methodology presented for the detailed analyses of the FWD (or HWD) test data between trafficked and non-trafficked lanes can be effectively used in asphalt pavements to detect unbound aggregate layer deterioration and its pavement damage potential.

Emery, J J. 2006. "Evaluation and Mitigation of Asphalt Pavement Top-Down Cracking.". *Proceedings*: Annual Conference & Exhibition of the Transportation Association of Canada (September 2006). Transportation Association of Canada. Charlottetown, Canada.

The purpose of this paper is to discuss different ways to stem top-down cracking in pavement in order to achieve satisfactory overall pavement performance. To do this, enhanced asphalt materials and construction technology along with the use of stone mastic asphalt and polymer modified asphalt binders have been shown to be very effective on a life-cycle performance and cost basis. It is important hat top-down cracking, which is a rather complex surface distress mode related to tensile and shear stresses associated with non-uniform tire stresses, interlayer slippage, thermal stresses, stiffness gradients, construction problems such as segregation, and premature asphalt binder age hardening, is mitigated in order to achieve satisfactory overall pavement performance.

Epps, A. 2000. "Design and Analysis System for Thermal Cracking in Asphalt Concrete.". Journal of Transportation Engineering. Vol. 126, No. 4, pp 300-307 (July-August 2000). American Society of Civil Engineers. Reston, VA.

A reliability based mix design and analysis system for thermal cracking in asphalt concrete pavements is described and demonstrated in this paper. Asphalt-aggregate mixture performance in terms of low temperature cracking and thermal fatigue is assessed independently for each type of cracking in the specified environment. Reliability factors are introduced into this assessment to account for the level of risk assumed by the engineer and the inherent variability in the estimates of mixture resistance to induced thermal stresses and the environmental demand in terms of conditions promoting either type of thermal cracking. Mixture resistance is measured in two laboratory tests, and environmental demand is estimated from predicted surface pavement temperatures. The paper concludes with an example of the analysis described, comparing unmodified and crumb rubber modified mixture performance at different levels of reliability in a high elevation desert climate. The system presented is recommended as a mix design and analysis tool for evaluating unmodified and modified asphalt-aggregate mixture performance in harsh climates where thermal cracking is of concern.

Federal Aviation Administration (FAA). 1989. *Airport Design*. Advisory Circular 150/5300-13, including all changes. Federal Aviation Administration, Washington, DC.

This advisory circular contains the FAA's standards and recommendations for airport design.

Federal Aviation Administration (FAA). 1995. *Airport Pavement Design and Evaluation*. Advisory Circular 150/5320-6D, including all changes. Federal Aviation Administration, Washington, DC.

This advisory circular provides guidance to the public for the design and evaluation of pavements at civil airports. (This is the document that will be replaced with the current draft.)

Federal Aviation Administration (FAA). 2004. *Use of Nondestructive Testing in the Evaluation of Airport Pavements*. Advisory Circular 150/5370-11A. Federal Aviation Administration, Washington, DC.

This advisory circular (AC) focuses on nondestructive testing (NDT) equipment that measures pavement surface deflections after applying a static or dynamic load to the pavement. It also briefly introduces other types of nondestructive measuring equipment to illustrate how supplementing NDT data with other test data may improve the quality and reliability of the pavement evaluation. This AC provides guidance and recommendations on datacollection equipment and methods of data analysis that are used to conduct NDT; however, other methods, techniques, and variations of those outlined here may be used provided the appropriate local Federal Aviation Administration (FAA) Airports Office approves them.

Federal Aviation Administration (FAA). 2005. *Standard Naming Convention for Aircraft Landing Gear Configurations.* Order 5300.7. Federal Aviation Administration, Washington, DC.

This Order establishes a standard convention for naming and characterizing aircraft landing gear configurations. Although this order is primarily directed at fixed wing airplanes, it is applicable to any aircraft using wheels for landing purposes.This Order impacts divisions in the Offices of Planning and Programming, Airport Safety and Standards, Air Traffic, Airway Facilities, and Flight Standards Services; the regional Airports, Air Traffic, Airway Facilities, and Flight Standards Divisions; and Airport District and Field Offices. It will also affect organizations and individuals external to the Federal Aviation Administration (FAA). A standardized naming convention will allow uniformity and consistency among Federal agencies and external entities when naming aircraft gear configurations. Pilots and airport operators will no longer need to learn multiple naming systems and will be able to use common aircraft landing gear names at all military and commercial facilities.

Federal Aviation Administration (FAA). 2006. *Standard Method of Reporting Airport Pavement Strength-PCN.* Advisory Circular 150/5335-5A. Federal Aviation Administration, Washington, DC.

This advisory circular provides guidance for using the standardized International Civil Aviation Organization (ICAO) method to report airport pavement strength. The standardized method is known as the Aircraft Classification No. – Pavement Classification No. (ACN-PCN) method.The International Civil Aviation Organization (ICAO) requires member countries to report pavement strength information for a variety of purposes. The ACN-PCN method has been developed and adopted as an international standard and has facilitated the exchange of pavement strength rating information. This AC provides specific guidance on how to report airport pavement strength using the standardized method.

Federal Aviation Administration (FAA). 2007. *Standards for Specifying Construction of Airports*. Advisory Circular 150/5370-10C. Federal Aviation Administration, Washington, DC.

This advisory circular provides standards for the construction of airports. Items covered in this AC include general provisions, earthwork, flexible base courses, rigid base courses, flexible surface courses, rigid pavement, miscellaneous, fencing, drainage, turfing, and lighting installation.The Federal Aviation Administration (FAA) recommends the guidelines and standards in this AC for materials and methods used in the construction of airports. This AC does not constitute a regulation and in general is not mandatory. However, use of these guidelines is mandatory for airport construction funded under the Airport Improvement Program (AIP) or Passenger Facility Charge (PFC) Program. Mandatory terms such as "must" used herein apply only to those who undertake construction projects using AIP or PFC funds.

Federal Aviation Administration (FAA). Draft. *Airport Pavement Design and Evaluation*. Advisory Circular 150/5320-6E. Federal Aviation Administration, Washington, DC.

This advisory circular provides guidance to the public on the design and evaluation of pavements at civil airports. (This is the draft document to be reviewed.)

Finn, F., and Monismith, C. 1984. "Asphalt Overlay Design Procedures.", NCHRP Synthesis of Highway Practice No. 116 (December 1984). Transportation Research Board. Washington, DC.

This synthesis will be of interest to pavement designers and others concerned with the design of asphalt concrete overlays. Information is presented on reasons for overlaying a pavement and on the various methods available for design of an asphalt overlay. A pavement overlay may be required because of inadequate ride quality, excessive pavement distress, reduced friction between tire and pavement, high user costs, or inadequate structural capacity for planned use. This report of the Transportation Research Board discusses the current methods used for designing asphalt concrete overlays with emphasis on deflection-based and analytical procedures.

Flintsch, G., Diefenderfer, B K., and Nunez, O. 2008. "Composite Pavement Systems: Synthesis of Design and Construction Practices", Federal Highway Administration and Virginia Department of Transportation Report No. FHWA/VTRC 09-CR2, Richmond, Virginia.

Composite pavement systems have shown the potential for becoming a cost-effective pavement alternative for highways with high and heavy traffic Vol.s, especially in Europe. This study investigated the design and performance of composite pavement structures composed of a flexible layer (top-most layer) over a rigid base. The report compiles (1) a literature review of composite pavement systems in the U.S. and worldwide; (2) an evaluation of the state-of-the-practice in the U.S. obtained using a survey; (3) an investigation of technical aspects of various alternative composite pavement systems designed using available methodologies and mechanistic-empirical pavement distress models (fatigue, rutting, and reflective cracking); and (4) a preliminary life cycle cost analysis (LCCA) to study the feasibility of the most promising composite pavement systems. Composite pavements, when compared to traditional flexible or rigid pavements, have the potential to become a cost-effective alternative because they may provide better levels of performance, both structurally and functionally, than the traditional flexible and

rigid pavement designs. Therefore, they can be viable options for high Vol. traffic corridors. Countries, such as the U.K. and Spain, which have used composite pavement systems in their main road networks, have reported positive experiences in terms of functional and structural performance. Composite pavement structures can provide long-life pavements that offer good serviceability levels and rapid, cost-effective maintenance operations, which are highly desired, especially for high-Vol., high-priority corridors. Composite pavements mitigate various structural and functional problems that typical flexible or rigid pavements tend to present, such as hot-mix asphalt (HMA) fatigue cracking, subgrade rutting, portland cement concrete (PCC) erosion, and PCC loss of friction, among others. At the same time, though, composite systems are potentially more prone to other distresses, such as reflective cracking and rutting within the HMA layer. Premium HMA surfaces and/or reflective cracking mitigation techniques may be required to mitigate these potential problems. At the economic level, the results of the deterministic agency-cost LCCA suggest that the use of a composite pavement with a cement-treated base (CTB) results in a cost-effective alternative for a typical interstate traffic scenario. Alternatively, a composite pavement with a continuously reinforced concrete pavement (CRCP) base may become more cost-effective for very high Vol.s of traffic. Further, in addition to savings in agency cost, road user cost savings could also be important, especially for the HMA over CRCP composite pavement option because it would not require any lengthy rehabilitation actions, as is the case for the typical flexible and rigid pavements.

Freitas, E., Pereira, P., and Picado-Santos, L. 2003. "Assessment of Top-Down Cracking Causes in Asphalt Pavements.". *Proceedings:* Maintenance and Rehabilitation of Pavements and Technological Control (July 2003), Guimaraes, Portugal.

Cracks observed at the surface may have several origins and causes. A crack may either begin at the bottom or at the top of an asphalt layer. Cracking originated at the top of asphalt layers (top-down cracking) observed in temperate-climate countries is a degradation mechanism which has not been fully researched. In order to access this "new" degradation in Portugal, a sampling plan has been set in a high thickness pavement, including the extraction of cylindrical cores and slabs. Cracks were carefully identified and several laboratory tests were performed on grading, bitumen content and air voids, fatigue strength and stiffness. Pavement bearing capacity was also measured. The assessment and comparison of the results show, in a first approach, that all cracks are surface-originated and top layers contains excessive fine aggregates and voids. A second approach concerning modulus, fatigue strength and bearing capacity is less conclusive, which leaves a long way to go. Nevertheless, it seems that construction quality continues to be the most probable cause of top-down cracking.

Fwa, T. 1991. "Remaining-Life Consideration in Pavement Overlay Design.". Journal of Transportation Engineering. Vol. 117, No. 6, pp. 585-601 (November/December 1991). American Society of Civil Engineers, Reston, VA

The thickness deficiency-based approach is a widely used procedure for pavement overlay design. There is a need to incorporate a remaining-life consideration into this approach. The traditional thickness deficiency design equations do not include this consideration. The 1986 AASHTO procedure introduced a remaining-life factor in its overlay design equations. While its concept of remaining life consideration is fundamentally correct, AASHTO's equations produce inconsistencies in overlay designs due to an inadequacy in the formula used for calculating the remaining-life factor. A new expression for the remaining-life factor is derived in this paper. It is shown that the traditional and the AASHTO design equations are special cases of this new expression. The salient features of the derived expression are described. A design procedure based on the new equations is proposed. Numerical examples are presented to demonstrate the applications of the proposed procedure.

Galal, K A., Diefenderfer, B K., and Alam, J. 2007. "Determination by the Falling Weight Deflectometer of the In-Situ Subgrade Resilient Modulus and Effective Structural No. for I-77 in Virginia.". Report No. VTRC 07-R1. Virginia Department of Transportation, Richmond, VA.

The Virginia Department of Transportation (VDOT) manages approximately 27,000 lane-miles of interstate and primary roadways, of which interstate pavements comprise approximately 5,000 lane-miles. These pavements consist of flexible, rigid, and composite pavements. Virginia's pavements are managed using an asset management system (AMS) that incorporates a pavement management system (PMS), which aids VDOT in determining the funding required for various levels of pavement maintenance (i.e., preventive maintenance, rehabilitation, or reconstruction activities). As part of VDOT's AMS (PMS) system, a large portion of the interstate pavement system was visually rated annually to determine a condition index based on load-related and non-load related distresses. Recently, VDOT began using an automated distress collection procedure for this task that incorporates the measurement of pavement condition data such as the international roughness index, rutting in both wheel paths, cracking, and No. of patches and potholes. However, there is no current protocol to assess the structural capacity of the pavement on a network level and thus determine the remaining load-carrying capacity (service life) of a pavement structure. Many state departments of transportation use the falling weight deflectometer (FWD) to collect pavement deflection data at the project or network level. The analysis of these data provides the effective roadway resilient modulus, the effective in-situ structural No., the pavement layer moduli, the effective in-situ layer coefficient, or all of these parameters. This process is accomplished through a backcalculation procedure using routines that use the FWD deflection data, known as the deflection basins; the FWD load history; and the pavement layer thicknesses as inputs to this procedure. VDOT currently uses the 1993 AASHTO Guide for Design of Pavement Structures for the design of its new or rehabilitated pavement structures. As VDOT moves to implement the proposed Mechanistic-Empirical Pavement Design Guide (MEPDG), characterizing existing pavement conditions, including the resilient modulus of the subgrade, is necessary to ensure optimum designs. This study collected the in-situ layer conditions, the in-situ structural No., and the in-situ subgrade resilient modulus and deflection data for Virginia's I-77 using FWD network level testing. This testing was found to be a viable tool to classify existing structural network conditions. The information can be used by pavement designers and pavement management engineers to address network needs in terms of rehabilitation strategies and fund management. The study recommends that structural testing on the network level be conducted for all interstate and primary routes in Virginia and used in conjunction with VDOT's AMS. Obtaining such data through traditional destructive testing requires coring and boring operations that incur traffic control, equipment, and personnel costs. To conduct such operations at the network level would cost VDOT approximately $5.06 million annually. The costs for the FWD network level testing used in this study are estimated at $83,200 annually, resulting in an annual cost savings for VDOT of almost $5 million.

