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16. Abstract

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Advisory Circular (AC) 150/5320-6D (1995), including changes 1, 2, and 3 (2004), and related references, including some unpublished FAA technical reports and full-scale test results, were reviewed.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 Much of the airport pavement surveyed data collected from previous FAA projects, including some unpublished ones, was also reviewed.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.

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LIST OF ACRONYMS AND SYMBOLS

2D	Two-dimensional
2S	Dual-tandem wheels
3D	Three-dimensional
AC	Advisory Circular
ASTM	ASTM International
CBR	California Bearing Ratio
CC1	Construction cycle 1
CC3	Construction cycle 3
CDF	Cumulative damage factor
CFR	Code of Federal Regulations
D	Durability crack
d/2	Dual-tandem landing gear geometry factor
DSCI	SCI deduct
ESWL	Equivalent single-wheel load
FAA	Federal Aviation Administration
FEM	Finite element method
FOD	Foreign object debris
GAO	Government Accounting Office
HMA	Hot mix asphalt
IAD	Washington Dulles International Airport
JFK	John F. Kennedy International Airport
LTD	Load transfer efficiency with respect to deflection
MEM	Memphis International Airport
msf	Million square feet
MWHGL	Multiple-wheel heavy gear load
NAPTF	National Airport Pavement Test Facility
NDT	Nondestructive test
PCC	Portland cement concrete
PCI	Pavement Condition Index
PHX	Phoenix Sky Harbor International Airport
PMS	Pavement maintenance system
R/σ	Design Factor
R_C	Compressive strength of concrete
R_F	Flexural strength of concrete
SCI	Structural Condition Index
USACE	U.S. Army Corps of Engineers
WES	U.S. Army Waterways Experiment Station

EXECUTIVE SUMMARY

The objective of this study was to evaluate whether the Federal Aviation Administration (FAA) thickness design standards for flexible and rigid pavements are consistent with the FAA's standard for a 20-year pavement design life requirement using the most up-to-date available information on the life of airfield pavements. This report concludes that airport pavements designed following FAA Advisory Circular (AC) 150/5320-6D have sufficient thickness to provide the required 20-year structural life if other related standards for material preparation, construction, and maintenance are appropriately applied. This conclusion was based on published reports, papers, and numerical analysis of available data using structural and failure models. Comments from representatives of the American Concrete Pavement Association, the Asphalt Institute, and the National Asphalt Pavement Association were obtained and are included in the report.

The FAA's requirement for a 20-year pavement design life is contained in AC 150/5320-6D. A 20-year pavement life represents the total anticipated load applications the pavement will be subjected to over a 20-year period. For the purpose of thickness design, the total number of load applications is the defining parameter. The common terminology that uses years as the measure of pavement life is simply the practical period of time for which the design assumptions are considered valid. The FAA pavement thickness design procedures refer to the determination of the thickness of a pavement structure and its components (surface, base, and subbase layers) not to the design of the materials in a pavement (e.g., hot mix asphalt or Portland cement concrete mixes). The intent is, for flexible pavements, to protect the lower layers, particularly the subgrade, from shear failure, and for rigid pavements, to protect the Portland cement concrete top layer from fatigue cracking.

One of the major problems in pavement evaluation is determining when the pavement has failed. A unique definition of pavement overall failure does not exist; rigid and flexible pavements exhibit fundamentally different failure modes. The FAA design procedures are calibrated against full-scale test results in which the failure criteria are carefully defined. Flexible pavement failure is defined as shear failure in the subgrade leading to 1-inch upheaval at the pavement surface. Rigid pavement failure is defined as full-depth cracking in 50 percent of the slabs. Various aspects of pavement performance, life, and failure for rigid and flexible pavements are described. Functional and structural failures are differentiated and discussed.

Pavement thickness design and resultant pavement life can be significantly affected by the choice of design inputs. The results from a sensitivity study show that for flexible pavements, pavement life is most sensitive to aircraft gross weight, subgrade strength, and total thickness. For rigid pavements, pavement life is most sensitive to slab thickness, flexural strength of the concrete, and aircraft gross weight.

Generally, predicted pavement life is different from actual pavement life because the design assumptions can never accurately reflect the operational conditions because of uncertainties in material properties, climatic conditions, changes in traffic characteristics and volumes, etc. Pavement Condition Index (PCI) is used to evaluate pavement condition and is calculated from a visual survey of all types of distresses in a pavement that, in combination or alone, can cause the

pavement to reach a condition in which it can no longer perform its intended purpose. The distresses are separated into two broad categories: structural and functional. The FAA thickness design procedure is based on structural failure of pavements observed in full-scale tests in which the functional failure-related failure modes are eliminated, or suppressed, as much as possible. The functional failure-related distresses that may occur during a full-scale structural failure test are ignored in the rating of the structural condition of the test pavement.

Structural Condition Index (SCI) is defined as the structural component of PCI. In this report, PCI data from the field condition surveys were converted into SCI data to evaluate airport pavement structural life. For rigid pavements, there is a clear definition and procedure for calculating SCI. But, currently, there is no definition or established procedure to calculate the SCI of flexible pavements. This report introduces a procedure to calculate SCI for flexible pavements using two distresses—alligator cracking and rutting. However, the measured rut depth may include a portion of rutting in the surface hot mix asphalt layer. This distress is related to material and construction defects rather than structural defects. Therefore, the SCI values reported for flexible pavements may be lower than they should be (a lower SCI value indicates more distress).

The PCI data from the airport pavement field surveys were used to calculate SCI for flexible and rigid pavements at the end of 20 years of use, which is the FAA standard for design. The end of the design life of the pavement is also assumed to be indicated by the pavement having reached an SCI of 80. Analysis of the survey data showed, on average, that the SCI for both flexible and rigid pavements, and for all types of pavements (runways, taxiways, and aprons), is higher than 80 after 20 years. More structural distress is indicated for flexible pavements than for rigid pavements over all pavement types, as shown below. The computed PCI values show the same trends, except that the differences between flexible and rigid pavements are larger. Flexible pavement runway and taxiway PCI values are about the same, but flexible pavement runway and taxiway PCI values are about 13 percent lower than the rigid pavement runway and taxiway PCI values. This suggests that a discrepancy exists in the relationship between pavement thickness design standards and other FAA standards related to the 20-year design life requirement.

Pavement Type	SCI of Flexible Pavements after 20 Years	SCI of Rigid Pavements after 20 Years	Percentage Difference
Runways	89.5	92.4	3.1
Taxiways	83.4	89.5	6.8
Aprons	82.4	91.3	9.8

1. INTRODUCTION.

1.1 OBJECTIVE.

The objective of this study was to evaluate whether the Federal Aviation Administration (FAA) thickness design standards for flexible and rigid airfield pavements are in accordance with the FAA standard for 20-year pavement design life requirement using the most up-to-date available information on the life of airfield pavements.

1.2 SCOPE.

The FAA's requirement for a 20-year pavement design life is contained in FAA Advisory Circular (AC) 150/5320-6D. A 20-year pavement design life represents the total anticipated number of load applications the pavement will be subjected to over a 20-year period. For the purpose of thickness design, the total number of load applications is the defining parameter.

Consistent with the FAA's scope for this study, no new data were collected. Rather, currently available information from published reports and papers, as well as other available data, was used for analysis. Definitions of pavement life and pavement failure are provided and discussed in this study that include or exclude, as appropriate, factors other than the structural performance of pavements. The relationship between initial design assumptions and the conditions existing during the life of the pavement are also considered.

1.3 LIMITATIONS.

Constructing and maintaining a structurally and functionally sound pavement requires strict adherence to FAA standards and practices pertaining to pavement thickness design, material selection, construction, inspection, and maintenance. The FAA standards related to various aspects of airport pavements are as follows:

- AC 150/5320-6D—Structural design requirements
- AC 150/5370-10A—Construction and material requirements
- AC 150/5320-12C—Friction requirements
- AC 150/5360-8B—Maintenance requirements
- Title 14 Code of Federal Regulations (CFR) Part 139—Operational Requirements

If any of these standards are not followed, a pavement's full design life will not be realized. Interaction and proper application of each of these standards can have a significant impact on pavement life. The standards recognize that pavements fail for different reasons, e.g., deficiencies in performance related to structure, materials, construction, environment, and other. These can be broadly classified as structural (e.g., thickness), functional (e.g., skid resistance, material durability), and operational (e.g., surface condition) factors. For this study, it was assumed that the pavement's functional and structural requirements were addressed and the pavement was properly maintained over its design life. Therefore, this study emphasized a review of a pavement's structural performance with respect to the FAA's thickness design standards.

The FAA's pavement thickness design procedure refers to the determination of the thickness of the pavement and its components (e.g., surface, base, and subbase layers) not to the design of the materials in the pavement (e.g., hot mix asphalt (HMA) or Portland cement concrete (PCC) mixes). This study concentrated on thickness and structural design considerations and addressed the limitations of readily available data sources. For example, the Pavement Condition Index (PCI) number is a numerical indicator that provides a measure of the condition of the pavement at the time of inspection. The PCI is based on the distresses observed on the surface of the pavement, and it also indicates to some extent the pavement's structural integrity and surface operational condition. However, the PCI cannot measure the structural capacity or provide direct measurement of skid resistance or roughness, and it does not differentiate between different failure modes (i.e., functional or structural). To differentiate between structural or functional performance, the basic condition survey data are required, which are often not available. Also, information related to material-specific distresses such as alkali silica reactivity in concrete pavements and tender mixes in HMA pavements are more often not included or available.

Satisfying the FAA's 20-year life requirement also assumes continued, proactive maintenance over the life of the pavement. All pavements need to be maintained, even those that are properly designed. This also relates to a life cycle cost analysis, which is required during the design stage by AC 150/5320-6D. Life cycle costs include initial construction, periodic maintenance, and periodic rehabilitation costs, as well as salvage value (which is admittedly difficult to define). Due to the many different failure modes, an evaluation of life cycle costs was beyond the scope of this study. For example, it is very difficult to get objective maintenance cost data on a facility basis from airport owners. Also, it is not known if the maintenance was a result of functional or structural defects or which standard(s) was not followed properly. The proper identification of the defects driving the maintenance is critical in evaluating the efficacy of the various standards. Therefore, while all these factors are recognized, the limitations of this study prevented an objective analysis of maintenance and life cycle costs to be performed. Therefore, this study concentrated on the structural component (i.e., thickness design) of pavement performance and life.

1.4 APPROACH.

The basic approach for this study was as follows:

- Compile the most up-to-date available data sources on the structural performance of airport pavements.
- Recruit industry experts to review the preliminary work plans and the draft final report.
- Extract applicable information from the available data sources and reports.
- Present the data as it becomes available (e.g., PCI, Structural Condition Index (SCI), quotes, etc.).
- Present the results from a study of the sensitivity of pavement life to be changes in the design inputs.

1.5 OVERVIEW.

This report summarizes the published reports, papers, and any available data that were collected for ongoing FAA projects. Section 2 defines various aspects of pavement performance, life, and failure for rigid and flexible pavements and describes the PCI concept. Functional and structural failures are differentiated and discussed in section 2, along with examples and photographs. Section 3 is an overview of the design procedure development (flexible pavement, rigid pavement, and full-scale tests) and design assumptions (with respect to the subgrade support, traffic, material strength, and thickness). In section 4, the effect of failure models on pavement life is presented. Section 5 discusses the sensitivity of inputs to pavement life. Section 6 summarizes the findings (e.g., quotes, findings, and conclusions) extracted from the published and unpublished data sources, reports, and papers. Section 7 presents the summary of findings and conclusions from the study. Section 9 summarizes the comments from the industry reviewers (the Asphalt Institute and the American Concrete Pavement Association).

2. DEFINITIONS.

2.1 PAVEMENT LIFE AND FAILURE.

An airport pavement is a complex engineering structure. Pavement analysis and design involves the interaction of four equally important components, which are often difficult to quantify: (1) the subgrade (naturally occurring soil), (2) the paving materials (surface layer, base, and subbase), (3) the characteristics of applied loads, and (4) climate.

According to AC 150/5320-6D [1] paragraph 302 a., "Pavements designed in accordance with these standards are intended to provide a structural life of 20 years that is free of major maintenance if no major changes in forecast traffic are encountered. It is likely that rehabilitation of surface grades and renewal of skid resistant properties will be needed before 20 years due to destructive climatic effects and deteriorating effects of normal usage." The FAA pavement design procedure refers to the determination of the thickness of pavement and its components (surface, base, and subbase layers) not to the design of the materials in pavements (e.g., HMA or PCC mixes). The material and construction requirements are specified in AC 150/5370-10A [2].

Failure in pavements is not a phenomenon of chance, but a phenomenon that has a definite mechanical cause. When the pavement is incapable of performing the task it was designed for, it has failed. Failure could be structural (deep structure rutting, alligator cracking, longitudinal or transverse cracks in slabs, etc.) or functional (surface rutting, roughness, loss of skid resistance, etc.). A unique definition of pavement failure does not exist. ASTM International D 5340 [3] uses PCI to rate a pavement. A pavement with a PCI value less than 10 is defined as failed. The current FAA design specification, AC 150/5320-6D Change 3, 2004, paragraph 708 b., states, "An SCI of 80 is consistent with the current FAA definition of initial failure of a rigid pavement, i.e., 50 percent of the slabs in the traffic area exhibit initial structural cracking. The SCI allows a more precise and reproducible rating of a pavement's condition than previous FAA condition factor ratings, C_b and C_r." It is also important to point out that the decision to take a pavement out of service (or end of pavement life) depends not only on pavement condition, but also on the pavement functional type (runway, taxiway, or apron) and other subjective parameters.

Since the objective of this report was to evaluate the FAA design specification, the pavement structural life was defined as the period from when the pavement starts its service (SCI = 100) to the time when SCI = 80. The use of SCI = 80 is understood as the definition of the end of structural life for PCC pavements; however, the use of SCI = 80 to define the end of structural life of flexible pavement is a new concept that is not well established. Alligator cracking and rutting (load-related distresses) were used in this report to calculate the SCI of flexible pavements.

2.2 PAVEMENT DISTRESSES.

Distresses in a pavement structure leads to all types of pavement failure. Pavement distresses are external indicators of pavement deterioration caused by loading, environmental factors, construction deficiencies, or combinations thereof. Pavement failure could be structural or functional. Structural failure requires careful and detailed examination of the failure mechanism and the pavement layer contributing to the failure. Repairs are generally very expensive and the pavement may need reconstruction. Functional failures are generally easier and less expensive to fix. The type of distress in the pavement gives an insight to the type of failure, either structural or functional. In some cases, the two types of distresses interact with each other. For example, if untreated, functional failures may lead to or accelerate the structural failure of the pavement. The following sections describe the different types of distresses for HMA (flexible pavements) and PCC (rigid) pavements, respectively, as considered in an airfield pavement evaluation per ASTM D 5340-03 [3]. The definitions of distresses in the following sections were quoted from reference 3 and the photographs were taken from the Strategic Highway Research Program Distress Identification Manual [4].

2.3 FLEXIBLE PAVEMENT DISTRESSES.

Sixteen types of distresses are considered in flexible airport pavements. These distresses could be caused by material, environmental, construction, structural, or operational factors, and are described in the following sections.

2.3.1 Alligator Cracking.

A series of interconnecting cracks caused by fatigue failure of the HMA surface under repeated traffic loading is called alligator cracking, as shown in figure 2-1. High tensile strains at the bottom of the HMA layer under a wheel load initiate fatigue cracks. The crack propagates to the surface as a series of parallel cracks, which get interconnected with repeated loading. Alligator cracking, which is predominant in the wheel paths, is considered to be a major structural distress that can lead to structural failure of the pavement.

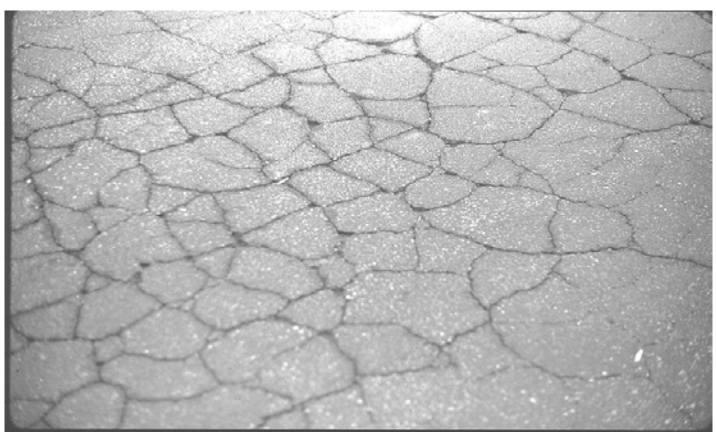


FIGURE 2-1. ALLIGATOR FATIGUE CRACKING

2.3.2 Bleeding.

Bleeding is a film of bituminous material on the pavement surface that usually becomes quite sticky. It is generally caused by excessive amounts of HMA or tars in the mix or low air void content, or both. It occurs when asphalt fills the voids of the mix during hot weather, then expands out onto the surface. It is caused by poor mix design and is a material-related distress. Bleeding leads to functional failure. Figure 2-2 shows high-severity bleeding.

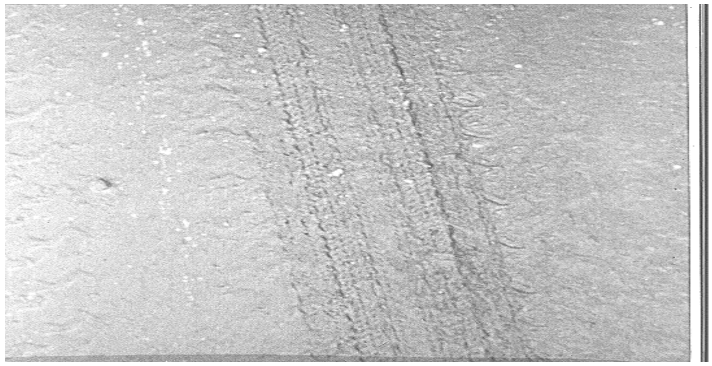


FIGURE 2-2. TIRE MARKS EVIDENT IN HIGH-SEVERITY BLEEDING

2.3.3 Block Cracking.

Block cracks are interconnected cracks dividing the pavement surface into approximately rectangular pieces (1 by 1 ft to 10 by 10 ft). It is generally caused by the shrinkage of the HMA and daily temperature cycling. It is not load-related and would classify as a distress caused by material and environmental factors. Block cracking normally occurs over a large portion of pavement area and indicates that the asphalt has hardened significantly. Block cracking causes functional failure of the pavement structure. Figure 2-3 shows high-severity block cracking.

FIGURE 2-3. HIGH-SEVERITY BLOCK CRACKING

2.3.4 Corrugation.

Corrugation is a series of closely spaced ridges and valleys (ripples) occurring at fairly regular intervals (usually less than 5 ft) along the pavement perpendicular to the direction of traffic. It is generally caused by the combined action of traffic and unstable pavement surface or base. Corrugation leads to functional failure of the pavement.

2.3.5 Depression.

Depressions are localized pavement surface areas having elevations slightly lower than those of the surrounding pavement. Depressions can be caused by settlement of the foundation soil, or it can be built during construction. Depressions could cause hydroplaning of the aircraft and leads to functional failure. Figure 2-4 shows a depression in a highway pavement.

FIGURE 2-4. DEPRESSION IN PAVEMENT

2.3.6 Jet-Blast Erosion.

Jet-blast erosion causes darkened areas on the pavement surface where the bituminous binder has been burned or carbonized. This type of distress is caused by operational factors.

2.3.7 Joint Reflection Cracking From PCC.

This type of distress occurs on pavements having an asphalt overlay on PCC slabs and is caused by the movement of the PCC slab beneath the HMA surface because of thermal and moisture changes, and in response to loading. It is therefore partially load-related, but it does not necessarily lead to further structural deterioration of the underlying structure and is considered to be a functional distress. Figure 2-5 shows high-severity joint reflection cracking.