Galal, K., Diefenderfer, B K., Alam, J., Tate, T.m and Wells, M. 2006. "FWD Determination of the In-Situ Subgrade Resilient Modulus and Effective Structural No. of Virginia's Interstate Network.". *Proceedings*: Airfield and Highway Pavements (April-May, 2006), American Society of Civil Engineers. Atlanta, GA.

The Virginia Department of Transportation (VDOT) manages approximately 25,000 lane-miles of interstate and primary roadway; of which approximately 2000 lane-miles is comprised of interstate pavements. These pavements consist of flexible, rigid, and composite pavements. Virginia?s pavements are managed utilizing a pavement management system (PMS) which aides VDOT in determining the amount of funding required for various levels of maintenance (i.e., preventative maintenance, rehabilitation, or reconstruction). As part of VDOT's PMS system, a portion of the interstate pavements are visually rated on an annual basis to determine a condition index based on load related and non-load related distresses. Recently, VDOT began utilizing an automated distress collection procedure for this task will incorporate the measurement of pavement condition data such as the International Roughness Index (IRI), rutting on both wheel paths, and a cracking index. However, there is no current method to asses the structural capacity of the pavement and thus determine the remaining load carrying capability of a pavement structure. Many state departments of transportation use the Falling Weight Deflectometer (FWD) equipment to collect pavement deflection data at the project or the network levels. The analysis of this data provides the effective roadway resilient modulus, the effective in-situ structural No., the pavement layer moduli, the effective in-situ layer coefficient or all of these parameters. This process is accomplished through the backcalculation procedure utilizing numerous developed in-house or commercial backcalculation routines that utilize the FWD deflection data, know as the deflection basin, the FWD load history, and using the pavement layer thicknesses as an input to this procedure. VDOT currently uses the 1993 AASHTO Guide for the Design of Pavement Structures for the design of its new or rehabilitated pavement structures. The AASHTO guide is based on empirical relationships that were developed during the 1960s as a result of the AASHTO road test. As VDOT moves to implement the Mechanistic-Empirical Pavement Design Guide (MEPDG), characterizing existing pavement conditions, including the resilient modulus of the subgrade, is an enhancement of the input level compared to using the MEPDG's default values. Therefore, there is an urgent need to collect the in-situ structural No. subgrade resilient modulus and deflection along VDOT's interstate system and to build the associated database for future implementation of the MEPDG. In addition, this information will allow for better management of VDOT resources on the network level for the interstate system. This paper describes the results of testing Interstate 77 and also describes the details and experiences during the process.

Garg, N., Guo, E., and McQueen, R.. ***Operational Life of Airport Pavements.*** **DOT/FAA/AR-04/46. Federal Aviation Administration, Washington, DC.**

The objective of the study was to determine whether the Federal Aviation Administration (FAA) standards used to determine the appropriate thickness for hot mix asphalt and concrete airfield pavements are in accordance with the FAA standard for a 20-year life requirement. FAA airport pavement design standards, Advisory Circular (AC) 150/5320-6D (1995), including changes 1, 2, and 3 (2004), and related references, including some unpublished FAA technical reports and full-scale test results, were reviewed. The effects of many parameters, directly used in the failure model and indirectly used through the pavement response model, on the pavement structural life were analyzed. A sensitivity analysis of parameters on pavement structural life was used to quantitatively evaluate the effects of the most important parameters in different airport pavement design procedures. Some full-scale test results were used to support the findings in the analysis. Much of the airport pavement surveyed data collected from previous FAA projects, including some unpublished ones, was also reviewed. A portion of that data was used in this report for the analysis. Based on the surveyed data, it was found that the average Structural Condition Index of both hot mix asphalt and Portland cement concrete pavements in all age groups is higher than 80. Based on the definition adopted in this report, the airport pavements designed following AC 150/5320-6D have sufficient thickness to provide a 20-year structural life.

Gopalakrishnan, K., and Thompson, M R. 2007. "Use of Nondestructive Test Deflection Data for Predicting Airport Pavement Performance.", Journal of Transportation Engineering. Vol. 133 No. 6 (June 2007), pp 389-395. American Society of Civil Engineers. Reston, VA.

Surface deflections using nondestructive tests (NDTs) were measured prior to and throughout the traffic testing at the U.S. Federal Aviation Administration's National Airport Pavement Test Facility (NAPTF). The first series of traffic tests involved repeated loading of six-wheel Boeing 777 and four-wheel Boeing 747 test gears on two different lanes until the pavements were deemed failed. The NAPTF structural failure criterion was defined as at least 25.4 mm (1 in.) surface upheaval adjacent to the traffic lane. A predetermined wander sequence was applied. Two low-strength subgrade and two medium-strength subgrade flexible pavement test sections were tested. Transverse surface profiles were measured periodically to monitor the progression of permanent deformation in pavements. Deflection basin parameters derived from NDT surface deflections were related to pavement rutting performance. An airport pavement functional failure criterion, defined in terms of No. of traffic load repetitions to reach specific rut depth levels, was used in characterizing the structural response-performance relations.

Guada, I., Signore, J., Tsai, B., and Monismith, C L. 2007. "Reflective Cracking Study: First-level Report on Laboratory Shear Testing.". Report No. UCPRC-RR-2006-11. California Department of Transportation. Sacramento, CA.

This report is one in a series of first-level analysis reports describing the results of Heavy Vehicle Simulator (HVS) testing on a full-scale experiment designed to validate California Department of Transportation (Caltrans) overlay strategies for rehabilitating cracked asphalt concrete. The report describes the results of the laboratory load shear tests on mixes used as overlays in the experiment and summarizes the results on binder tests. It details the shear testing that was conducted and includes the effects of mix temperature, air-void content, aging, mixing and compaction, aggregate gradation effects, and shear stress level.

Haider, S., Chatti, K., Buch, N., Lyles, R., Pulipaka, A., and Gililand, D. 2007. "Effect of Design and Site Factors on the Long-Term Performance of Flexible Pavements.". Journal or Performance of Constructed Facilities. Vol. 21 No. 4 (July/August 2007), pp 283-292. American Society of Civil Engineers. Reston, VA.

Results are presented from a study to evaluate the relative influence of design and site factors on the performance of in-service flexible pavements. The data are from the SPS-1 experiment of the Long-Term Pavement Performance program. This experiment was designed to investigate the effects of HMA surface layer thickness, base type, base thickness, and drainage on the performance of new flexible pavements constructed in different site conditions (subgrade type and climate). Base type was found to be the most critical design factor affecting fatigue cracking, roughness (IRI), and longitudinal cracking (wheel path). The best performance was shown by pavement sections with asphalt treated bases (ATB). This effect should be interpreted in light of the fact that an ATB effectively means a thicker HMA layer. Drainage and base type, when combined, also play an important role in improving performance, especially in terms of fatigue and longitudinal cracking. Base thickness has only secondary effects on performance, mainly in the case of roughness and rutting. In addition, climatic conditions were found to have a significant effect on flexible pavement performance. Wheel path longitudinal cracking and transverse cracking seem to be associated with a wet-freeze environment, while nonwheel path longitudinal cracking seems to be dominant in a freeze climate. In general, pavements built on fine-grained soils have shown the worst performance, especially in terms of roughness. Although most of the findings from this study support the existing understanding of pavement performance, they also provide an overview of the interactions between design and site factors and new insights for achieving better long-term pavement performance.

Haifang, W., and Titi, H. 2005. "Guidelines for the Surface Preparation/Rehabilitation of Existing Concrete and Asphaltic Pavements Prior to an Asphaltic Concrete Overlay.", Report No. WHRP 05-10. Wisconsin Department of Transportation. Madison, WI.

A large percentage of the asphaltic paving projects performed in Wisconsin are asphaltic overlays of existing concrete or asphaltic pavements. Due to varying performance of overlay, a standard set of guidelines is needed to determine the amount of surface preparation which provides a consistency along with more accurate and stable project budgets for this type of work. Literature review of Wisconsin Department of Transportation (WisDOT) and national practices of pre-overlay repair of existing concrete and asphaltic pavements was conducted. Previous asphalt overlay projects were reviewed and overlay performance was analyzed. In addition, three overlay projects during 2004 construction season were studied in the field. For asphalt overlay of existing concrete pavements, it was found that overlays with doweled concrete base patching performed best, followed by non-doweled concrete base patching and then asphaltic base patching. Partial depth repair is needed to fix the medium severity transverse cracks and longitudinal/transverse distressed joints in existing concrete pavement. A minimum of 3 in., practically 3 1/2 in., overlay thickness was found to be able to mitigate reflective cracking in overlay. All high-severity joints/cracks/patches should be repaired. The current International Roughness Index (IRI) in overlay was highly correlated with initial IRI of overlay, indicating the importance of profile index. The roughness prediction model used in the NCHRP 1-37A 2002 design guide was calibrated with locally available data. For asphalt overlay of existing asphalt pavements, block cracking in existing asphalt pavement does not adversely affect the overlay when milling is used. Existing asphalt pavement with extensive alligator cracking should be pulverized to prevent the reflection of underlying alligator cracking. Milling the existing asphalt pavement can not eliminate the reflection of transverse cracking in existing asphalt pavement. The ratio of overlay thickness to milling depth should be kept a minimum of 3 to prevent longitudinal cracking from re-occurring in overlay. A set of guidelines was developed to be included in the Facility Development Manual and Construction and Material Manual.

Hammitt, G. 1974. *Comparative Performance of Structural Layers in Pavement Systems, Vol. III: Design and Construction of MESL*. FAA-RD-73-198, Vol. 3; ADA-005-893. Army Engineer Waterway Experiment Station, Vicksburg, MS.

This report describes design and construction procedures for membrane-encapsulated soil layers in airport pavement systems based on analyses of results of recent tests of full-scale accelerated traffic test sections. Included are descriptions of material and equipment requirements and recommended test methods. The procedures are applicable of both rigid and flexible airport pavement systems. Recent material developments and subsequent testing have demonstrated the structural integrity of MESL-type construction. It is believed that substantial savings can be realized using MESL's in airport pavements because of less strict material quality requirements and lower maintenance requirements due to the waterproofing protection provided by the MESL.

Harmelink, D., Shuler, S., and Aschenbrener, T. 2008. "Top-Down Cracking in Asphalt Pavements: Causes, Effects, and Cures.". Journal of Transportation Engineering Vol. 132 No. 1 (January 2008) pp 1-6. American Society of Civil Engineers. Reston, VA.

A section of I-25 north of Denver was rehabilitated by cold milling the existing surface to a depth of 3 inches and replacing with new hot mix asphalt. The contractor received bonuses for quality and smoothness and the mixture passed both Hamburg and French LCPC rut tests. Within 1 year longitudinal cracks appeared in the pavement surface. The cracking appeared in the driving lanes of both the north and southbound directions. The severity of the cracking ranged from low to high. An investigation concluded that cracking on the project was surface initiated and caused by a No. of factors. One factor was segregation observed at the bottom of the upper pavement lift that was not visible on the surface. After this so-called ?top-down cracking? was discovered on this first project, other pavements began manifesting similar traits. Therefore, a statewide evaluation was conducted to determine the extent of this distress in other pavements. As a result of this study 28 sites were evaluated and of these 18 contained top-down cracking. Based on this finding a change in the mixture design process was implemented to allow for increased asphalt content in hope that the richer mixtures would not be as prone to segregate. In addition, a segregation task force was created with industry to develop a specification for segregation. This task force is in the process of developing a specification to identify subsurface segregation with the elimination of top-down cracking as the goal. Paving equipment manufacturers have also identified areas within the laydown equipment that can promote segregation. As a result manufacturers have taken the initiative to develop an antisegregation retrofit for some laydown machines. As a result of these efforts top-down cracking has been generally eliminated or, at least, greatly reduced.

Hicks, R G., Zhou, H., and Connor, B. 1988 "Development of an Improved Overlay Design Procedure for the State of Alaska. Vol. III: Field Manual.". Report No. FHWA-AK-RD-88-06B. Alaska Department of Transportation and Public Facilities; Federal Highway Administration, Fairbanks, AL.

This manual describes a mechanistic overlay design procedure for use in the state of Alaska. This procedure is based upon the fundamental characteristics of pavement layer properties and uses a linear elastic program ELSYM5 to determine strains at critical locations. The tensile strain at the bottom of the surface layer is used to control fatigue, while the compressive strain on top of the subgrade is used to control rutting. Failure criteria developed by the Asphalt Institute are used to determine pavement life. Seasonal effects on pavement layer properties and traffic are also considered. Miner's rule is used to address total pavement damage for all seasons. A comprehensive example illustrating the design procedure is presented.

Him, S., Ceylan, H., and Gopalakrishnan, K. 2007. "Effect of M-E Design Guide Inputs on Flexible Pavement Performance Predictions.". Road Materials and Pavement Design Vol. 8 No. 3 pp 375-397, Hermes Science Publications, France.

This paper describes a study which focused on assessing the level of impact Mechanistic-Empirical Pavement Design Guide (MEPDG) input parameters relating to asphalt concrete, traffic, and climate have on specific damage models for typical flexible pavements. Two existing flexible pavements in Iowa were examined using the MEPDG software. For each pavement structure, 20 individual inputs were evaluated by studying the effect of each input on five different performance measures: 1) longitudinal cracking, 2) alligator cracking, 3) transverse cracking, 4) rutting, and 5) roughness. It was found that most input parameters influenced the predicted longitudinal cracking and total rutting, while alligator cracking, transverse cracking, and roughness did not show sensitivity to most input parameters. Researchers will next focus on comparing the predicted measures against the recorded pavement distress in the Pavement Management Information System (PMIS) database maintained by the Iowa Department of Transportation

Huurman, R., and Molenaar, A. 2006. "Permanent Deformation in Flexible Pavements with Unbound Base Courses.". Journal of the Transportation Research Board No. 1952 pp 31-38. Transportation Research Board. Washington, DC.

Permanent deformation of the unbound base and subbase layer of a flexible pavement is one of the damage types that can require extensive maintenance. To be able to predict the development of permanent deformation, two options can be considered. The first is the development of models that allow the prediction of permanent deformation as a function of the applied stresses, the No. of load repetitions, and the material characteristics and moisture regime. The second is the determination of stress conditions at which no significant permanent deformations develop. Such a stress level is, of course, dependent on the material characteristics, the degree of compaction, and the moisture content. The development of such a ?shake down limit? for unbound base materials made of mixtures of recycled concrete and masonry is the topic of this paper. It has been found that the allowable ratio of applied vertical stress at a given confining stress level and the vertical stress at failure at that same confinement level depend to some extent on the composition of the mixture of crushed concrete and crushed masonry but mainly on the degree of compaction; ratios between 0.45 and 0.61 were found. Furthermore, it was possible to estimate the parameters controlling the resilient, as well as the failure behavior from physical parameters such as gradation, mixture composition, angularity of the particles, and the degree of compaction. The relationships indicate that performance-related specifications for the materials as investigated still can be based on well-known parameters that can easily be assessed in a short period of time and at a low cost.

Jameson, G. 2006. "Austroads Asphalt Overlay Fatigue Life Prediction.". Research into Practice: 22nd ARRB Conference, Australian Road Research Board (October/November, 2006), Canberra, Australia

The purpose of this paper is to discuss the development of procedures used in Pavement Rehabilitation. A Guide to the Design of Rehabilitation Treatments for Road Pavements This publication covers the selection and design of pavement rehabilitation procedures, including the design of structural overlays on flexible pavements. The procedure to predict the fatigue life of asphalt overlays from measured surface curvatures using a simple design chart approach is used in the Guide and is applicable to curvatures measured with a range of devices, FWD, deflectograph and Benkelman Beam. The method is applicable to curvatures measured on pavements which do not contain cemented materials. This new Austroads procedure is a substantial improvement over the previous procedure as it is more consistent with the design procedures for new pavements and applicable to a wide range of deflection devices.

Jones, D., Monismith, C L., and Harvey, J T. 2007. "Reflective Cracking Study: Summary Report.". Report No. UCPRC-RR-2007-01. California Department of Transportation. Sacremento, CA.

This report summarizes a series of eight first-level Heavy Vehicle Simulator testing reports, two laboratory reports on shear and fatigue testing, a forensic investigation report, a report on the backcalculation of deflection measurements, and a second-level analysis report, all of which describe the results of Heavy Vehicle Simulator (HVS) testing on a full-scale experiment designed to validate California Department of Transportation (Caltrans) overlay strategies for rehabilitating cracked asphalt concrete with modified binder overlays.

Khazanovich, L., Velaquez, R., and Nevijiski, E G. 2005, "Evaluation of Top-Down Cracks in Asphalt Pavements by Using a Self-Calibrating Ultrasonic Technique.". Journal of the Transportation Research Board No. 1940 pp 63-68. Transportation Research Board, Washington, DC.

To select the optimal strategy for treatment of a cracked asphalt pavement, it is important to determine the extent of cracking (partial depth or full depth). This paper presents the results of an explanatory study aimed at examining the applicability of the ultrasonic technology for evaluation of cracks and longitudinal joints in flexible pavements. It was shown that this technology, which has been used successfully for many years for the evaluation of concrete structures, could provide a simple, quick, and objective procedure for evaluation of surface distresses in asphalt concrete pavements. The results of laboratory testing and field testing at the Minnesota Road Research Project test facility demonstrate the potential of this technology.

Kim, I., and Tutumluer, E. 2006. "Field Validation of Airport Pavement Granular Layer Rutting Predictions.". Journal of the Transportation Research Board No. 1952 pp 48-57. Transportation Research Board. Washington, DC.

This paper presents research findings on the prediction performances and field validations of the recently developed granular base/subbase layer permanent deformation models using the full-scale pavement test section data from the FAA's National Airport Pavement Test Facility (NAPTF). The FAA-designated P209/P154 aggregate materials were used in the construction and testing of the NAPTF flexible pavement test sections with variable-thickness base and subbase courses. To account for the rutting performances of these substantially thick granular layers, a comprehensive set of repeated load triaxial tests, considering both constant and variable confining pressure (CCP and VCP) conditions, were conducted on the P209 base and P154 subbase granular materials. On the basis of the laboratory test results, both CCP- and VCP-type permanent deformation models were developed to predict maximum ruts that occurred at the NAPTF under both six-wheel and four-wheel gear loadings applied following a wander pattern. The developed rutting models were first calibrated for the field conditions and then evaluated for predicting the field accumulation of permanent deformations by properly taking into account the NAPTF trafficking data, effects of stress rotation due to moving wheel loads, and loading stress history effects. A comparison of the measured and predicted permanent deformations indicated that a good match for the measured rut magnitudes and the accumulation rates could be achieved only when the magnitudes and variations of stress states in the granular layers, No. of load applications, gear load wander patterns, previous loading stress history effects, trafficking speed or loading rate effects, and finally, principal stress rotation effects due to moving wheel loads were properly accounted for in the laboratory testing and permanent deformation model development.

Kim, Y.R., and Park, H. 2002. "Use of Falling Weight Deflectometer Multi-load data for Pavement Strength Estimation.". Report No. FHWA/NC/2002-006. Federal Highway Administration and North Carolina Department of Transportation. Raleigh, NC.

The objective of this study is to develop a mechanistic-empirical method for assessing pavement layer conditions and estimating the remaining life of flexible pavements using multi-load level Falling Weight Deflectometer (FWD) deflections. A dynamic finite element program, incorporating a stress-dependent soil model, was developed to generate the synthetic deflection database. Based on this synthetic database, the relationships between surface deflections and critical pavement responses, such as stresses and strains in each individual layer, have been established. A condition assessment procedure for asphalt pavements that uses multi-load level FWD deflections has been developed using these relationships. The verification study was conducted using field data. The results indicate that the proposed procedure can estimate the base and subgrade layer conditions. It was found from the study for the nonlinear behavior of a pavement structure that an FWD test with a load of 12 kip or less does not result in any apparent nonlinear behavior of the subgrade in aggregate base pavements. The study also indicated that the deflection ratio obtained from multi-load level deflections may predict the type and quality of the base/subgrade materials. With regard to the condition assessment of the asphalt concrete (AC) layer, the AC layer modulus and the tensile strain at the bottom of the AC layer were found to be better indicators than deflection basin parameters. The procedures for performance prediction of fatigue cracking and rutting were developed for flexible pavements. The drastically increasing trend in fatigue cracking with time may not be predicted accurately using the proposed procedure. Such trends may be due to the environmental effects and the inconsistent distress measurements. Predicted rut depths using multi-load level deflections show good agreement with measured rut depths over a wide range of rutting. However, the procedure using single load level deflections consistently underpredicts the rut depths. It was concluded that the rutting prediction procedure using multi-load level deflections can estimate an excessive level of rutting quite well and, thus, improve the quality of prediction for rutting potential in flexible pavements. The layer condition assessment procedure and the remaining life prediction algorithms developed in this project were incorporated into APLCAP (Asphalt Pavement Layer Condition Assessment Program) version 2.0, the VisualBasic program developed under the NCHRP 10-48 project.

Kingham, Ian and Jester, Robert. 1990. Edition. *Deflection Method for Designing Asphalt Concrete Overlays for Asphalt Pavements*. Research Report No. 81-1 (RR-83-1). The Asphalt Institute, Lexington, KY.

This is an updated version of the Research Report of the same title issued under the designation RR-69-3 and written by R. Ian Kingham. This report revised by Robert N. Jester, contains a design procedures based on research carried out in the Asphalt Institute's Structural Design Research Program. Contains 8 illustrations. 20 pages.

Kingham, R. 1970. "A Pavement Deflection Study to Develop an Asphalt Overlay Design Method and Discussion.". *Proceedings*: Association of Asphalt Paving Technologists", Association of Asphalt Paving Technologists, St Paul, MN.

A method for determining the thickness required of an asphalt concrete overlay on an asphalt pavement is described. The method is based on a study of Benkelman beam pavement deflections and its supported elastic theory, field observations, and engineering experience.

Ksaibati, K., Whelan, M L., Burczyk, J M., and Farrar, M J. 1994. "Selection of Subgrade Modulus for Pavement Overlay Design Procedures.". Report No. 94-34. Wyoming Department of Transportation. Cheyenne, WY.

This study was conducted to better understand how selecting a subgrade resilient modulus (M sub R) value influences the thickness of an asphalt overlay pavement. The objectives of this study were to: 1) investigate the importance of several fundamental soil properties (water content, plasticity index, liquid limit, group index) on selecting a design M sub R value for cohesive soils; 2) define the actual relationship (correction factor) between backcalculated and laboratory-based M sub R values for typical cohesive subgrade soils in Wyoming; 3) compare actual subgrade field deviator stresses to the deviator stress assumed in determining a design M sub R value from laboratory testing; and 4) determine the effect of selecting an M sub R value on the design overlay thicknesses for typical pavement sections in Wyoming. The data analysis resulted in several important conclusions about factors that influence the determination of the M sub R value and how this value affects the final overlay thickness design for a given pavement section. Chapter 1 of this report provides an introduction. Chapter 2 reviews the traditional methods used to characterize subgrade soils, methods to determine M sub R for subgrade soils, and the AASHTO overlay design procedure. Chapter 3 describes the data collection process and overall evaluation strategies followed in this research. Chapter 4 discusses the laboratory testing, backcalculation testing, and several important results on the factors that influence the selection of a design subgrade M sub R value. Chapter 5 discusses the impacts of selecting a particular method for determining a design M sub R value on the resulting overlay thickness. Chapter 6 summarizes the study, presents the conclusions, and makes recommendations for needed future research.

Ladd, D., Parker, F., and Pereira, T. 1976. *Structural Design of Pavements for Light Aircraft*. FAA-RD-76-179; ADA-041-300. Army Engineer Waterway Experiment Station, Vicksburg, MS.

This report presents structural design criteria for airfield pavements to be used by light aircraft; i.e., those with gross weights less than 30,000 lb. Presented are criteria for conventional flexible and rigid pavements, for rigid and flexible pavements containing stabilized layers and membrane-encapsulated soil layers, and for unsurfaced areas; a cost-benefit analysis; and a construction guide for thin concrete pavements.

Lee, J., Turner, D., Oshinski, E., Stokoe II, K., Bay, J., and Rasmussen, R. 2003. "Continuous Structural Evaluation of Airport Pavements with the Rolling Dynamic Deflectometer.". Airfield Pavements: Challenges and New Technologies. American Society of Civil Engineers. Reston, VA

Nondestructive testing (NDT) is an important tool in the design, management, maintenance, and rehabilitation of airport pavements. An NDT device known as the Rolling Dynamic Deflectometer (RDD) was developed at the University of Texas at Austin to evaluate the structural condition of both flexible and rigid pavements. The RDD is a large truck with a servo-hydraulic vibrator that applies static and dynamic loads to the pavement through two specially designed loading rollers and measures deflections using an array of rolling sensors. Compared to other commonly used nondestructive testing devices that perform testing at discrete locations, the RDD measures continuous deflection profiles, efficiently and robustly. Among the many uses, continuous deflection profiles represent an excellent means of: (1) identifying locations where further discrete type testing should be performed, and (2) relating the relative character of the discrete test sites to the overall pavement system. The RDD has been used to test pavements at the Dallas-Fort Worth International, Hartsfield Atlanta International, Seattle-Tacoma International, Portland International, Meacham International, and Greenville Municipal Airports. Results from the Meacham International and Greenville Municipal Airports are presented in this paper. At Meacham International, a Jointed Concrete Pavement (JCP) was tested with both the RDD and a Falling Weight Deflectometer (FWD). At Greenville Municipal, a JCP with an asphalt overlay was tested. Analysis methods using the collected data are discussed. These analysis methods include statistical descriptors, location of "unique" pavement features, and the comparison of deflections at joints and mid-slab regions.