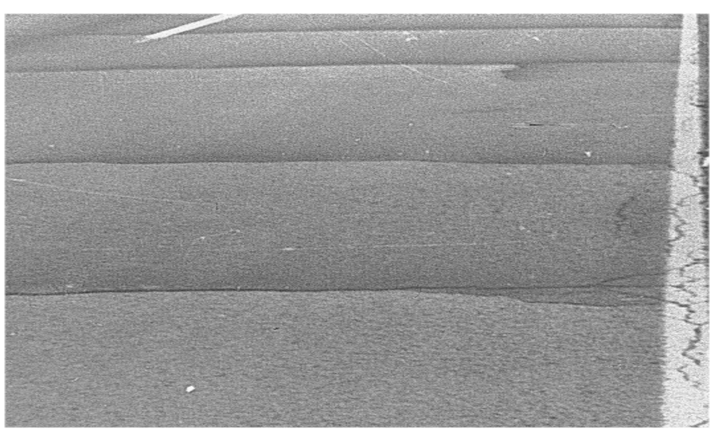


FIGURE 2-5. JOINT REFLECTION CRACKING FROM PCC

2.3.8 Longitudinal and Transverse Cracking (Non-PCC Joint Reflective).

The longitudinal cracks are parallel to the pavement centerline or lay down direction. They may be caused by (1) a poorly constructed paving lane joint, (2) shrinkage of the HMA surface due to low temperatures or the hardening of the asphalt, or (3) a reflective crack caused by cracks beneath the surface course, including cracks in the slabs (but not at the joints). Transverse cracks extend across the pavement approximately at right angles to the pavement's centerline or direction of lay down. They may be caused by (2) or (3) above. These types of cracks are not associated with loading. This type of distress is mainly caused by construction, material, and environmental factors. This type of distress leads to the functional failure of pavement. Figure 2-6 shows longitudinal and transverse cracking.

(a) Longitudinal Cracking (b) Transverse Cracking

FIGURE 2-6. HIGH-SEVERITY LONGITUDINAL AND TRANSVERSE CRACKING

2.3.9 Oil Spillage.

Deterioration or softening of the pavement surface caused by spilled oil, fuel, or other solvents is called oil spillage. Oil spillage is caused by operational factors and leads to functional failure.

2.3.10 Patching and Utility Cut Patch.

A patch is an area where the original pavement has been removed and replaced with either similar or different material. A patch is considered a defect, no matter how well it performs. Traffic load, material, and poor construction practices can cause patch deterioration. A patch is considered to be a function distress. Figure 2-7 shows a high-severity patch.

FIGURE 2-7. HIGH-SEVERITY PATCH

2.3.11 Polished Aggregate.

Aggregate polishing is caused by repeated traffic applications. The surface binder is worn away to expose coarse aggregate. Skid resistance is reduced and leads to functional failure. Figure 2-8 shows polished aggregate in a pavement surface.

FIGURE 2-8. POLISHED AGGREGATE

2.3.12 Raveling and Weathering.

Wearing away of the pavement surface caused by dislodging of aggregate particles and loss of asphalt binder leads to raveling and weathering. It indicates significant hardening of the asphalt binder and can cause severe foreign object debris (FOD) problems and leads to functional failure. Figure 2-9 shows raveling and weathering of a pavement surface.

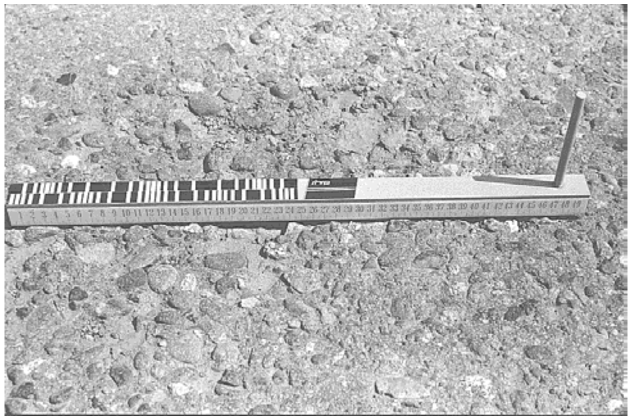


FIGURE 2-9. HIGH-SEVERITY RAVELING AND WEATHERING

2.3.13 Rutting.

Rutting is a longitudinal surface depression in the wheel path. Pavement uplift may occur along the sides of the rut. Rutting is caused by the permanent deformation occurring in any of the pavement layers or subgrade, which is usually caused by consolidation or lateral movement of the materials due to traffic loads. Significant rutting can lead to major structural failure of the pavement. A rut depth of 1 inch is considered to indicate functional failure due to the ponding it can cause. Figure 2-10 shows rutting in a pavement.

FIGURE 2-10. RUTTING IN A PAVEMENT

2.3.14 Shoving .

Shoving is the longitudinal displacement of a localized area of the pavement surface, as shown in figure 2-11. Braking and turning aircraft are generally the cause. Also, PCC pavements occasionally increase in length where they adjoin flexible pavements. This growth shoves the asphalt pavement, causing it to swell and crack. The PCC slab growth is caused by a gradual opening of the joints, as they are filled with incompressible materials that prevent them from closing. Shoving is a materials-related distress, and a functional failure that can lead to structural failure.

FIGURE 2-11. SHOVING IN A PAVEMENT

2.3.15 Slippage Cracking.

Slippage cracking occurs when braking or turning wheels cause the pavement surface to slide and deform. The slippage cracks, shown in figure 2-12, are crescent- or half-moon-shaped cracks having two pointed ends away from the direction of traffic. It generally occurs when there is a low-strength surface mix or a poor bond between the surface and the next layer of pavement structure. This is classified as a functional distress that can lead to structural failure if not corrected.

FIGURE 2-12. SEVERE SLIPPAGE CRACKING

2.3.16 Swell Distress.

An upward bulge in the pavement's surface characterizes swell distress. It may occur sharply over a small area or as a longer, gradual wave. Either type of swell can be accompanied by surface cracking. It is generally caused by frost action in the subgrade or by swelling soil, but a small swell can occur on the surface of an asphalt overlay over PCC as a result of a blowup (refer to section 2.4.1) in the PCC slab. A swell is classified as a functional distress.

2.4 RIGID PAVEMENT DISTRESSES.

Common types of distress in concrete pavement are pumping, faulting, cracking, and joint deterioration. ASTM D 5340-03 [3] lists 15 distress types for jointed concrete pavements, which are described in the following sections. The distress definitions are the same for both plain and reinforced jointed concrete pavements, with the exception of linear cracking, which is defined separately for plain and reinforced jointed concrete pavements.

2.4.1 Blowup.

Blowups, as shown in figure 2-13, occur in hot weather, usually at transverse cracks or joints that are not wide enough to permit expansion of the concrete slabs. The infiltration of the incompressible materials into the joint space leads to insufficient width. When expansion cannot relieve enough pressure, a localized upward movement of the slab edges or shattering occurs in the vicinity of the joint. Blowups can also occur at utility cuts and drainage inlets. Blowups increase FOD potential and lead to functional failure.

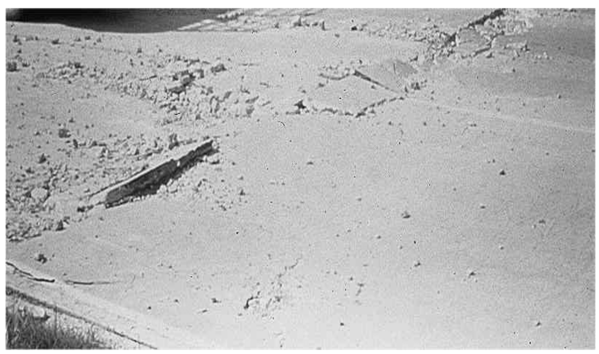


FIGURE 2-13. BLOWUP IN A PAVEMENT

2.4.2 Corner Break.

A corner break is a crack that intersects the joint at a distance less than or equal to one-half of the slab length on both sides, measured from the corner of the slab. Factors causing corner breaks are load repetitions combined with loss of support and curling stresses. Figure 2-14 shows a corner break.

FIGURE 2-14. CORNER BREAK

2.4.3 Longitudinal, Transverse, and Diagonal Cracks.

Longitudinal, transverse, and diagonal cracks divide the slab into two or three pieces and are generally caused by a combination of load repetitions, curling stresses, and shrinkage stresses. Low-severity cracks are usually warp- or friction-related and are not considered major structural distresses. Medium- or high-severity cracks are usually working cracks and are considered major structural distresses. Figures 2-15 and 2-16 show high-severity longitudinal and transverse cracking respectively. Low-severity cracks have little or minor spalling (no FOD potential). If nonfilled, they have a mean width less than approximately 1/8 inch. A filled crack can be of any width but the filler material must be in satisfactory condition. For medium-severity cracks, one of the following conditions must exist: (1) a filled or nonfilled crack is moderately spalled (some FOD potential); (2) a nonfilled crack has a mean width between 1/8 and 1 inch; (3) a filled crack is not spalled or lightly spalled, but the filler is in unsatisfactory condition; or (4) the slab is divided into three pieces by two or more cracks, one of which is at least medium severity. For high-severity cracks, one of the following conditions must occur: (1) a filled or nonfilled crack is severely spalled (definite FOD potential); (2) a nonfilled crack has a mean width greater than approximately 1 inch, creating potential tire damage; or (3) the slab is divided into three pieces by two or more cracks, one of which is at least high severity.

FIGURE 2-15. HIGH-SEVERITY LONGITUDINAL CRACKING

FIGURE 2-16. HIGH-SEVERITY TRANSVERSE CRACKING

2.4.4 Durability Cracking.

Durability (D) cracking is caused by the concrete's inability to withstand environmental factors such as freeze-thaw cycles. It appears as a pattern of cracks running parallel to a joint or a crack. This type of cracking can eventually lead to disintegration of the concrete within 1 to 2 feet of the joint or crack and is classified as a function distress. Figure 2-17 shows high-severity D cracking.