Li, Y., and Metcalf, J. 2004. "Fatigue Characteristics of Asphalt Concrete from Asphalt Slab Tests.". Journal of Materials in Civil Engineering Vol. 16 No. 4 (July-August 2004). American Society of Civil Engineers. Reston, VA.

It is widely known that pavement fatigue life is often greater than that predicted based on laboratory bending beam or indirect tensile fatigue tests. Among many factors contributing to this difference, the effect of sample dimensions on fatigue life can be significant due to the multiphase nature of asphalt concrete. In this research, fatigue tests were conducted on asphalt slabs, which realistically simulate the fatigue cracks in pavements, to investigate crack initiation and propagation. The fatigue crack at the underside of asphalt slabs was identified as a semielliptical surface crack, which is the same type of fatigue crack found in pavements. A three-dimensional finite-element analysis was conducted to evaluate the stress intensity factor. Paris's law was used to characterize the crack propagation rate. The fatigue crack initiation life and propagation life determined from this research were compared with those calculated using the Strategic Highway Research Program existing parameters and equation. It was found that the two results are significantly different. The factors that may contribute to this difference were discussed.

Losa, M., Leandi, P., and Bacci, R. 2008. "Monitoring and Evaluating Performance Requirements of Flexible Road Pavements". *Proceedings*: The First International Symposium on Transportation and Development Innovative Best Practices". American Society of Civil Engineers, Reston , VA; China Academy of Transportation Services. Beijing, China.

This paper presents key findings of a research project, which is entrusted with setting up guidelines for monitoring and evaluation of pavement performance requirements with the aim of improving durability and both road safety and environmental sustainability. The study has investigated some techniques for monitoring structural and surface characteristics of asphalt pavements, such as Ground Penetration Radar, Falling Weight Deflectometer, Laser Profilmeter, Skiddometer, and has addressed problems arising from the use of these techniques; the paper describes some measures needed to overwhelm them and for optimizing data collection procedures. In order to enhance the evaluation of asphalt pavements, the study has focused on data analysis and interpretation methodologies; it proposes refinements to the existing procedures for both data processing and interpretation.

Lui, L. 2007. "GRP for Fast Pavement Assessment.". Report Nu. JHR 07-310. University of Connecticut, Storrs, CT; Connecticut Department of Transportation, Rocky Hill, CT.

This report summarizes the findings of Project JHRAC 00-2 for studying the fast assessment of road pavement with the ground penetrating radar (GPR). The report contains four parts. The first two are on actual GPR data acquisition and analysis: one on laboratory measurements and one on field surveys. The last two parts examine the GPR signal forward modeling and signal data processing. The author conducted laboratory tests of engineered materials using the RAMAC radar system with the 1-GHz antenna. The experiments used geotechnical fundamental measurements as the known parameters and correlated with the electromagnetic (EM) parameters obtained by GPR measurements. The fundamental parameters include thickness, density, aggregate material ratio, and air void ratio, etc. The GPR experiments test the materials to get the dielectric constant that directly determines EM wave velocity. Asphalt tests have been conducted on 30 asphalt specimens. The asphalt specimens produced at the Connecticut Advanced Pavement Laboratory have a dimension of 15 cm in diameter with 11.5 cm in height. Serving as the strong reflector to mark the travel time, an aluminum plate is placed underneath the specimen, to allow an easier identification of the time travel to and reflected back from the plate. The calculated EM wave velocity is about 131 m/microsecond for most dry asphalt specimens. The corresponding dielectric constant ranges from 4 to 5. The author also tested the response of the asphalt specimens in different ambient states (dry, saturated with water, and frozen). GPR data using the 1-GHz antenna were collected several times on a then-newly paved road segment on the Depot Campus of the University of Connecticut under different meteorological conditions. The GPR data clearly showed signature changes of the asphalt response between dry and water-saturated conditions. The radar wave velocity reduced about 5% when the asphalt was watersaturated (after heavy rain, collected in the morning of September 20, 2000). This change makes a relatively clearer identification. Meanwhile, the reflectivity increased 60%, making the reflection from the bottom of the base of the asphalt layer much more visible. To theoretically understand the effect of a thin dielectric layer (such as the hot mixed asphalt pavement) on radar signal propagation, a finite difference time domain (FDTD) forward modeling algorithm was developed. The simulation clearly demonstrated the wave guide effect of this surface thin layer. This simulation can be used as guidance for developing multi-channel GPR systems. To improve the resolution and preserve the penetration, if high-and low-frequency surveys are conducted simultaneously over the same road segment, it is possible to extrapolate the high frequency signal to a deeper depth numerically through a digital signal processing algorithm known as extrapolation with deterministic deconvolution (EDD) and consequently gain a higher resolution to a greater depth.

Lundstrom, R., Ekblad, J, Isacsson, U., and Karlsson, R..2007 "Fatigue Modeling as Related to Flexible Pavement Design: State of the Art.". Road Materials and Pavement Design, Vol. 8 No. 2, pp 165-205. Hermes Science Publications. France.

A literature review of fatigue modeling of flexible pavements is presented. Focus is on different approaches used for fatigue modeling as it relates to flexible pavement design. A brief description of different types of pavement design methods is given, with intent directed at presenting different views on fatigue modeling. The design methods involve either modeling fatigue on a purely empirical basis using semi-mechanistic methods or as intrinsic constitute behavior using numerical methods. Also included is a description of three broad categories of fatigue approaches used for asphalt fatigue characterization.

Mamlouk, M., Zaniewski, J., and He, W. 2000. "Analysis and Design Optimization of Flexible Pavement." Journal of Transportation Engineering, Vol. 126, Issue 2, pp. 161-167 (March/April 2000). American Society of Civil Engineers. Reston, VA

A project-level optimization approach was developed to minimize total pavement cost within an analysis period. Using this approach, the designer is able to select the optimum initial pavement thickness, overlay thickness, and overlay timing. The model in this approach is capable of predicting both pavement performance and condition in terms of roughness, fatigue cracking, and rutting. The developed model combines the American Association of State Highway and Transportation Officials (AASHTO) design procedure and the mechanistic multilayer elastic solution.

The Optimization for Pavement Analysis (OPA) computer program was developed using the prescribed approach. The OPA program incorporates the AASHTO equations, the multilayer elastic system ELSYM5 model, and the nonlinear dynamic programming optimization technique. The program is PCbased and can run in either a Windows 3.1 or a Windows 95 environment. Using the OPA program, a typical pavement section was analyzed under different traffic Vol.s and material properties. The optimum design strategy that produces the minimum total pavement cost in each case was determined. The initial construction cost, overlay cost, highway user cost, and total pavement cost were also calculated. The methodology developed during this research should lead to more cost-effective pavements for agencies adopting the recommended analysis methods.

Minjoto, M J C., Pais, J C., and Pereira, P A A. 2008. "The Temperature Effect on the Reflective Cracking of Asphalt Overlays.". Road and Pavement Materials, Vol. 9 No. 4, pp 615-632, Hermes Science Publications, France.

This paper describes a study which examined the influence of temperature variation in the reflective cracking of flexible pavements. This was accomplished through evaluating asphalt overlay damage arising from traffic and temperature variations during the year. A numerical simulation was conducted of the asphalt overlay behavior based on a 3D finite element analysis. The reflective cracking overlay life was predicted using a mechanistic-based overlay design method. An increase in the reflective cracking phenomenon occurred as a result of climatic temperature variations in pavements. The stress and strains created by temperature resulted in the premature distress of the asphalt overlay. The study also presents a comparison between the expected performance of asphalt rubber hot mixes and conventional asphalt overlays.

Molenaar, A. 2007. "Prediction of Fatigue Cracking in Asphalt Pavements: Do We Follow the Right Approach?". Journal of the Transportation Research Board No. 2001, pp 155-162. Transportation Research Board. Washington, DC.

The fatigue performance of asphalt concrete pavements is difficult to predict not only because many of the input parameters needed for the analyses are difficult to obtain but also because the fatigue phenomenon itself is not well understood. Most analyses, for example, take the tensile strain at the bottom of the asphalt layer as the factor that explains fatigue, although many pavements exhibit top-down cracking that has nothing to do with the tensile strain at the bottom of the asphalt layer. This approach implies that calibrating fatigue predictions on the basis of tensile strains at the bottom of the asphalt layer with the amount of cracking observed at the pavement surface is rather unrealistic. Furthermore, it is unlikely that fatigue cracks, if initiated at the bottom of the asphalt layer, show up at the pavement surface as clearly defined cracks. There are many indications that fatigue at the bottom of the asphalt layer is a matter of the development of a deteriorated zone with microcracks rather than development of clearly defined, discrete cracks. It is evident that large shift factors are needed to apply laboratory fatigue relations to field predictions. The magnitude of these shift factors depends, among other determinants, on the type of fatigue test and mode of loading. As discussed in this paper, only the slope of the fatigue relation can be estimated with confidence from laboratory fatigue tests. Finally, this paper shows that the existence of an endurance limit can be debated; in any case it is not a constant value of about 70 m/m. Experiments have shown that the suggested value is too high.

Monismith, C., Hudson, W., Finn, F., Skok Jr, E., and Shook, J. 2007. "What Pavement Research Was Done Following the AASHO Road Test and What Else Could Have Been Done but Was Not.". Transportation Research E-Circular No. E-C118. Transportation Research Board. Washington, DC.

The AASHO Road Test provided significant results that led to improved pavement design following the Road Test and to an expanded research effort by the pavement engineering community worldwide. In particular, it resulted in the development of what is termed today mechanistic-empirical (M-E) pavement design. Results of the AASHO Road Test also contributed to the development of nondestructive pavement evaluation, including overlay pavement design and to the development of pavement management concepts.

Mooney, M., Miller, G., Teh, S. and Bong, W. 2000. "Importance of Invasive Measures in Assessment of Existing Pavements.". Journal of Performance of Constructed Facilities, Vol. 14, Issue 4, pp. 149-154 (November 2000). American Society of Civil Engineers, Reston, VA

Countless miles of aged interstate highway pavement in the United States and significant expenditures associated with reconstruction highlight the importance of cost-effective rehabilitation measures. However, the prescription of ill-posed rehabilitation strategies due to improper assessment can be costly. This paper details a forensic investigation of a 22.5-km (14-mi) stretch of interstate highway in Oklahoma, consisting of multiple asphalt concrete (AC) overlays accumulated over a 40-year period. A thorough nondestructive investigation was carried out using falling weight deflectometer testing and ground penetrating radar. This was followed by a detailed invasive investigation involving coring, drilling and sampling, laboratory testing, and trenching. The pavement profile deduced from nondestructive test results alone failed to reveal a significantly weakened subsurface AC layer that was clearly revealed during invasive testing. Mechanistic analysis of the perceived pavement and actual pavement profiles reveal a significant difference in fatigue life. The reliance on nondestructive testing alone for pavement analysis and rehabilitation design would have been in significant error.

Newman, K., and Janoo, V. 1999. "Evaluation of Thermal Cracking at Elmendorf AFB Using SHRP Technology", Journal of Performance of Constructed Facilities. Vol. 13, No. 2 (May 1999) pp 76-81. American Society of Civil Engineers. Reston, VA.

An evaluation of runway and taxiway pavements was conducted using technology developed or utilized during the Strategic Highway Research Program (SHRP) to determine the effectiveness for identifying thermal cracking propensity of asphalt pavements. SHRP performance grades (PG) of PG52-28 and PG58-28 were measured for the 3 and 6% (weight-to-weight ratio) styrene-butadiene-styrene copolymer-modified asphalt binders employed in taxiway and runway construction. The high temperature SHRP performance grades were above that required by SHRP for the Anchorage, Alaska, area according to the SHRP weather database. The low temperature SHRP PG of the binders were found to be insufficient for the area. No rutting has been observed; however, the pavements developed transverse cracks after the first winter following construction of both the runway and taxiway pavements in 1994 and 1996, respectively. The SHRP thermal cracking model failed to predict any cracking within a 10-year period for both pavements. No obvious cause for the model failure could be ascertained. The thermal stress restrained specimen test revealed no significant difference between cracking temperatures for the 3 and 6% styrene-butadiene-styrene-modified binders.

Noureldin, S., Zhu, K., Harris, D., and Li, S. 2005. "Non-Destructive Estimation of Pavement Thickness, Structural No., and Subgrade Resilience Along INDOT Highways.". Report No. FHWA/IN/JTRP-2004/35. Joint Transportation Research Program, Purdue University, West Lafayette, IN; Federal Highway Administration; Indiana Department of Transportation. Indianapolis, Indiana.