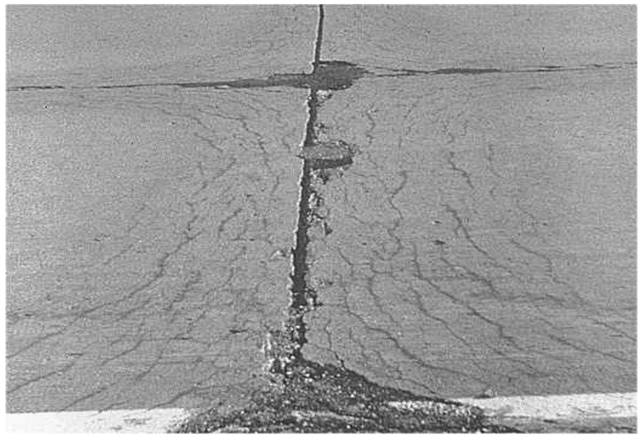


FIGURE 2-17. HIGH-SEVERITY D CRACKING

2.4.5 Joint Seal Damage.

Joint seal damage is any condition that enables soil or rocks to accumulate in the joints or allows a significant amount of water infiltration. Accumulation of incompressible materials prevents the slabs from expanding and may result in buckling, shattering, or spalling. Typical joint seal damage are (1) stripping of joint sealant, (2) extrusion of joint sealant, (3) weed growth, (4) hardening of filler, (5) loss of bond to the slab edges, and (6) lack or absence of sealant in the joint. Joint seal damage is classified as a functional distress. Figure 2-18 shows joint seal damage.

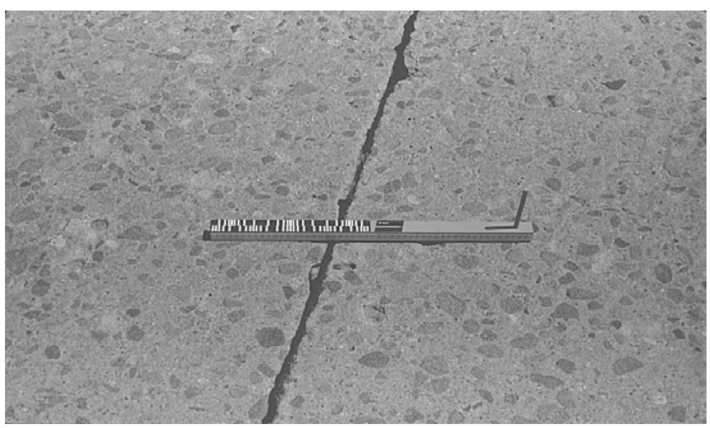


FIGURE 2-18. JOINT SEAL DAMAGE

2.4.6 Patch, Small (less than 5 ft^2).

A patch is an area where the original pavement has been removed or replaced by a filler material. Poor construction of the patch, loss of support, heavy load repetitions, moisture, and thermal gradients can all cause distress. A patch is classified as a functional distress. Figure 2-19 shows a small patch distress.

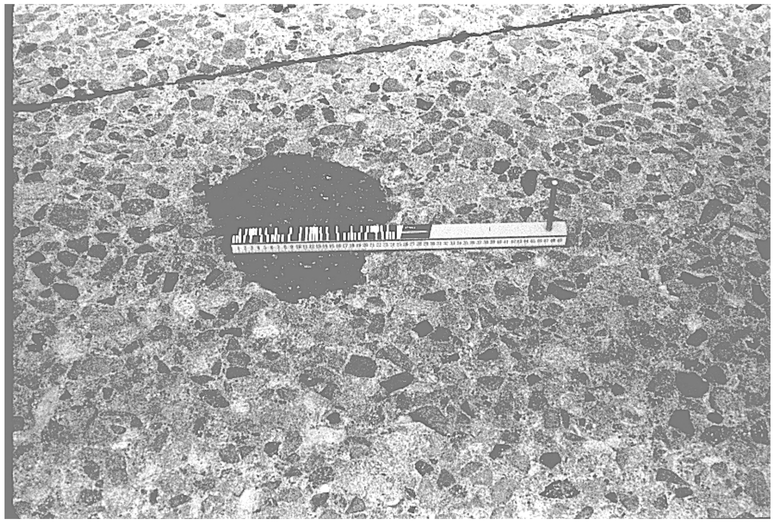


FIGURE 2-19. SMALL PATCH DISTRESS

2.4.7 Patch, Large (over 5 ft^2).

An example of a large patch could be a patch that has replaced the original pavement because of the placement of underground utilities and is classified as a functional distress. Figure 2-20 shows a large patch.

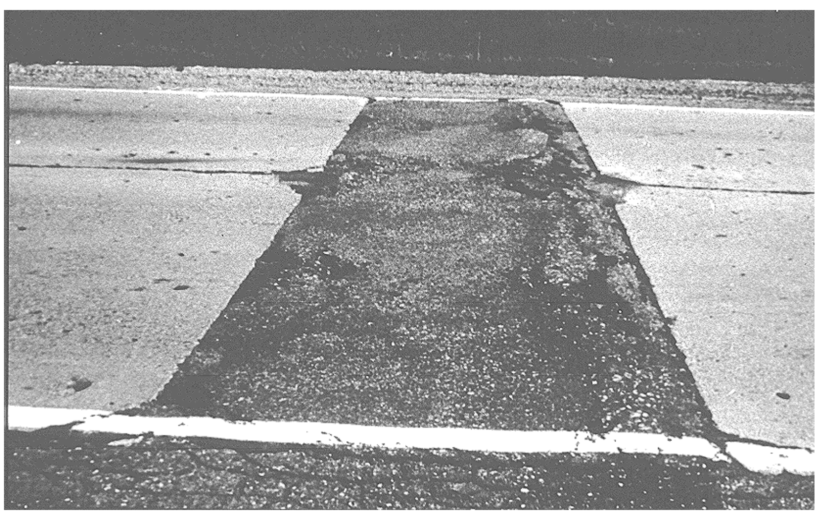


FIGURE 2-20. LARGE PATCH DISTRESS

2.4.8 Popouts.

A popout is a small piece of pavement that breaks loose from the surface due to a freeze-thaw action in combination with expansive aggregates, see figure 2-21. Popouts usually range from approximately 1 to 4 inches in diameter and from 1/2 to 2 inches in depth. This type of distress is generally caused by material and environmental factors. It is a functional distress (FOD) rather than a structural distress.

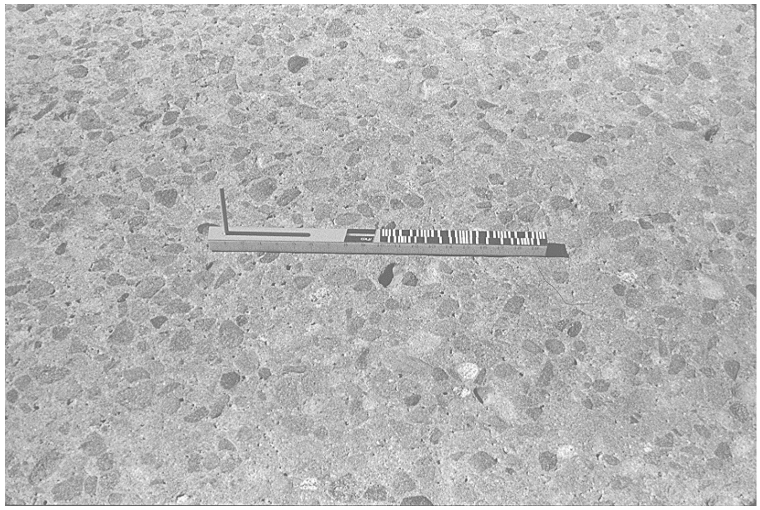


FIGURE 2-21. POPOUT DISTRESS

2.4.9 Pumping.

Pumping is the ejection of material by water through joints or cracks caused by the deflection of the slab under moving loads. As water is ejected, it carries particles of gravel, sand, clay, or silt, resulting in loss of pavement support. Pumping near joints indicates poor joint sealer and loss of support that will lead to cracking under repeated loads. Pumping can occur at cracks as well as joints and is classified as a functional distress. Figure 2-22 shows water bleeding and pumping.

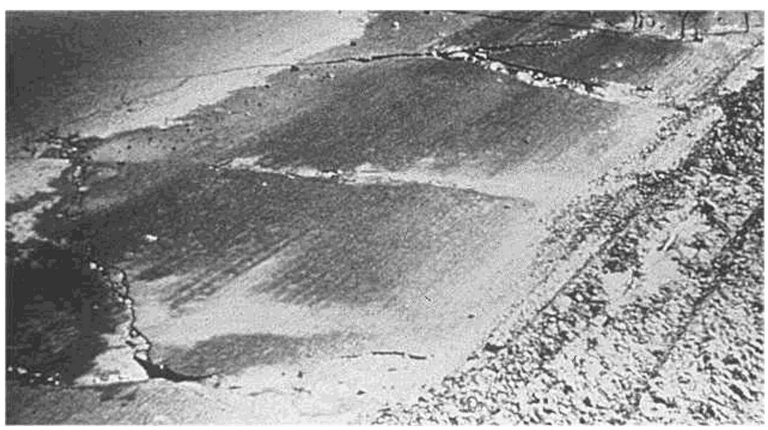


FIGURE 2-22. WATER BLEEDING AND PUMPING

2.4.10 Scaling, Map Cracking, and Crazing.

This distress refers to a network of shallow, fine, or hair-like cracks that extend only through the upper surface of the concrete. The cracks intersect at angles of 120°. It is generally caused by overfinishing the concrete and may lead to surface scaling. Breakdown of the slab surface occurs to a depth of 1/4 to 1/2 inch. Deicing salts, improper construction, freeze-thaw cycles, and poor aggregate may also cause scaling. These distresses are classified as a functional distress and are construction-related. Figures 2-23 and 2-24 show scaling and map cracking, respectively.