Nondestructive testing has become an integral part for evaluation and rehabilitation strategies of pavements in recent years. Pavement evaluation employing the Falling Weight Deflectometer (FWD) and the Ground Penetrating Radar (GPR) can provide valuable information about pavement performance characteristics and be a very useful tool for project prioritization purposes and estimation of construction budget at the network level. FWD deflection testing is an accurate tool for determining pavement structural capacity and estimating the required thickness of overlays and hence is an accurate tool for planning for or estimating required current and future construction budgets. GPR is the only tool that a highway agency may use to develop an inventory of pavement layers thicknesses in the most efficient manner possible. By estimating pavement layer thicknesses and stiffness properties more reliable projections of network rehabilitation strategies and needs can be established, thus resulting in cost effective use of available funds. Traditional obstacles for the use of FWD and GPR in pavement evaluation at the network level used to be expenses involved in data collection, limited resources and lack of simplified analysis procedures. This report presents Indiana experience in pavement evaluation with the FWD and GPR at the network level. A network level FWD and GPR testing program is implemented as a part of a study to overcome those traditional obstacles. This testing program included Interstate Highways I-64, I-65, I-69, I-70 and I-74 and a No. of U.S. Roads and State Routes. It is concluded that network level testing employing the FWD and GPR is a worthwhile, technically sound program that will provide a baseline of structural capacities of in-service pavements in Indiana. Periodical generation of necessary data will be useful for determining how best to quantify structural capacity and estimate annual construction budget. FWD data on 2200 lane miles of the Indiana Department of Transportation (INDOT) network is recommended annually for network level pavement evaluation. Only three FWD tests per mile are recommended. This amount of testing can easily be conducted in one testing season. The information collected will allow the equivalent of 100% coverage of the whole network in 5 years. GPR data is recommended to be collected once every 5 years (if another thickness inventory is needed), after the successful network thickness inventory conducted in this study. GPR data collection is also recommended at the project level and for special projects. Both FWD and GPR data is recommended to be used as part of the pavement management system (together with automated collected data of international roughness index, IRI, pavement condition rating, PCR, rut depth, pavement quality index, PQI, and skid resistance).

Pais, J., Pereira, P., Capito, S., and Sousa, J., 2003. "The Reflective Cracking in the Pavement Overlay Design.". ***Proceedings*****: Maintenance and Rehabilitation of Pavements and Technological Control (July 2003), Guimaraes, Portugal.**

This paper describes a new pavement overlay design method that takes into account the reflective cracking as the most predominant type of overlay distress. This phenomenon is characterized as the propagation of old cracks to the new pavement layer. The models proposed are based on a finite element model that closely approximates actual field phenomena based on measurements done on Arizona and Portugal. Test sections have been constructed to access the effect of overlay thickness, crack width and load to validate the design method. Crack activity under the effect of traffic loads was initially calculated and compared with field results, and strains in the zone above cracks were calculated. Expected performance of asphalt-rubber hot mix and conventional asphalt overlay was calculated using the proposed model and a conventional overlay design simulation was made.

Park, S., and Lytton, R. 2000. "Model for Predicting Rut Development in Unbound Aggregate Layers", Proceedings: International Center for Aggregates Research 8th Annual Symposium: Aggregates -Asphalt Concrete, Bases and Fines (April 2000). Denver, CO.

There are several major practical consequences of the recently identified stress dependent orthotropic properties of unbound aggregate base courses. Among those are the Poisson's ratios that may rise above 0.5, the compressive stresses that are generated in such base courses under load, the stiffening and strengthening effect of repeated loading to progressively increase the base course resistance to rutting, and the increased risk of moisture accelerated rutting in the event that water enters the base course through cracks or joints in the surface layer. All of these effects are illustrated in this paper by a combination of field observations of accelerated loading tests results and their duplication with computations made by stress-dependent finite element analyses, guided by theory which explains the effects noted above. Because of its progressive work-hardening, the percent of the total rutting that takes place in the base course continually decreases with the No. of load repetitions. Moisture-accelerated rutting in the base course occurs at greatly different rates depending upon its critical soil suction vs.-water content-vs.-hydraulic conductivity curves.

Pidwerbesky, B D., Steven, B D., and Arnold, G. 1997. "Subgrade Strain Criterion for Limiting Rutting in Asphalt Pavements.". ***Proceedings*****: Eighth International Conference on Asphalt Pavements (August 1997), Federal Highway Administration. Seattle, WA.**

The subgrade strain criterion for asphalt pavements was investigated by instrumenting pavement layers and underlying subgrades. Vertical compressive strains under wheel loads were recorded under accelerated and normal rates of loading. Four test pavements in the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF), Christchurch, New Zealand, and one in-service pavement were included in the research. The asphalt surfacings were 25 mm to 85 mm thick, the subsurface granular layers were 135 mm to 300 mm thick, and the subgrade CBR ranged from 4% to 28%. Vertical compressive strains measured in the unbound granular layers and subgrade of flexible pavements are substantially greater than predicted values. Vertical compressive strains in unbound granular layers under thin asphalt surface layers can be equal in magnitude to the subgrade strains. A new subgrade strain model is developed and presented in this paper, which substantially reduces the required thicknesses of overlays.

Powell, R L. 2008. "Modeling Rutting Performance on the NCAT Pavement Test Track.". ***Proceedings*****: Transportation Research Board 87th Annual Meeting (January 2008). Transportation Research Board. Washington, DC.**

Experimental sections on the 2.8 kilometer Pavement Test Track are cooperatively funded by external sponsors, most commonly state departments of transportation (DOTs), with operation and research managed by the National Center for Asphalt Technology (NCAT). Forty-six different flexible pavements are installed at the facility, each at a length of 60 meters. Materials and methods unique to section sponsors are imported during construction to maximize the applicability of results, producing a variety of mix types. A design lifetime of truck traffic is applied in an accelerated manner over a two-year period of time, with field performance documented weekly. This high level of control provides a unique opportunity to compare the field record to laboratory testing in order to develop a predictive methodology for rutting performance. A new method of characterizing traffic is developed and referred to herein as ?load-temperature spectra.? Axle loadings are first banded in accordance with high temperatures in the performance grading (PG) system for binders and a weight factor is developed for each band in order to reflect increased rutting potential at higher temperatures. Regression methods are then used to generate a distinct model for

various laboratory tests to predict field performance at an empirically determined single pavement age resulting from the application of banded, weighted traffic. Finally, a time-dependent shift factor is developed to change the model output to predict rutting performance at all other ages. Incremental rutting is thus computed at various ages and added cumulatively to estimate total rut depth at any age.

Priest, A. and Timm, D. 2006. *Methodology and Calibration of Fatigue Transfer Functions for Mechanistic-Empirical Flexible Pavement Design*. NCAT Report 06-03. National Center for Asphalt Technology, Auburn University, AL.

Abstract not available.

Public Works and Government Services Canada. 1992. *Manual of Pavement Structural Design.* Public Works and Government Services Canada. Hull, QB, Canada.

This manual presents standards and guidelines on the structural design of both airside and groundside airport pavements, the design of restoration measures, as well as the design of new pavement facilities. The objective of this manual is to detail uniform design practices that will ensure the safety, quality, and cost effectiveness of pavement facilities subject to Canadian climatic, construction and operational conditions.

Public Works and Government Services Canada. 1995. *Pavement Structural Design Training Manual.* Public Works and Government Services Canada. Hull, QB, Canada.

Pavements are constructed to provide ground surfaces for the movement of vehicles or other types of traffic and the design of these facilities is based on the needs and characteristics of the traffic to be served and on economic considerations. This manual is limited in scope to the structural design of pavements serving aircraft or ground vehicle traffic. Geometric design requirements are given in other publications.The structural design of pavements outlined in this manual is primarily concerned with the thickness of pavements and their component layers, as necessary to provide sufficient bearing strength for the traffic loadings, and to attenuate frost effects. Other quality characteristics of a pavement structure, such as durability, roughness and skid resistance, are controlled by material and workmanship requirements contained in construction specifications. The design methods given in this manual are used for pavements constructed at Canadian airports.

Rabab'ah, S., and Liang, R. 2008. "Evaluation of Mechanistic-Empirical Design Approach over Permeable Base Materials.". *Proceedings*: Transportation Research Board 87th Annual Meeting. Transportation Research Board. Washington, DC.

The in situ moduli of unbound pavement materials vary on a seasonal basis as a function of temperature and moisture conditions. An understanding of material properties as affected by environment is critical to address moisture related distresses originating from the base and subgrade. This paper reviews the characterization of open graded materials in the Mechanistic-Empirical Pavement Design Guide (MEPDG), and applies this characterization to predict performance of Ohio DOT specific permeable bases. Flexible pavement designs and performance derived from the MEPDG approach are systematically studied for a range of different base materials, AC thickness, and subgrade soils. The impact of the following parameters on the rutting based service life was evaluated: thickness design (asphalt concrete thickness), environment (moisture content variation), and material properties (base and subgrade resilient modulus). The parametric study results showed that Subgrade resilient modulus had exerted more impact on permanent deformation than the base resilient modulus. As shown consistently in this study, the impact of unbound materials on performance as predicted by the MEPDG methodology is less pronounced than the impact of asphalt concrete layer thickness.

Raju, S., Kumar, Reddy, K S., S S., Bose, S., and Pandey, B B. v2008. "Analysis of Top-Down Cracking Behavior of Asphalt Pavements.". *Proceedings*: Transportation Research Board 87th Annual Meeting (January 2008). Transportation Research Board. Washington, DC.

Construction of four-lane and six-lane national highways under the National Highway Development Program has been taken up in a massive way. Several stretches of asphalt pavements constructed under this program have already been opened to traffic. Premature surface cracks were observed on some stretches within a year or two of construction and these cracks were found to initiate from top and progress downwards which is generally known as top-down cracking. In the present study, an effort was made to examine the causes of the top-down cracking susceptibility of asphalt layers. Temperature gradients of typical asphalt layers were measured in the field as well as in the laboratory under high pavement temperature condition. Finite element analysis of typical asphalt pavements was carried out for the evaluation of the influence of high pavement temperature on surface tensile strains in the asphalt layer. The effect of heavy axle loads in combination with high pavement temperatures was analyzed for the evaluation of initiation of surface cracking. Surface traction forces developed during rolling and braking were also considered. Parametric analytical studies indicated that temperature was a critical parameter influencing the top-down cracking susceptibility of asphalt layer, especially in combination with heavy axle loading.

Sale, J., Hutchinson, R., Ulery Jr., H., Ladd, D., and Barker, W. 1977. *Comparative Performance of Structural Layers in Pavement Systems. Vol. II. Analysis of Test Section Data and Presentation of Design and Construction Procedures*. FAA-RD-73-198 Vol. 2; ADA-052-154. Army Engineer Waterway Experiment Station ,Vicksburg, MS.

Comparative performance analyses of full-scale pavement test sections indicated that reductions in conventional flexible and rigid pavement thickness requirements are warranted when high-quality stabilized layers are incorporated in the pavement structure. Based on the findings in the analyses, design procedures for airport pavement systems incorporating certain types of stabilized layers were developed. Construction procedures for stabilized layers were developed based on field operations. This Vol. of the report summarizes pertinent data from the test sections, describes the analyses, and presents the design and construction procedures. Vol. I of the report describes in detail the design, construction, and behavior under traffic of test sections specially designed for this study. Vol. III describes analysis of membrane-encapsulated soil layers (MESL) and presents design and construction procedures for airport pavements incorporating MESL. Vol. IV describes the performance and analysis of insulating layers in pavement test sections.

Shukla, P K, and Das, A. 2008. "A Re-Visit to the Development of Fatigue and Rutting Equations used for Asphalt Pavement Design.". International Journal of Pavement Engineering Vol. 9 No. 5, pp 355-364. Taylor and Francis Limited. London, United Kingdom.

Fatigue and rutting equations are essentially used in design of asphalt pavements by mechanistic-empirical method and they are developed from the laboratory and/or the field performance data. Conventionally, ordinary least square regression analysis is employed for developing these equations. The present paper discusses the underlying assumptions of ordinary least square regression analysis and shows that these assumptions are violated while developing fatigue/rutting equations from the data obtained from field or laboratory measurements. This paper proposes the use of a measurement error based estimator in order to alleviate the inaccuracy associated with use of least square regression analysis while developing fatigue and rutting equations. A step-by-step methodology is presented on how to develop the fatigue or rutting equation from a given data set using the proposed method. The confidence intervals of the estimators are obtained using bootstraping technique. Finally, a design example is presented to show the differences in design thickness values obtained by the conventional and the proposed method.

Sidess, A., Bonjack, H., and Zoltan, G. 1992. "Overlay Design Procedure for Pavement Maintenance Management Systems.". Transportation Research Record No. 1374, pp 63-70, Transportation Research Board. Washington, DC.

A methodology for flexible pavement rehabilitation and development of overlay thickness design curves for pavement maintenance management systems (PMMSs) is presented. The methodology is based on nondestructive testing of deflection basin measurements and on the rational approach that characterizes pavement response to major deterioration criteria, such as fatigue and rutting. Within the general framework, subgrade and pavement were classified into three categories of strength: weak, medium, and strong according to the measured deflection (at a distance of 1.80 m from the loading plate) and the surface curvature index parameter. With reference to these categories, a structural index (SI) was defined. Between the SI and overlay thickness there is a dependence that can be expressed by a design curves system related to different traffic categories and subgrade type. The present methodology can readily be incorporated as a subsystem within the general PMMS, and it enables fast solutions at the network level for economic evaluation and rehabilitation priority order determination of extensive road systems.

Smith, D M. 2000. "Response of Granular Layers in Flexible Pavements Subjected to Aircraft Loadings", Army Corps of Engineers, Report No. ERDC/GL TR-00-3, Vicksburg, Michigan.

Airfield pavement design is a complex blend of relatively simple linear elastic theory, fatigue concepts, correlations with small-scale and full-scale tests, and pragmatic adjustments to reflect observations of in-service pavements. The granular base and subbase have always posed the most difficult analytical problem in traditional pavement design methodologies. For this reason, the granular layers have never been treated explicitly in design as have the asphalt concrete (AC) layer and subgrade layer, which have used predictive models for cracking in the AC and rutting in the subgrade as a function of linear-elastic strain and material properties. Instead, these granular layers were carefully specified in terms of gradation, plasticity, and in-situ density to minimize deformation under traffic. However, today's designers are being asked to predict pavement performance under a variety of nonstandard conditions. This is a far more complex task than simply providing safe thickness and specifications for the material. To deal with this new challenge, the design community must have material models that predict cumulative deformations under repetitive aircraft loads. In order to apply these material models, mechanical response data are required to calibrate the necessary model parameters. The parameters used to define strength, failure, and deformation properties must be defined for any material to be modeled. This report describes the constitutive model requirements, laboratory tests, and analysis used in developing a response model for an unbound granular base course typical of an airfield pavement.

Sonyok, D., and Zhang, J. 2008. "Ground Penetration Radar for Highway Infrastructure Condition Diagnostics: Overview of Current Applications and Future Development.". Proceedings: Transportation Research Board 87th Annual Meeting. Transportation Research Board. Washington, DC.

Ground Penetration Radar (GPR) is a powerful, reliable and high performance nondestructive testing tool for solving various kinds of engineering problems related to geotechnical site investigation, construction, and maintenance of highways and bridges. It has been increasingly utilized for the management of highway infrastructures such as road pavements and bridges on a network level which requires condition assessment and deterioration modeling. GPR can determine the layer thickness and estimate moisture content of the in-situ soil underlying the pavement. It has a promising future because of its efficiency, quality, and non-destructiveness. GPR provides reliable and significant information to pavement condition evaluation that is very useful to predict the pavement's structural capacity and performance. This will further help improve pavement maintenance and rehabilitation strategies, and in the long term, it provides rationalities in allocating funds. GPR in combination with other nondestructive evaluation methods such as Falling Wheel Deflectometer (FWD) or Rolling Wheel Deflectometer (RWD), can provide complete data for pavement layer thickness, deflection and elastic moduli for the mechanistic pavement performance prediction model. However, the application of GPR in highway engineering is limited because of our incomplete understanding about the dielectric properties of highway materials. This paper presents the state-of-the-art GPR applications in highway infrastructure condition assessment and future development. It also highlights the possible integration of GPR with other nondestructive testing methods and its development for the network-level pavement management system. Finally, limitations and issues related to further investigation of the GPR improvement are discussed.

Sousa, J., Pais, J., Saim, R., Way, G., and Stubstad, R. 2002. "Mechanistic-Empirical Overlay Design Method for Reflective Cracking.". Journal of the Transportation Research Board No. 1809, pp 209-217. Transportation Research Board. Washington, DC.

A new and innovative mechanistically based pavement overlay design method is described that considers the most predominant type of overlay distress observed in the field: reflective cracking above old cracks in the underlying pavement surface. Both dense-graded hot-mix asphalt (HMA) and gap-graded asphalt rubber (wet process) mixes were studied in the laboratory and in the field to derive the necessary mechanistic relationships and statistically based equations. The models proposed are based on a finite element model that closely approximates actual field phenomena. Many field test sections, mainly in Arizona, were studied during the course of the research. Other HMA mixes used for overlays may also be calibrated and used through the proposed method, but the relevant mix properties of any additional materials or environmental zones must first be determined. The two mix types studied are mainly used in the desert southwest region of Arizona and California. The overlay design program is available from the Rubber Pavements Association or Arizona Department of Transportation in the form of an Excel spreadsheet with an easy to- use Visual Basic computer program (macro).

Steven, B., Jones, D., and Harvey, J T. 2007. "Reflective Cracking Study: First-Level Report on the HVS Rutting Experiment.". Report No. UCPRC-RR-2007-06. California Department of Transportation, Sacramento, CA.

This report is the seventh in a series of first-level analysis reports describing the results of Heavy Vehicle Simulator (HVS) testing on a full-scale experiment designed to validate California Department of Transportation (Caltrans) overlay strategies for rehabilitating cracked asphalt concrete. The report presents results of the six HVS rutting testing sections, designated as 580RF through 585RF, carried out six different overlays. The objective of this project is to develop improved rehabilitation designs for reflective cracking for California by evaluating the reflective cracking performance of asphalt binder mixes used in overlays for rehabilitating cracked asphalt concrete pavements.

Sun, L., Bi, Y., Hu, X., and Li, F. 2006 "Top-Down Cracking Analysis and Control for Asphalt Pavements.". 10th International Conference on Asphalt Pavements. International Society for Asphalt Pavements (August 2006). Quebec City, Canada.

In recent years, there are increasing evidences that suggest most single or parallel longitudinal cracks under wheel path initiate at the surface course and propagate downward (Top-down crack). In general, these types of cracks occur on the heavy-duty pavements, therefore they are perceived heavy load-related. In order to analyze the mechanisms that cause the top-down cracks, a device to measure the distribution of the contact pressure between the real tire and the pavement was developed. With this device, the contact pressure distributions of 91 cases covering 5 types of real tire were measured. Based on the measured results, the mechanical analysis of typical heavy-duty pavement structures was conducted with 3D Finite Element Method (FEM). Through the comparison between the position and direction where the maximal stresses locate at and where the top-down cracks initiate at, the repeated shear stress under the non-uniform contact pressure between tire and pavement was considered to be the main causation of the top-down cracks. The control of the shear strength of the mixture is one of the measures to control the top-down cracks. For this reason, a uniaxial penetration test (UPT) to measure the shear strength of mixture was developed based on the similarity principle, and the corresponding test parameter and data process method were presented.

The Asphalt Institute (AI). Second Edition, 2000. *Asphalt Overlays for Highway and Street Rehabilitation*. Manual Series No. 17 (MS-17). The Asphalt Institute. Lexington, KY.

This is the new and revised edition, which has been developed to provide the state-of-the-practice for evaluating and designing asphalt overlays for both asphalt and concrete pavements. Emphasis has been placed on alternatives to combine sound pavement management principles, accurate distress identification, and a detailed structural analysis for an asphalt overlay design to carry the projected vehicular loading. This new edition has been rewritten to reflect changes in rehabilitation strategies that took root in the late 1980s and early 1990s. Chapter 1 has been written to capture the asphalt applications available to meet demands placed on an aging national highway infrastructure Chapters 6 and 10 on Nondestructive Testing and Fractured Slab Technology respectively have been added. Chapters 12, 13 and 14 have been developed as Guide Specifications for the more popular rehabilitation techniques for concrete pavement-Rubblization, Crack/Break and Seat, and Saw-Cut and Seal Lastly, Chapter 15 on "Drainage has been expanded to include the latest innovations using longitudinal edge drains and permeable bases for use in conjunction with rehabilitation of concrete pavements. 177 pages, numerous illustrations and tables.

The Asphalt Institute (AI). Second Edition. *A Simplified Method for the Design of Asphalt Overlays for Light to Medium Traffic Patterns*. Information Series No. 139 (IS-139). The Asphalt Institute. Lexington, KY.

Gives guidelines for overlaying deteriorated road, street, and highway pavements of every kind and includes information for determining overlay thickness; 4-page folder.

The Asphalt Institute (AI). Seventh Edition. *The Asphalt Handbook*. Manual Series No. 4 (MS-4). The Asphalt Institute. Lexington, KY.

The Asphalt Handbook is the Asphalt Institute's comprehensive manual on the use of asphalt. For 70 years, it has served the asphalt industry as the primary reference manual for contractors, engineers, consultants, specifiers and user agencies. This new edition of MS-4 showcases the latest asphalt technologies. New topics covered include: Superpave asphalt binder, Superpave mix design, stone matrix asphalt, open graded friction courses, quality control

& acceptance, pavement management, and rehabilitation of concrete pavements with HMA. Fully illustrated, 832 pages.

The Asphalt Institute (AI). Third Edition, 1987. *Thickness Design – Airports*. Manual Series No. 11 (MS-11). The Asphalt Institute. Lexington, KY.

Intended primarily for use by design engineers, this manual is an engineering guide for design and construction of airport pavements serving aircraft of more than 270kN (60,000 lb.) gross. Presents a fundamental, rational design procedure that is approved on a case-by-case basis by FAA. Includes 141 illustrations and 13 tables. Spiral bound, 236 pages.

The Asphalt Institute (AI). Third Edition. *Thickness Design – Asphalt Pavements for General Aviation*. Information Series No. 154 (IS-154). The Asphalt Institute. Lexington, KY.

This publication formerly titled Full-Depth Pavements for General Aviation is a guide to the design and construction of asphalt pavements for airports intended to serve aircraft up to 270kN (60,000 lb.). Approved by FAA on a case-by-case basis for Light Aircraft. Contains tables, drawings, lists of aircraft and photos. 24 pages.

Thompson, M R., and Bejarano, M O. 1997. "Subgrade Criteria for Airport Flexible Pavement Design", *Proceedings*: Aircraft/Pavement Technology In the Midst of Change pp 18-32, American Society of Civil Engineers, Seattle, Washington.

Current mechanistic-empirical airport pavement design procedures use Elastic Layer Programs (ELP) to predict pavement responses (deflections, stresses, strains) generated by the gear load. The procedures incorporate subgrade strain criteria for controlling pavement rutting. WES/CE/FAA and AI vertical compressive strain criteria were developed from ELP analyses of pavement sections per the revised CBR equation. The limited scope of pavement test sections and performance data required extrapolation of other subgrade, loading and climatic conditions. The large and varying rutting criteria used to interpret the test section performance data are not consistent with the more rigorous criteria generally associated with high type airport pavements. A subgrade stress ratio (SSR = repeated deviator stress/soil strength) approach is presented. The University of Illinois (U of IL) SSR criteria ensure the pavement exhibits "stable" permanent deformation performance. Subgrade rutting is controlled by limiting SSR to acceptable levels, depending on traffic. The WES/CE/FAA strain criteria expressed in SSR terms are below about 0.4. These SSRs are very conservative and result in increased pavement thickness. Permissible SSRs for airport subgrades are probably in the range of 0.5 to 0.7. Load pulse characteristics (stress level and duration) of multiple-wheel gear configurations and stress history effects (subsequent of stress level applications) on subgrade permanent deformation.

Tighe, S., Haas, R., and Ningyuan, L. 2002. "Development of Asphalt Overlay Performance Models from the C-LTPP Experiment", *Proceedings*: Ninth International Conference on Asphalt Pavements (August 2002), Copenhagen, Denmark.

The Canadian Long Term Pavement Performance (C-LTPP) study was initiated in 1989. The study involves 65 sections in the 24 provincial sites that received rehabilitation comprising various thicknesses of asphalt overlays. This paper describes the impacts of the various alternative rehabilitation treatments on pavement performance in terms of roughness progression under comparative traffic loading, climate, and subgrade soil conditions. Factor effects, including climatic zone, subgrade type and traffic level were also evaluated. The methodology developed in this study on pavement roughness evaluation can be applied to performance trends analysis of other LTPP data.

Tighe, S., Haas, R., and Ponniah, J. 2003. "Life-Cycle Cost Analysis of Mitigating Reflective Cracking", Transportation Research Record No. 1823, pp 73-79. Transportation Research Board. Washington, DC.

Reflective cracking is a major and costly problem in many countries. It occurs in the top (overlay) layers above existing cracks in the lower (existing) pavement. This type of cracking can lead to premature deterioration of the pavement structure through the infiltration of moisture and debris. Although extensive research has been directed toward mitigation of the problem, work needs to be done, as it still appears to be a major problem. The problem is related in part to the fact that most of the work being done involves rehabilitation. One of the most common types of pavement rehabilitation is the use of an asphalt overlay. The focus of the present analysis is the economic benefits of reducing and treating reflective cracking before the placement of an asphalt overlay. A methodology for converting crack spacing to roughness is also presented. This information is used to examine how cracking is related to the measured international roughness index values. A model relating the amount of cracking to the loss of serviceability or a reduction in service life is presented. That model indicates that a reduction of transverse crack spacing from 5 to 20 m should result in a 5-year extension of service life, with a cost savings of $25,000 (2002 U.S. dollars) per two-lane kilometer. Measurement and treatment of cracking can also yield significant benefits. Benefit-cost ratios from the measurement of cracking can range from about 5 to 50, while proper and timely crack treatment (routing and sealing) can result in an extension of pavement life by 2 years and cost savings of $7,000 per lane kilometer.