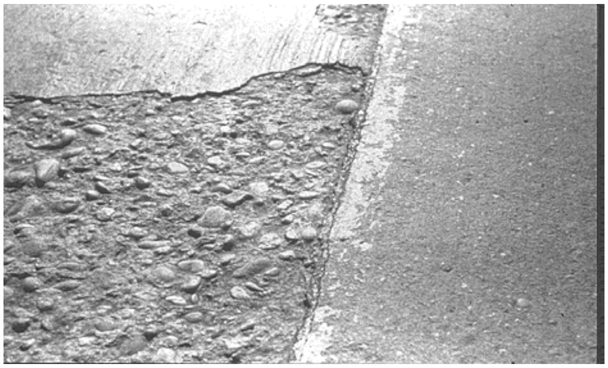


FIGURE 2-23. SCALING

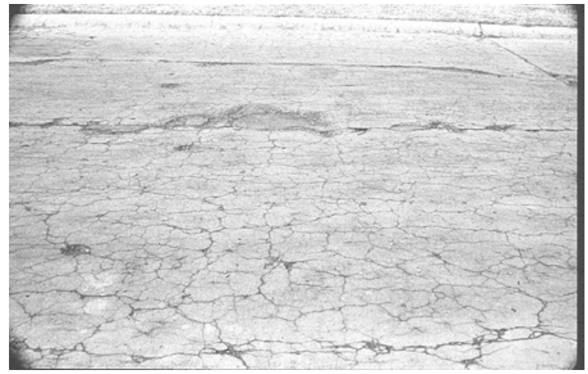


FIGURE 2-24. MAP CRACKING

2.4.11 Settlement or Faulting.

Difference of elevation at a joint or crack, caused by upheaval, consolidation, or a buildup of loose materials under the trailing slabs, is called settlement or faulting and is classified as a functional distress. Figure 2-25 shows faulting of transverse cracks.

FIGURE 2-25. FAULTING OF TRANSVERSE CRACKS

2.4.12 Shattered Slab/Intersecting Cracks.

Cracks that break the slab into four or more pieces due to overloading or inadequate support or both are called intersecting cracks, whereas a high-severity level of this distress is referred to as a shattered slab and is classified as structural failure. Figure 2-26 shows a shattered slab condition.

FIGURE 2-26. SHATTERED SLAB CONDITION

2.4.13 Shrinkage Cracks.

Shrinkage cracks are hairline cracks that are usually only a few feet long and do not extend across the entire slab. They are formed during the setting and curing of concrete and usually does not extend through the depth of the slab and is classified as a functional distress. However, shrinkage crack is also the term used to describe a load-induced crack that extends only part of the way across a slab. In this case, it is classified as a structural distress (see change 3 of reference 1).

2.4.14 Spalling (Transverse and Longitudinal).

Joint spalling is the breakdown of slab edges within 2 feet of the side of the joint. A joint spall does not extend vertically through the slab but intersects the joint at an angle. Spalling is caused by excessive stresses at the joint or crack caused by infiltration of incompressible materials or traffic loads or weak concrete at the joints (caused by overworking) combined with traffic loads and is classified as a structural distress. Figure 2-27 shows spalling at a transverse joint.

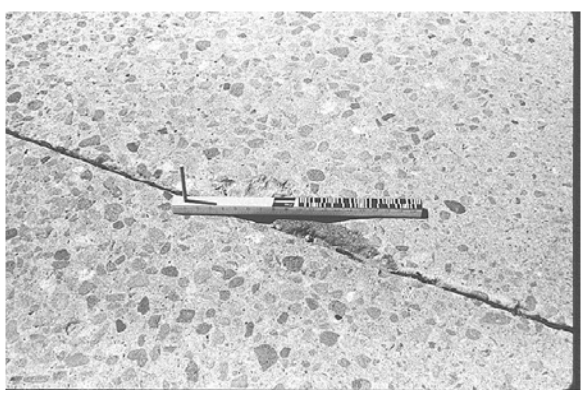


FIGURE 2-27. JOINT SPALLING AT A TRANSVERSE JOINT

2.4.15 Spalling (Corner).

Corner spalling is the raveling or breakdown of the slab within 2 feet of the corner. A corner spall differs from a corner break in that the spall usually angles across to intersect the joint, while a break extends vertically through the slab. A corner spall is classified as a structural distress. Figure 2-28 shows a corner spall.

FIGURE 2-28. CORNER SPALLING

2.5 ASSESSING PAVEMENT FAILURE.

One of the major problems in pavement evaluation is determining when the pavement has failed. According to the ASTM Standard Test Method for Airport Pavement Condition Surveys [3], a PCI value is computed based on the amount and severity of distresses (described in sections 2.3 and 2.4). The PCI value is a numerical rating of the pavement condition that ranges from 0 to 100, with 0 being the worst possible condition and 100 being the best possible condition. A pavement is defined as failed when the PCI value drops below 10. However, the reduction in the PCI value could be caused by distresses that lead to structural failure or distresses that lead to functional failure, or both, and airport pavements are typically considered to be unusable well before the PCI has reached a value of 10.

The FAA design procedure is based on the structural failure of pavements observed in full-scale tests [5]. For rigid pavements, the failure criterion is initial crack, which is defined as the condition when at least 50 percent of the slabs contain one or more cracks due to loading of the slab (see references 13 and 48). This corresponds to an SCI of 80, where the SCI is defined as the structural component of the PCI. The failure criterion for flexible pavements is based on 1 inch of upheaval (shear failure in the subgrade) outside the traffic path or significant cracking (cracking in HMA layer has occurred to such an extent that the pavement is no longer waterproof).

Since the FAA thickness design procedure is based on structural failure (rather than functional failure), only those factors or distresses that lead to structural failure were included in the evaluation of pavement life in subsequent sections of this report.

Table 2-1 summarizes the distresses in flexible and rigid pavements with the typical failure mechanism for each distress.

TABLE 2-1. DISTRESS MECHANISMS

Pavement	Distress	Mechanism	Failure Mode
HMA	Alligator Cracking	Load	Structural
	Bleeding	Materials	Functional
	Block Cracking	Climate	Functional
	Corrugation	Materials	Functional
	Depression	Construction	Functional
	Jet Blast	Operational	Functional
	Joint Reflection Cracking	Climate	Functional
	Longitudinal/Transverse Cracking	Climate	Functional
	Oil Spillage	Operational	Functional
	Patching	Other	Functional
	Polished Aggregate	Materials	Functional
	Weathering and Raveling	Climate	Functional
	Rutting	Load	Structural
	Shoving	Materials	Functional/Structural
	Slippage Cracking	Construction	Functional
	Swell	Other	Functional
PCC	Blow-up/Buckling	Climate	Functional
	Corner Break	Load	Structural
	Linear Cracking	Load	Structural
	D Cracking	Materials	Functional
	Joint Seal Damage	Climate	Functional
	Patch, Small	Other	Functional
	Patch, Large/Utility Cut	Other	Functional
	Popouts	Materials	Functional
	Pumping	Other	Functional
	Scaling/Map Cracking/Crazing	Construction	Functional
	Faulting	Other	Functional
	Shattered Slab	Load	Structural
	Shrinkage Cracking	Load/Materials	Structural/Functional
	Joint Spalling	Construction/Load	Structural
	Corner Spalling	Construction/Load	Structural

Of these, the following distresses are characterized as structural (i.e., load) related for the purpose of defining pavement life for consideration in the FAA thickness design standard:

Flexible Pavement	Rigid Pavement
Alligator cracking	Corner break
Rutting*	Linear Cracking
	Shattered Slab
	Shrinkage Cracking*
	Joint Spalling*
	Corner Spalling*

* These distresses could also be caused by nonstructural mechanisms.

Based on the above, this report used a procedure to convert the surveyed PCI data into SCI data for evaluation of airport pavement life. The details will be discussed in section 6. The following section discusses the factors affecting the structural life of airport pavements, overview of development of design procedures, and design assumptions.

3. PAVEMENT DESIGN PROCEDURES REVIEW AND EFFECTS OF THE RESPONSE MODEL ON PAVEMENT LIFE.

This section presents a review of conventional and mechanistic procedures used by the FAA for the design of flexible and rigid pavements. An understanding of the design theory is important in order to properly define and understand pavement structural life. It is also important to note that the failure mechanisms in flexible and rigid pavements are fundamentally different. Flexible pavement failure is typically defined as shear failure in the subgrade, leading to rutting at the pavement surface. Rigid pavement failure is typically defined as a function of cracking of the slabs. The failure criteria for each were developed from full-scale pavement tests dating from the 1940s to the present time.

3.1 FLEXIBLE PAVEMENTS.

3.1.1 Development of California Bearing Ratio Equation—Pavement as Single Layer.

The need for pavement design for heavier aircraft started in the 1940s. Richard Ahlvin chronicles the development of airport pavement design procedures in his report "Origin of Developments for Structural Design of Pavements" [6]. The earliest design method was based on an established empirical highway design method based on California Bearing Ratio (CBR). The CBR method was selected for the following reasons:

- It was correlated with service behavior and construction methods (1928-1942).
- It could be quickly and easily adapted to airfield pavement design.
- It was thought to be reasonable.
- It had been successfully used in states other than California.
- The subgrade strength (CBR) could be assessed using simple portable test equipment in the laboratory or in the field.
- Testing could be performed on soil samples representing field conditions.

3.1.1.1 Previous Full-Scale Tests and Studies Related to Flexible Pavement Design.

Design curves, using the single-wheel criterion, were developed for 7,000- and 12,000-lb wheel loads in 1942. Later on, using extrapolation, design curves were provided for 25,000-, 40,000-, and 70,000-lb wheel loads. At that time, only single-wheel aircraft existed, with B-17 and B-24 military bombers being the heaviest aircraft. The mathematical models for stress, strain, and deflection extended only to a single uniform layer (homogenous, isotropic, half-space) and only permitted special case (such as under the load center) determinations. A set of CBR versus thickness curves for single-wheel aircraft for wheel loads from 4,000 to 70,000 lb was derived. These design curves were verified and modified by extensive accelerated traffic tests on existing pavements of known composition and specially constructed full-scale test sections. Some minor adjustments were made in the high CBR range curves.