Turner, V. 1990 *A Basic Guide to Overlay Design Using Nondestructive Testing Equipment Data*. ADA-234-472. Washington University, Seattle, WA.

The purpose of this paper is to provide a basic and concise guide to designing asphalt concrete (AC) overlays over existing AC pavements. The basis for these designs is deflection data obtained from nondestructive testing (NDT) equipment. This data is used in design procedures which produce required overlay thickness or an estimate of remaining pavement life. This guide enables one to design overlays or better monitor the designs being performed by others. This paper will discuss three types of NDT equipment, the Asphalt Institute Overlay Designs by Deflection Analysis and by the effective thickness method as well as a method of estimating remaining pavement life, correlations between NDT equipment and recent correlations in Washington State. Asphalt overlays provide on of the most cost effective methods of improving existing pavements. Asphalt overlays can be used to strengthen existing pavements, to reduce maintenance costs, to increase pavement life, to provide a smoother ride, and to improve skid resistance.

Unified Facilities Criteria (UFC). 2001. *Airfield Pavement Evaluation*. UFC 3-260-03. Department of Defense, Washington, DC.

This document presents criteria for evaluation of the load-carrying capacity of pavements used (or to be used) for the support of aircraft. An evaluation is conducted to assess the allowable traffic that a pavement can sustain for given loading conditions or the allowable load for a given amount of traffic without producing unexpected or uncontrolled distress. This document is for use in evaluating Army, Air Force, Navy, and Marine Corps Airfields and Heliports and is applicable to conventional-type pavements. The procedures presented include direct sampling and nondestructive testing techniques. The document also describes computer programs that can be used for pavement evaluation.

Unified Facilities Criteria (UFC). 2001. *Pavement Design for Airfields*. UFC 3-260-02. Department of Defense, Washington, DC.

This document prescribes procedures for determining the thickness, material, and density requirements for airfield pavements in nonfrost and frost areas. It includes criteria for the California Bearing Ratio (CBR) procedure and elastic layered analysis for flexible pavements and the Westergaard Analysis and elastic layered analysis for rigid pavements. The elastic layered analysis for rigid pavements covers only plain concrete, reinforced concrete, and concrete overlay pavements.

Unified Facilities Criteria (UFC). 2001. *Standard Practice Manual for Flexible Pavements*. UFC 3-250-03. Department of Defense, Washington, DC.

This manual provides guidance for the preparation of drawings and specifications for road and airfield flexible pavements using asphalt cement materials. The term “asphalt” is used herein instead of bituminous (a generic term for both asphalt and tar materials) because this is the material most widely used in pavement construction. In current practice tar or coal-tar is used only in instances where fuel resistance is required. This manual also provides useful information for design engineers, laboratory personnel, and project managers concerning mix design, materials, production, and placement of the various asphalt mixtures. This manual prescribes materials, mix design procedures, and construction practices for flexible pavements.

Unified Facilities Criteria (UFC). 2004. *Pavement Design for Roads, Streets, Walks, and Open Storage Areas*. UFC 3-250-01FA. Department of Defense, Washington, DC.

This manual provides criteria for the design of pavements for roads, streets, walks, and open storage areas at U.S. Army and Air Force installations. This manual provides criteria for plain concrete, reinforced concrete, flexible pavements, and design for seasonal frost conditions. These criteria include subgrade and base requirements, thickness designs, and compaction requirements, criteria for stabilized layers, concrete pavement joint details, and overlays.

Unified Facilities Criteria (UFC). 2004. *Pavement Design for Roads, Streets, and Open Storage Areas, Elastic Layered Methods*. UFC 3-250-10FA. Department of Defense, Washington, DC.

This manual provides an elastic layered method for the thickness design of pavements for roads, streets, walks, and open storage areas at US Army and Air Force installations for the loadings and conditions set forth herein. The elastic layered procedure is used to determine thickness requirements of flexible and rigid pavement structures subject to vehicular traffic loads. This manual treats the thickness design of concrete pavements (both plain and reinforced), conventional flexible pavements, bituminous concrete pavements, and flexible pavements with stabilized layers. Other aspects of design, such as the preliminary investigation, mix design, material requirements, joints, overlays, reinforced concrete pavements, and the requirements for compaction and frost considerations are described in TM 5-822-5/AFM 88-7, Chap. 3 (UFC 3-260-02).

Unified Facilities Criteria (UFC). 2005. *Design: Pavements*. UFC 3-250-12N. Department of Defense, Washington, DC.

Design criteria for use by qualified engineers are presented for the design of pavements and supporting materials for roads, parking areas, and walks. The contents include procedures for conducting preliminary site reconnaissance, soil investigations, and traffic analyses, and criteria for the design of subgrade, subbase, and base courses, flexible and rigid pavements, low-cost roads, and sidewalks.

US Army Corps of Engineers. 2005. *Pavement-Transportation Computer Assisted Structural Engineering (PCASE) User Manual*, Version 2.08. ERDC/GSL SR-XX-XX DRAFT. US Army Corps of Engineers. Washington, DC.

PCASE, or "Pavement-Transportation Computer Assisted Structural Engineering", is a software program that incorporates all transportation design and evaluation criteria into a stand-alone software package. This package allows research to be modularized into a set of scalable and reusable software components, which are then combined to create the PCASE desktop system designed to be installed on a single computer. PCASE calculations are based on the following UFC manuals: UFC 3-260-02 & UFC 3-260-02.PCASE software automates day-to-day engineering tasks by giving engineers a software tool. This process allows engineers to try multiple design or evaluation scenarios without having to start over each time. While every engineer using PCASE should have a thorough understanding of the pavement design and evaluation process, the software gives them a tool to automate repetitive tasks. Secondly, PCASE is the tool for technology transfer of pavement transportation criteria. What good is research that never gets used? PCASE puts the Vol.s of UFC criteria into a single stand-alone application that rests as an icon on the users' desktop.

Uzarowski, L., Maher, M., and Balasundaram, A. 2005. "Practical Application of GPR to Supplement Data from FWD for Quick Pavement Performance Prediction". *Proceedings*: 2005 Annual Conference of the Transportation Association of Canada (September 2005). Transportation Association of Canada, Calgary, Canada.

This paper describes how conventional geotechnical investigation and laboratory testing will provide material related information for pavement evaluations. A number of road agencies are using the Falling Weight Deflectometer (FWD) in order to supplement the data from the geotechnical investigation. The FWD load/deflection test provides information on the structural capacity of the entire pavement structure, as well as a basis for determining the condition of particular pavement layers. However, an extensive coring/drilling program is required to determine the existing pavement structure for pavement rehabilitation design. The Ground Penetrating Radar (GPR) is used to measure a continuous profile of pavement layers and identify areas of poor condition in the pavement structure. This information is also be used to improve the accuracy of FWD analysis. The GPR survey

combined with the FWD testing and borehole and coring investigation form a comprehensive pavement condition/structure package. This package can be used for a quick pavement performance prediction at the rehabilitation design, repair or post construction stage.

Von Holdt, C., and Scullion, T. 2006. "Methods of Reducing Joint Reflection Cracking: Field Performance Studies.". Report No. FHWA/TX-06/0-4517-3Texas Transportation Institute, College Station, TX; Federal Highway Administration; Texas Department of Transportation, Austin, TX.

Selecting rehabilitation options for Jointed Concrete Pavements (JCP?s) continues to be one of the most challenging tasks for pavement engineers. In project 0-4517, the performance of numerous treatments was investigated. Reports 0-4517-1 and 0-4517-3 identified treatments that are performing well and those that are not. Report 0-4517-2 proposed a field investigation plan for testing future candidate projects that combines visual inspections with nondestructive testing (NDT). A sequenced approach is proposed for NDT evaluation that includes Ground Penetrating Radar (GPR) and deflection testing and in some instances Dynamic Cone Penetrometer (DCP) testing. Deflections can be taken with either the Falling Weight Deflectometer (FWD) or Rolling Dynamic Deflectometer (RDD). Three treatments were found to be performing well. For JCPs without major failures, the use of an asphalt overlay should be considered. Good agreement was found between field reflection cracking performance and the results from laboratory testing with Texas Transportation Institute's (TTI's) overlay tester. Future overlay of JCPs should follow the overlay tester criteria proposed in this report. For sections with moderate levels of deterioration, with adequate support and no trapped moisture the use of rubblization is a good alternative. The rubblized concrete was found to provide a more uniform support than Crack and Seat. For those sections that are not candidates for rubblization because of poor slab support, then a flexible base overlay appears to be a good alternative. The flexible base overlays on US 59 in Lufkin and US 83 in Childress are performing very well.

Von Quintus, H., Mallela, J., Weiss, W. Shen, S. 2008. *Techniques for Mitigation of Reflective Cracking.* Report AAPTP 05-04. Auburn University, AL.

Abstract not available.

Von Quintus, H., Mallela, J., Weiss, W. Shen, S. 2008. *Techniques for Mitigation of Reflective Cracking.* Technical Guide AAPTP 05-04. Auburn University, AL.

Abstract not available.

Wang, H., Zhang, Q., and Tan, J. 2009. "Investigation of Layer Contributions to Asphalt Pavement Rutting.". Journal of Materials in Civil Engineering Vol. 21, No. 4, pp 181-185 (April 2009),. American Society of Civil Engineers, Reston, VA.

As one of the main distresses that may occur in asphalt pavements, rutting affects pavement performance the most significantly. This paper presents the investigation of the contributions of the structural layers to the total rutting in the asphalt pavements with semi-rigid bases through in situ excavation of transverse trenches and cores taken in the selected typical pavements on a real-world expressway. The selected pavements exhibited different depths of rutting. Laboratory rut testing was conducted on the core samples. Statistical analysis was undertaken to evaluate the effects of the layer properties sensitive to the variations of temperature on the rutting. The results indicated that the high temperature stability of the asphalt intermediate course had the greatest effect on the rut development. This paper presents the regression relationship established between rut depth and dynamic stability for each structural layer in asphalt pavements. In addition, the criteria for the dynamic stability are provided for asphalt base courses on different longitudinal grades.

Wang, Y., Mahboub, K., and Hancher, D. 2008. "Dynamic Panel Data Model for Predicting Performance of Asphalt Concrete Overlay.". Journal of Transportation Engineering Vol. 134 No. 2. American Society of Civil Engineers, Reston, VA.

In order to identify the improvement needs of transportation facilities and the associated budget, the transportation agencies need to have a reasonable estimate of the facilities' future condition. Using asphalt concrete overlay over fractured portland cement concrete pavement as an example, this paper shows how to use the existing pavement condition and other recorded information in a pavement management system to predict the future condition of pavements. A comprehensive pavement performance indicator, condition points, was used as the response variable of the prediction model. A dynamic panel data prediction method was employed to make the prediction model reflect the influence of the existing pavement condition, exogenous variables, and the heterogeneity of different pavement sections. The predicted condition points were compared with the actual values and the results indicated that the prediction accuracy was reasonable.

Weiss, R. 1980. *Pavement Evaluation and Overlay Design Using Vibratory Nondestructive Testing and Layered Elastic Theory. Vol. I. Development of Procedure.* ADA-087-186. Army Engineer Waterway Experiment Station ,Vicksburg, MS.

A procedure is developed for determining the allowable load-carrying capacities and the required overlay thicknesses of airport pavements. A layered elastic theory approach is used with vibratory nondestructive tests supplying the dynamic responses of pavements. For a given pavement, a computer program SUBE is used to determine the value of the subgrade Young's modulus from the measured dynamic responses, and a computer program PAVEVAL, which is based on the layered elastic theory, is used to calculate the allowable load-carrying capacity and the required overlay thickness. Limiting subgrade strains and horizontal stresses in pavement layers are used as criteria for determining load-carrying capacities and overlay thickness requirements. Single- and multiple-wheel loadings are considered. Vol. II of this report presents a validation of these procedures for three airport sites.

Wekmeister, S., and Alabaster, D. 2007. "Estimation of Remaining Pavement Life of Low-Vol. Roads with Falling Weight Deflectometer Results: A Practical Method.". Journal of the Transportation Research Board No. 1989, pp 261-269. Transportation Research Board. Washington, DC.

A practical method to estimate the remaining pavement life of low-Vol. roads is presented that uses the results of falling weight deflectometer (FWD) measurements. The method was developed from observations of accelerated pavement tests, and additional field results from Transit New Zealand?s asset management database are presented. Methods currently used in Australia and New Zealand to predict the remaining life of pavements are based on linear elastic backcalculation of FWD tests with the association of Australian and New Zealand road transport and traffic authorities (Austroads) approach, in which the life of the pavement is related to the vertical compressive strain at the top of the subgrade. The vertical compressive subgrade strain is usually determined by linear elastic backcalculation of FWD tests. Industry has highlighted significant concern about the Austroads procedure in New Zealand and whether the traditional FWD analysis with the Austroads subgrade strain criterion can predict pavement life accurately. In response to industry concerns, accelerated pavement test results from the Canterbury Accelerated Pavement Testing Indoor Facility were used to develop a new pavement performance criterion that predicts rutting of a low-Vol. road. The remaining-life prediction procedure is based on central deflections from the FWD measurement results and uses the change in FWD test results between the start and end of the postcompaction period.

Werkmester, S., Dawson, A., and Wellner, F. 2004. "Pavement Design Model for Unbound Granular Materials.". Journal of Transportation Engineering Vol. 130 No. 5, pp 665-674 (September 2004). American Society of Civil Engineers. Reston, VA.

This paper reports the results of several repeated load triaxial tests performed on a crushed rock aggregate at different stress levels. The development of the resulting permanent deformation that accumulates with the repeated loading is described and compared with the types of responses usually described by the shakedown approach. It is shown that the existing shakedown approach can describe some, but not all, of the observed responses. Thus, a modified set of possible responses is defined in shakedown terms and some explanations of the differences from the conventional approach are given. The elastic shakedown limit can be used to predict whether or not stable behavior occurs in the unbound granular material layer of a pavement construction or whether rutting will occur. With this information a pavement's granular base may be dimensioned. The elastic shakedown limit of a Granodiorite has been used in this manner and the resulting design compared with an empirical German design procedure. The paper concludes with an assessment of the benefits and limitations of the new approach.

Witczak, M W. 2008. "Development of Performance Related Specifications for Asphalt Pavements in the State of Arizona.". Report No. FHWA-SPR-08-402-2. Federal Highway Administration; Arizona Department of Transportation. Phoenix, Arizona.

This report presents an Executive Summary of a comprehensive study conducted by Arizona State University regarding a series of 11 separate projects relating to the implementation of the Mechanistic-Empirical Pavement Design Guide (MEPDG) for the state of Arizona. The individual study project reports deal with the characterization of a variety of AC binder types used by the Arizona Department of Transportation (ADOT) (Project 2); characterization results of E* Master Curve results for typical AC mixtures used in Arizona (Project 3); the characterization of these typical AC mixtures for Thermal Fracture (Project 4), Permanent Deformation (Project 5), and Load associated Fatigue (Project 6). In addition, recommendations are made in Project 7 regarding the implementation of the Simple Performance Test for AC Mixtures. Projects 8 and 9 focus on unbound bases/subbases and subgrades. Project 8 describes the results of nonlinear modulus of resilience response for a variety of typical unbound bases and subgrades. Project 9 deals with the development of an unbound material permanent deformation database and development of a more universal permanent strain model. In the Project 10 study report, the state of Arizona has been subdivided on the basis of relatively unique climatic regions based upon regional geomorphology. Finally, Project 11 is a very comprehensive report study of the existing ADOT traffic files for eventual inclusion with the MEPDG. A computerized (spreadsheet) traffic database of the entire ADOT highway network was conducted. This database incorporates every mile of the Arizona highway network (6 Interstates, 13 U.S. Highways and 86 state highways). Four significant traffic factors are included in the database: Average Annual Daily Traffic (AADT), Annual Growth Rate, Percent Trucks, and Vehicle Classification Percentage (VCP). Relatively homogeneous traffic units were selected based on existing ADOT HPMS and VCP stations.

Wu, Z., Chen, X., Gaspard, K., and Zhang, Z. 2008. "Structural Overlay Design of Flexible Pavement by Nondestructive Test Methods in Louisiana.". ***Proceedings*****: Transportation Research Board 87th Annual Meeting (January 2008). Transportation Research Board. Washington, DC.**

The Louisiana Department of Transportation and Development (DOTD) currently uses empirical book values in the overlay thickness design. This can lead to errors since the values do not necessarily represent actual field conditions. The objective of this study was to establish an overlay thickness design method for flexible pavement in Louisiana based on in-situ pavement conditions and non-destructive test methods. Four overlay rehabilitation projects with different soil subgrade types and traffic levels were selected for this study. On each pavement section, nondestructive deflection tests (NDT) including Falling Weight Deflectometer (FWD) and Dynaflect were performed at a 0.1-mile interval, and a detail condition survey data including cracking, rut depth, International Roughness Index (IRI), mid-depth temperature, and pavement thickness was also collected. Four NDT-based overlay design methods, including the 1993 AASHTO procedure, were investigated and used in the design of required overlay thicknesses. Results indicated that the 1993 AASHTO procedure method was generally over-estimated the effective structural No. for existing pavements, which would result in an under-designed overlay thickness. On the other hand, other NDT methods, such as Asphalt Institute MS-17, Arkansas ROADHOG and ELMOD5, were found not directly applicable to Louisiana pavement condition. Therefore, a modified NDT overlay design method based on the 1993 AASHTO pavement design guide was proposed in this study. After comparing with the current DOTD overlay design and new AASHTO Mechanistic-Empirical (M-E) design methods, the proposed NDT-based overlay design method was found to better reflect the in-situ pavement structural condition and be more effective in a routine use.

Xu, Q., and Mohammad, L N. 2008. "Modeling Asphalt Pavement Rutting Under Accelerated Testing", Road Materials and Pavement Design Vol. 9 No. 4, pp 665-687, Hermes Science Publications, France.

This paper describes a mechanistic-empirical (ME) model for simulating the rutting depth of asphalt pavement under accelerated testing. The model empirically correlates the layer rutting depth to the cyclic resilient deformation of the pavement layers. Finite element modeling was used to determine the layer deflection. The ME model was implemented for a full-scale accelerated loading facility (ALF) field test. Results showed that thickness and modulus of pavement layer have significant influence on rutting depth.

Zhang, H., Zhang, Q., and Tan, J. 2007. "Effect of Asphalt Pavement Layers on Rutting Development", *Proceedings*: International Conference on Transportation Engineering (July 2007). American Society of Civil Engineers, Chengdu, China.

Rutting is one of main damages in the asphalt pavement. The features of rutting in each asphalt pavement layer were analyzed by surveying the rutting depth, excavating transverse profile section and boring samples in the typical sections of expressways in which different degree rutting damages appeared. With the combination of the indoor rutting test on the sample from a corresponding location of road shoulders, the influence of the high temperature stability of each asphalt pavement layer on rutting was analyzed with the method of ANOVA (Analysis Of Variance). The result indicated that the stability of the middle layer impacted the rutting development most. Finally, the relationship between the RD (rutting depth) and the DS (dynamic stability) of each asphalt pavement layer was built with the nonlinear regression method. The controlling criterion of the DS of the bottom layer asphalt mixture has been put forward according to the regression equation.

Zhang, J., and Huang, X. 2008. "Analysis of Asphalt Pavement Rut Based on Elastic-viscoplastic Theory.". Journal of Highway and Transportation Research and Development , Vol. 3 No. 1, pp 39-42. Research Institute of Highway. Beijing, China.

In this work, elastoplastic theory was applied to study the permanent deformation of asphalt mixture, and a creep model was adopted for its good agreement with the actual results. ABAQUS, the finite element modeling technique, was used to simulate and investigate rut of the asphalt pavements with flexible base, and then the results were validated according to the circular track test. It is indicated that: rut increases fast in early age, but more slowly in later stages; pavement rut consists primarily of absolute rut, and lateral upheaval only accounts for 15-30%; deformation rate increases as the depth increases and achieves the largest value at the depth of 8 cm and then decreases gradually; and pavement absolute rut is composed of rut of the layer within the depths of 4-20 cm, especially within the depths of 4-12 cm. In addition, pavement ruts were simulated under different loads, and the axle load conversion indices were obtained by the rut equivalent method. or the adopted structures the index is 5.9.

Zhou, F. and Scullion, T. 2007. *Guidelines for Evaluation of Existing Pavements for HMA Overlay*. Report No. FHWA/TX-07/0-5123-2. Texas Transportation Institute, College Station, TX; Texas Department of Transportation, Austin, TX; Federal Highway Administration, Washington, DC.

This report discusses the application of nondestructive test (NDT) tools for evaluating existing pavements for hot-mix asphalt (HMA) overlays. The NDT tools covered in this report include ground penetrating radar (GPR), falling weight deflectometer (FWD), and rolling dynamic deflectometer (RDD). The GPR is used to estimate the thickness of existing pavement layers, and identify section breaks and potential trapped moisture problems. The FWD is used to evaluate the structural capacity of the existing pavement, and the in-situ layer modulus can be backcalculated from FWD data. In addition, for existing concrete pavements, the FWD can be used to determine load transfer efficiency (LTE) at joints and/or cracks. The application of the RDD to evaluate existing concrete pavements is also discussed. The major advantage the RDD has over other discrete NDT devices (e.g., FWD) is that it provides continuous deflection profiles of the pavement, which can be used to identify joints with poor LTE. However, no software is available to automatically interpret the RDD data. After reviewing RDD data collected on several different concrete pavements, the researchers developed some basic interpretation criteria for the RDD data. Based on the measured RDD deflection data and the monitored field reflective cracking performance on IH20, threshold values for RDD Sensor 1 deflection and the differential deflection between Sensors 1 and 3 are recommended. If either the Sensor 1 deflection or the differential deflection between Sensors 1 and 3 is larger than the proposed thresholds, the corresponding joint and/or cracks is recommended for pretreatment before placing a new HMA overlay. Finally, general guidelines for evaluating existing pavements for HMA overlays are proposed in this report.

Zhou, F., Hu, S., Chen, D., and Scullion, T. 2007. "Overlay Tester: Simple Performance Test for Fatigue Cracking.". Journal of the Transportation Research Board No. 2001, pp 1-8. Transportation Research Board. Washington, DC.

It is well known that fatigue cracking is not just a material problem; it also correlates highly with pavement structure and environmental and traffic conditions. This unique characteristic highlights the importance of a simple performance test (SPT) that can be used for daily hot-mix asphalt (HMA) mixture design and also for predicting fatigue cracking. This paper investigates the use of the Texas Transportation Institute's overlay tester (OT) as an SPT for fatigue cracking. Fatigue cracking occurs in two stages: crack initiation and crack propagation. However, as

noted in this paper, crack initiation is theoretically related to crack propagation. Therefore, the OT that characterizes mainly crack propagation can be used for fatigue cracking. To validate experimentally the OT for fatigue cracking, the OT results were compared with laboratory bending beam fatigue test and field accelerated pavement fatigue test results from the FHWA's accelerated loading facility test site. Positive results were obtained. In addition, other requirements (e.g., cost-related) for being an SPT are discussed. On the basis of all findings, the OT meets most requirements for being an SPT for fatigue cracking. Potential application of the OT to HMA mixture design also is proposed.

Zhou, F., Hu, S., and Scullion, T. 2006. "Integrated Asphalt (Overlay) Mixture Design, Balancing Rutting and Cracking Requirements.". Report No. FHWA/TX-06/0-5123-1. Federal Highway Administration. Washington, DC; Texas Department of Transportation, Austin, Texas.

The focus of this project is to develop an integrated hot-mix asphalt (HMA) mixture design method which balances both rutting and cracking requirements. The Hamburg Wheel Tracking Test (HWTT) and Overlay Tester (OT) devices were used to evaluate the rutting and cracking resistance of HMA mixtures, respectively. Eleven mixtures commonly used in Texas were designed following the current Texas Department of Transportation (TxDOT) mixture design process and then evaluated under the HWTT and the OT. It was found that the Dense-Graded and Superpave mixtures designed following current TxDOT mixture design procedures were rut resistant, but generally not crack resistant. However, all three Stone-Matrix Asphalt (SMA) mixtures were both rut and crack resistant. These observations are consistent with the past experience and field performance. The balanced design procedure proposed in this project recommends minor changes to TxDOT's current mixture design procedure. Seven mixtures including dense graded and Superpave mixtures were used to verify and demonstrate this balanced mixture design approach. It was found that a balanced HMA mixture could always be designed providing the aggregates used were not highly absorptive. Statistic analyses on the OT results showed that Performance-Grade (PG) of asphalt binder, effective asphalt content in Vol. (VBE), film thickness (FT), and surface area (SA) had significant impact on crack resistance of mixtures. Note that the influence of asphalt absorption by aggregates was included in the VBE and FT. The influence of air void content was not significant on crack resistance. Similarly, statistic analyses indicated that the following factors had significant influence on rutting resistance: 1) PG, 2) voids in the mineral aggregate, 3) FT, 4) SA, and 5) air void content. Additionally, the minimum and maximum asphalt contents for different mixtures to pass the cracking and rutting criteria were recommended based on extensive laboratory testing results. The recommended values were preliminarily verified by field performance data from IH20, WesTrack, and National Center for Asphalt Technology test track. Furthermore, a simplified version of the balanced HMA mixture design procedure was also proposed. Instead of Vol.tric design, trial asphalt contents for different mixtures were recommended for performance evaluation in the simplified mixture design procedure. The procedure was verified in two case studies.