Design curves for 150,000- to 200,000-lb single-wheel loads were developed and verified using the results from Stockton No. 2 tests [7].

In the 1940s, single-wheel load design criteria were used to provide criteria for dual and dual-tandem landing gear loads by replacing the total strut load of a multiple-wheel gear with an equivalent single-wheel load (ESWL). Studies on pavement responses (stress and deflection) showed that at, and near, the pavement surface individual wheels act independently. At very large depths, the overlapping of wheel-load effects resulted in a similar stress and deflection, as would be induced by the total strut load acting on a single wheel. The depth at which interaction effects start to become significant was established to be half the spacing between the edges of the tire prints of duals or the dual half of dual tandems and given the designation d/2. The depth at which the total strut load acts as if it were applied on a single wheel was established to be twice the center-to-center distance between duals or twice the center-to-center distance of diagonally opposite wheels of dual tandems and given the designation duel-tandem wheel (2S). Between depths d/2 and 2S, the ESWL was represented a by log-log plot of load versus depth by a straight line relationship between the load on one wheel at a depth of d/2 and the total strut load at a depth of 2S. The stresses and deflections were determined using Newmark's charts.

Tests conducted in 1952 [8] led to the development of an improved method in which the ESWL is defined as the single-wheel load that causes the same deflection at some specified depth in the pavement as all wheels in the landing gear acting together. Deflections are calculated using Boussinesq's equation. The specified depth is usually the depth of the top of the layer to be protected from shear failure (subgrade, subbase, or base). This method, commonly referred to as the CBR method, was adopted by the U.S. Army Waterways Experiment Station (WES) and the FAA to replace the d/2-2S method for ESWL determination, but the d/2-2S method is still used by some organizations and countries.

In the 1940s, design curves were developed for 100-psi tire pressure using the d/2-2S method. With increasing aircraft weights and tire pressures, design curves were developed for tire pressures of 200 and 300 psi using the CBR method. Accelerated pavement testing confirmed the validity of these extrapolations [9 and 10].

The single-wheel curves were depicted as a set of curves by using the early CBR equation:

$$t = k\sqrt{P} \tag{3-1}$$

where

t = thickness of structure
P = wheel load
k = a constant for a particular CBR and tire pressure

The constant k varies inversely with CBR and increases somewhat with tire pressure. Also, it was realized that the interaction between load magnitude and repetitions is a significant factor for the purpose of pavement design and evaluation. The multiple-wheel tests of 1949-1950 [8] found the design methods based on d/2 and 2S to be reasonably satisfactory but a bit unconservative.

In 1956, the CBR equation (equation 3-1) was modified to include tire pressure and CBR directly, as shown in equation 3-2 [11 and 12].

$$t = \sqrt{P\left(\frac{1}{8.1\overline{CBR}} - \frac{1}{p\pi}\right)} \tag{3-2}$$

or

$$t = \sqrt{\frac{P}{8.1\overline{CBR}} - \frac{A}{\pi}} \tag{3-3}$$

where

t	=	thickness of structure, in.
P	=	wheel load or ESWL, lb
$\overline{CBR}$	=	the strength test rating value
A	=	tire contact area, in^2

Equations 3-2 and 3-3 represent the basic form of the CBR equation without the provision for variation in stress repetitions (coverages). According to reference 6, no specific date can be related to the acceptance of coverages as an element of design and evaluation applicable to the critical or design load. A 1949 study showed the trend of required thickness versus coverages. In the following years, the design criteria were accepted as representing a 5000 coverages useful life (nominally 20 years). The mathematical relationship relating percent of design thickness to coverages of critical load with 5000 coverages at 100 percent design thickness is shown in equation 3-4.

$$\%t\left[\frac{1}{100}\right] = 0.23 * LogC + 0.15 \tag{3-4}$$

(%t = 100 at C = 5000)

where C = coverages and log is the common or base 10 log. Equation 3-5 gives a more general form of the CBR equation:

$$t = (0.23 * LogC + 0.15)\sqrt{P\left(\frac{1}{8.1\overline{CBR}} - \frac{1}{p\pi}\right)} \tag{3-5}$$

With the advent of new aircraft such as the Boeing 747 and C-5, multiple-wheel heavy gear load (MWHGL) tests were carried out at WES to verify and modify greatly extrapolated design and evaluation criteria. The failure criteria for the pavements were excessive cracking in the HMA layer or 1-inch upheaval outside the traffic lane. The upheaval is indicative of shear failure in the subgrade or any one or more of the other layers in the pavement that are composed of unbound materials. The results of these tests [5] were used to modify equation 3-5 (for C = 5000), and a third degree form of the CBR equation was developed as follows:

$$\frac{t}{\sqrt{A}} = -0.0481 - 1.1562\left[Log\frac{\overline{CBR}}{p}\right] - 0.6414\left[Log\frac{\overline{CBR}}{p}\right]^2 - 0.4730\left[Log\frac{\overline{CBR}}{p}\right]^3 \quad (3\text{-}6)$$

The α-factor was used to adjust thickness for other than 5,000 coverages.

$$\frac{t}{\sqrt{A}} = \alpha\left\{-0.0481 - 1.1562\left[Log\frac{\overline{CBR}}{p}\right] - 0.6414\left[Log\frac{\overline{CBR}}{p}\right]^2 - 0.4730\left[Log\frac{\overline{CBR}}{p}\right]^3\right\} \quad (3\text{-}7)$$

3.1.1.2 Advisory Circular 150/5320-6D for Flexible Pavement Design.

The FAA design procedures (AC 150/5320-6D [1]) are based on the WES criteria (CBR method), as described above. Full-scale test data and observations of in-service pavements were used to determine pavement thicknesses necessary to protect subgrades with various CBR values from shear failure. These thicknesses were developed for single-wheel loadings. For gear assemblies other than single wheel, the ESWL for the assembly was computed based on deflection. Once the ESWL was established, the pavement section thickness was determined from relationships given in equations 3-6 and 3-7. The load repetitions are represented on the design curves in terms of annual departures, which are assumed to occur over a 20-year life. The departures are then converted to coverages. Each pass (departure) of an aircraft is converted to coverages using a single pass-to-coverage ratio, which is developed assuming a normal distribution and applying standard statistical techniques. (A full description of the pass-to-coverage concept is given in reference 5, together with examples of calculating numerical values.) The pass-to-coverage ratios used in developing the flexible pavement design charts in AC 150/5320-6D are listed in table 3-1. Annual departures are converted to coverages by multiplying by 20 and dividing the product by the pass-to-coverage ratios listed in table 3-1. The pavement section thickness determined from a design chart is multiplied by the appropriate load repetition factor (α-factor) to give the final pavement thickness required for various traffic levels.

The design procedure is based on a design aircraft. In a mix of aircraft (for which the pavement is being designed), the design aircraft is the aircraft requiring the greatest pavement thickness. The design aircraft is not necessarily the heaviest aircraft in the mix.

In the CBR method, the calculation of ESWL assumes that pavement layers all have the same stiffness. Equivalency factors are used to convert the thickness of one type of material to another. With the advent of the tridem landing gear configuration (B-777), it was found that, due to the high degree of interaction among individual wheel deflections, the CBR-based design procedure gives very conservative thickness designs. An alternative design procedure, based on a layered elastic analysis model, was developed.

TABLE 3-1. PASS-TO-COVERAGE RATIOS FOR FLEXIBLE PAVEMENTS

Design Curve	Pass-to-Coverage Ratio
Single Wheel	5.18
Dual Wheel	3.48
Dual Tandem	1.84
A300 Model B2	1.76
A300 Model B4	1.73
B-747	1.85
B-757	1.94
B-767	1.95
C-130	2.07
DC 10-10	1.82
DC 10-30	1.69
L-1011	1.81

3.1.2 Layered Elastic Design Procedures.

In 1995, the FAA published AC 150/5320-16 [13] for the thickness design of airport pavements serving the B-777 airplane. In 2004, AC 150/5320-16 was cancelled and the layered elastic design procedures were included in chapter 7 of AC 150/5320-6D. The use of the LEDFAA design procedures is now approved for all aircraft types and traffic mixes containing aircraft with gross weights in excess of 30,000 lb. Thickness design for pavements not subject to B-777 aircraft can also be determined using the design charts in chapter 3 of AC 150/5320-6D as previously discussed. The layered elastic failure criterion was revised based on recent full-scale tests at the National Airport Pavement Test Facility (NAPTF).

3.1.2.1 Full-Scale Tests at the NAPTF.

The design of the NAPTF was primarily determined by the need to develop pavement design procedures for the new generation of large civil transport aircraft, including the B-777, B-747 stretch, and the Airbus A380 class aircraft. Current design procedures predict a significant amount of interaction between the loads from multiple wheels and closely spaced trucks that will be used on these aircraft, particularly within the subgrade in flexible pavements. But the true degree of interaction is not known, and measurements from full-scale tests are required to determine how closely wheels and trucks can be spaced without significant load interaction. The NAPTF is located at the FAA William J. Hughes Technical Center, Atlantic City International Airport, New Jersey. A 1.2-million-lb pavement test vehicle spans two sets of railroad tracks that are 76 feet apart. The vehicle is equipped with six adjustable dual-wheel loading modules. A hydraulic system applies the load to the wheels of the modules. The major specifications for the test pavement area are as follows:

- Test pavement is 900 feet long by 60 feet wide.

- Nine independent test items (six flexible and three rigid) along the length of the track. The pavement cross-sectional details can be found on the FAA Airport Technology Research and Development Branch web site: www.airporttech.tc.faa.gov.

- Twelve test wheels capable of being configured to represent two complete landing gear trucks having from two to six wheels per truck and adjustable up to 20 feet forwards and sideways.

- Wheel loads adjustable to a maximum of 75,000 lb per wheel.

The primary objectives of the testing were to

- provide additional traffic data for incorporation in new thickness design procedures under development by the FAA.

- provide full-scale pavement response and failure information for use in airplane landing gear design and configuration studies.

- re-examine the CBR method of design for flexible pavements.

All three of these objectives were established with particular reference to the level of pavement damage expected from the six-wheel landing gear on the B-777 airplane relative to the dual and dual-tandem landing gears used on aircraft in the remainder of the fleet.

The failure criteria for the flexible test items are the same as those defined in reference 5 for the U.S. Army Corps of Engineers (USACE) MWHGL test series. The pavement was defined as failed when either of the following conditions occurred:

- One-inch upheaval outside the traffic lane, signifying structural shear failure in the subgrade or other supporting layers

- Surface cracking to the point that the pavement is no longer waterproof, signifying complete structural failure of the surface layer

The results from the full-scale flexible pavement testing at the NAPTF are summarized in reference 14. The construction cycle one (CC1) flexible pavement test items were constructed with both conventional crushed stone base (P-209) and asphalt stabilized base (P-401 Base). The conventional structures had base courses consisting of 8 inches of FAA specification P-209, crushed aggregate base course, and the stabilized base structures had base courses consisting of 5 inches of FAA specification P-401, plant mix bituminous pavement. Full structural failure did not occur in the low-strength CC1 test items, probably because the subgrade material contained a significant amount of silt and the upper layers of the subgrade dried somewhat over the long period of time between construction and the start of traffic testing. Subgrade strengths of approximately 5 CBR were measured after testing was completed instead of the 3 CBR design target. Therefore, results for only the medium-strength test items, which did show the expected subgrade shear failures, were used in the modification of the failure model. During construction cycle 3 (CC3), four new flexible pavements with 5-inch P-401, 8-inch P-209 crushed stone base, and P-154 subbase (16, 24, 34, and 43 inches for LFC-1, LFC-2, LFC-3, and LFC-4 respectively) were constructed. Test items LFC-1, LFC-2, and LFC-3 failed during traffic testing, and the data from these test items were used for the modification of the failure model.

For updating the flexible pavement failure model in the LEDFAA computer program for airport pavement thickness design, a database of existing full-scale test data for heavy airplane wheel loads was assembled, and the CC1 and CC3 test results were added. The vertical strain at the top of the subgrade was then calculated for each data point using the LEDFAA structural model. Material properties were based on best estimates of the data provided in the reports of the full-scale tests. The pass-to-coverage ratio is calculated in LEDFAA for an equivalent wheel width projected onto the top of the subgrade; the same procedure is used for constructing the LEDFAA failure model. Consequently, coverages to failure in LEDFAA are not the same as coverages to failure in the CBR method, as discussed below.

Figure 3-1 shows the results of vertical strain plotted against coverages to failure on a log-log scale. The NAPTF results match well with the existing data, and the six-wheel data points do not show significant deviation from the rest of the data points (ideally, whatever the gear geometry, wheel load, and structural properties, all the data points should fall on a single describing curve). The data points also indicate a change in characteristic slope at high coverage levels.

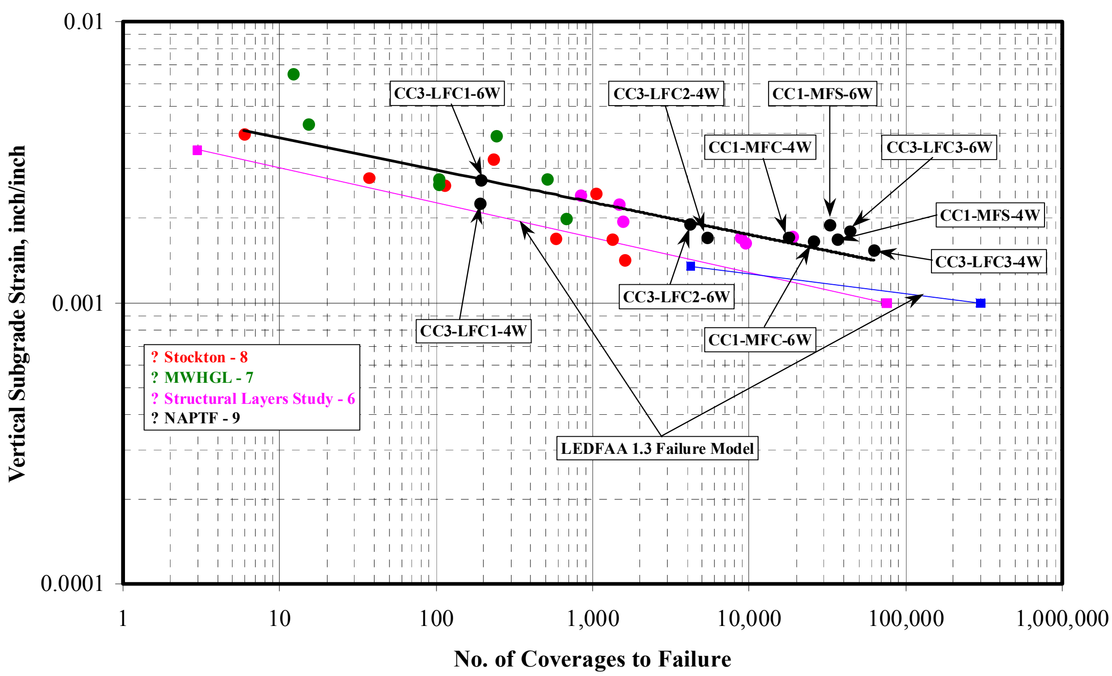


FIGURE 3-1. LEDFAA FAILURE MODEL FOR ALL AVAILABLE FULL-SCALE TEST DATA POINTS WITH REPORTED SUBGRADE SHEAR FAILURE

The dark line passing through the middle of the data points in figure 3-1 is the least squares curve fit to the data. The pair of straight lines marked LEDFAA 1.3 Failure Model is the failure model implemented in LEDFAA 1.3, which provides thickness designs with the LEDFAA structural and traffic models that are compatible with the thickness designs determined according to the conventional CBR procedures. The second straight line in the LEDFAA failure model was introduced to maintain compatibility with the CBR method and to make allowance for the leveling off of the data points indicated in figure 3-1. Further adjustment of this failure model is expected before the follow-on computer program FEDFAA is finalized, but not to the extent that thickness designs will be significantly changed.

3.1.2.2 Advisory Circular 150/5320-6D, Chapter 7.

The layered elastic design procedures included in chapter 7 of AC 150/5320-6D are computerized with the LEDFAA program. LEDFAA implements layered elastic theory-based design procedures developed under the sponsorship of the FAA for new and overlay design of flexible and rigid pavements. The major changes in the response calculation from AC 150/5320-6D to LEDFAA are as follows:

- The critical response in LEDFAA is the vertical strain at the top of the subgrade and the horizontal strain at the bottom of the HMA layer. Deflection was used in AC 150/5320-6D.

- The pavement model used in computing critical response was changed from a single layer elastic semispace to a multiple-layer elastic system.

- The response calculation changed from constant tire contact area to constant tire pressure.

Design charts (nomographs) were replaced with a computer program. The design aircraft concept was replaced by design for fatigue failure expressed in terms of a cumulative damage factor (CDF) using Miner's rule. Also, the major material property of the pavement layers is now uniformly expressed as an elastic modulus instead of the previous CBR for flexible pavements. The design process considers two modes of failure for flexible pavement:

- Vertical strain in the subgrade to limit subgrade rutting.
- Horizontal strain at the bottom of the HMA layer to limit fatigue failure.

The major assumptions are as follows:

- Materials are homogenous, linear elastic, and isotropic.

- Since the solution is performed in radial space, only circular loads can be analyzed. Other load shapes can only be approximated as sums of superimposed circular loads.

- Loads must be vertical.

- All the layers are of uniform thickness and infinite in horizontal extent.

3.2 RIGID PAVEMENTS.

The first step in predicting pavement life is to calculate the critical stresses of the pavement based on a pavement structural model with well-defined boundary conditions, material properties, and traffic loads. Then the critical stress is adjusted to obtain the working stress that will be used in the failure model for design. Therefore, the calculated stresses have significant effects on the predicted pavement life. The major effects on the pavement life through the calculated stresses are analyzed in this section.

3.2.1 Slab Models.

For more than a half century, an infinitely large, elastic, and thin plate was used to model the PCC pavement slabs for calculating the critical stress [15 and 16]. The same model was used in pavement thickness design by most design procedures [17, 1, 18, and 19]. The application of this model assumes that the behavior of a PCC slab under a load perpendicular to the slab surface is similar to a thin plate under the same load. Therefore, the model simplifies a three-dimensional (3D) problem into a two-dimensional (2D) one, and it makes all tools that were developed for thin plate analysis applicable for calculating the pavement slab critical stress. These tools include close-form equations [15 and 16], an integration solution by computer program H51 [20], and 2D finite element solutions [21-24].

The use of the thin plate model has been supported by long-time theoretical analysis: 3D solids can be modeled by a 2D plate if the length of one dimension is smaller than one-third of the other two dimensions. For an 18-inch-thick, 15- by 15-foot slab, the ratio is 1/10; much less than 1/3. Two models other than the thin plate are often used. One models the PCC slab by an infinitely large elastic layer with the same thickness as the slab [25 and 26]. The other models the slab with 3D finite elements [27 and 28].

Although there are pros and cons associated with each model and the simplified models were effectively used in the past, the advent of more complex landing gears as well as the need to model all layers of the support system has led to a preference for 3D finite element methods (FEM) slab models.

3.2.2 Foundation Models.

The modeling of the pavement foundation led to more uncertainties. A real PCC pavement consists of slabs on several layers of different materials. The multiple-layer foundation supports the slabs on top through its ability to resist compression, shear, and bending forces (and moments). The foundation is a very complicated system. First, it is not a linear elastic system. The modulus of the materials is functions of stress states. Second, the interface behaviors between two adjacent layers, especially the two top layers, are very complicated. The friction forces vary along with the load magnitude, direction, gear configuration, and repetition number of the loads. Before 3D finite element techniques were used in pavement engineering, it was almost impossible to model the foundation behaviors in detail.

A very simple model—dense liquid foundation—has been used for more than a half century [15]. Only one parameter, foundation modulus *k*, is used to describe the foundation's resistance to compression. The major differences between the dense liquid foundation model and the multiple layer foundation are as follows:

- The dense liquid foundation model does not consider shear deformation.

- A pavement with multiple layers needs many parameters, thickness, modulus, and Poisson's ratio of each layer to define its characteristics. The dense liquid foundation is defined by only one parameter, the foundation modulus *k*. An empirical procedure [17 and 29] was used to calculate the effective *k* at the top of a layered support system as an expedient. However, direct characterization of all supporting layers, such as used in a 3D FEM model, is preferable.

Currently, all foundation models are still used in various design methodologies but the variation of the parameters used to define the pavement foundation can lead to variations in the calculated critical stresses, which further lead to variations in the predicted pavement life of the design.

<u>3.2.3 Effects of Concrete Elastic Modulus</u>.

As previously mentioned, the working stress (based on the calculated critical stress under a load) is defined as the damage indicator of the pavement. Therefore, the pavement life is very sensitive to the calculated critical stress, and it is important to verify the correctness and accuracy of the calculated stress of different models. Unfortunately, it is difficult to do so by testing, and the problem remains unsolved. Determination of the concrete elastic modulus E_c for design and analysis is still a pending problem. The critical stress in the slabs, and hence the pavement life determination, is influenced by the uncertainty in the concrete modulus.

Westergaard [16] proposed a closed-form solution (see equation 12 in reference 16) for calculating the critical edge stress of a slab when a circular load is applied tangential to the free edge of an infinitely large slab.

$$\sigma_E = \frac{3(1+\mu)P}{(3+\mu)\pi h^2}\{Ln(\frac{E_c \times h^3}{100ka^4}) + 1.84 - \frac{4}{3}\mu + \frac{1}{2}(1-\mu) + 1.18 \times (1+2\mu)\frac{a}{l}\} \tag{3-8}$$

where

P = Load magnitude in lbs
μ = Poisson's ratio of concrete
h = Thickness of the slab, in.
E_c = Elastic Modulus of the concrete, psi
k = Foundation modulus in PCI
a = Radius of the tire foot print, in.
l = Relative radius of stiffness of the pavement = $(E_c \times h^3/[12(1-\mu^2)k])^{0.25}$

For comparison, a 30- by 30-ft slab was used to calculate the critical edge stress from the Westergaard solution with a 2D finite element model [30]. Both cases used the same tire pressure, load magnitude, and load center location. Six- by six-inch elements were used in the 2D model. All other input data are listed in table 3-2. The critical stresses calculated by equation 3-8 and the 2D program using two values of concrete modulus are presented in table 3-3.

TABLE 3-2. INPUT DATA

Thickness	Slab Size	Poisson's Ratio
17 inches	30 by 30 ft	0.15
Load P	Tire pressure	Foundation modulus *k*
50,000 lbs	180 psi	200 psi

TABLE 3-3. CALCULATED CRITICAL STRESS

	E_c	Equation 8 of reference 16	Jslab2004
	4,000,000	432.3	429
	6,000,000	455.2	452
Difference in Percent	50	5.3	5.4

The results presented in table 3-3 suggest that the calculated critical stress is not sensitive to the elastic modulus E_c of the concrete. Two calculations using E_c = 6,000,000 and 4,000,000 psi have only about a 5 percent difference.

Figure 3-2 indicates that a significant discrepancy exists when E_c = 4,000,000 psi is used to compare the calculated and measured strains at the NAPTF. However, when E_c = 5,500,000 psi (obtained from tests per ASTM C 215) is used, the discrepancy becomes significantly less.

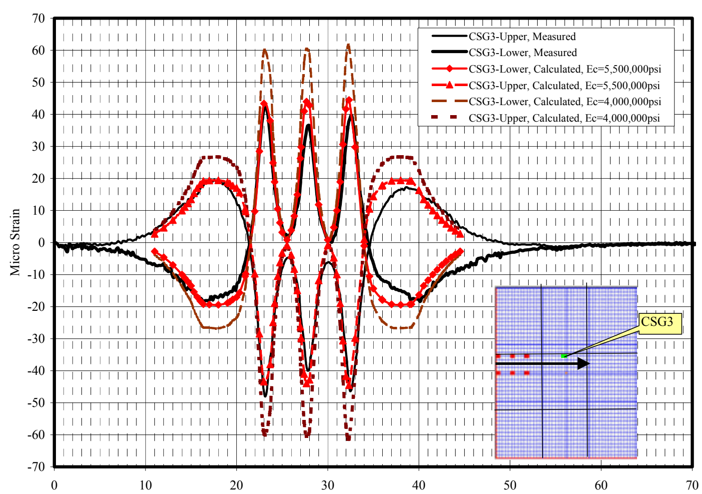


FIGURE 3-2. EFFECTS OF THE CONCRETE MODULUS ON THE COMPARISON OF MEASURED AND CALCULATED STRAINS

The above analysis indicates that though the accuracy of E_c is important in strain comparison, it is much less important in stress comparison. Therefore, the FAA has been using E_c = 4,000,000 psi in the design standards without change.

When concrete modulus of elasticity is obtained by the static test method, according to ASTM C 469, it should be noted that the standard states, "The modulus of elasticity and Poisson's ratio values are applicable with a customary working stress range (0% to 40% of the ultimate concrete strength)." The document also states, "The modulus of elasticity values obtained will usually be less than moduli derived under rapid load applications (dynamic and seismic rates, for example), and will usually be greater than values under slow load application or extended load duration, other test conditions being the same."

3.2.4 Variation of Joint Load Transfer Capabilities.

Load transfer in rigid pavements needs to be appropriately considered for PCC pavement analysis and evaluation. Therefore, the FAA supported several projects [31-34] to improve the understanding of the joint characteristics. Variation in load transfer efficiency that differs from design assumptions can affect calculated slab stresses and resultant pavement life.

Doweled joints have been verified to provide more uniform load transfer year round. WES surveyed test pavements at the Denver Airport for several years. Dong [33] reported that the load transfer capability of the joints defined by the ratio of falling weight deflectometer deflections on the two sides of the joint load transfer efficiency with respect to deflection (LTD) varied very little for the hinged joints. The average LTDs varied from 88.7% to 84.3% between summer and winter. The average LTDs varied from 81.3% in summer down to 28.2% in the winter for dummy joints because the width of the joint opening increases in the winter and the load transfer capability through the joint interlock action becomes much less. However, though the joint opening width is also increased in the winter for the doweled joints, the dowels still keep the joint load transfer capability at an acceptable level. The average LTDs measured at the Denver airport varied from 75% in the summer to 66% in the winter. Based on surveyed data from five airports (900,000 linear feet of joints total), reference 31 reports dummy and key joints received much higher joint-related distresses (spalling and cracks) than doweled and hinged joints when different types of joints are located in the same area. Though the tie bars in the hinged joints can always tightly hold the two slabs together and provide the maximum load transfer capability, the hinged joints cannot be used for all joints since they cannot allow the slab free movement when the slabs need to move under temperature or moisture variations. Therefore, doweled joints are needed when the load transfer capability must be kept at an acceptable level, and the dowel modeling becomes necessary in pavement design and analysis.

Brill [32] found that two simplified equations were used in engineering practice to calculate the joint stiffness that makes the doweled joint functionally equivalent to an interlocked joint. The joint stiffness calculated by one equation is approximately twice that calculated by another that was proposed by Huang [22]. Brill proved that the equation proposed by Huang was correct. Based on the correct equation, the equivalent joint stiffness of a doweled joint using 1.25-inch dowels with 15-inch spacing or 1.5-inch dowels with 18-inch spacing should be 47,000 psi and 55,000 psi respectively. However, a joint stiffness value of 100,000 psi has been continuously used in PCC pavement design and analysis for many years. The assumptions governing load transfer may contribute to the uncertainty of the critical stress computations. Additionally, the following factors may also increase the uncertainty of the slab critical stress.

- In FAA design specifications, 25% of the load is assumed to be transferred from one side to the other when the load is located at the joint. The same values are used year around. However, most joints in service are dummy joints [31] and provide very different load transfer capabilities between summer and winter. Reference 34 (see figures 5.20, 5.21, and 5.22 of reference 34) reports that the measured opening widths of doweled and dummy joints are significantly higher than the hinged joint when the measured average temperature of the pavement is low. The dowel bars do not reduce the variation of joint opening. Therefore, this variation increases the variation of the slab critical stress.

- The load transfer capability of dowel bars also reduces along with the number of repetitive heavy loads. Experimental research [35 and 36] indicates that gaps defined as dowel looseness are developed between the dowels and the surrounding concrete. Five percent of the total number of repetitive loads produced about 50% of the dowel looseness at the end of pavement life. That is, the dowel bars mostly work under a certain looseness and reduce the load transfer capability. This looseness has not been considered in airport pavement design specifications, leading to the increase of critical stresses in the slab.

3.2.5 Effects of Interface Model Between the Slab and the Layer Underneath.

For new PCC pavement design, the FAA specification assumes that the interface between the concrete slab and the layer underneath is frictionless or fully unbonded. The assumptions used for bonding at layer interfaces can have an effect on the computation of the critical stresses with a resultant impact on pavement life.

The data obtained from the test pavement at the Denver Airport [33] showed that the interface is partially bonded rather than fully unbonded. To further investigate the interface behavior, the FAA conducted several tests. The results obtained from a single slab test showed the uncertainty of the interface behavior. The detailed pavement structure, materials, and testing procedures are located in reference 37.

When the test was first conducted (October 20, 2003) using an 18-inch plate, the slab had reached its highest curling. The friction force in the interface shifted the neutral axis of the cross section from the middle of the slab thickness to approximately 1.5 inches from the slab top surface. The strains in one of the embedded strain gages (CSG2A), 1.5 inches from the slab top, held at almost zero when the load increased from 0 to 20,000 lb (figure 3-3).

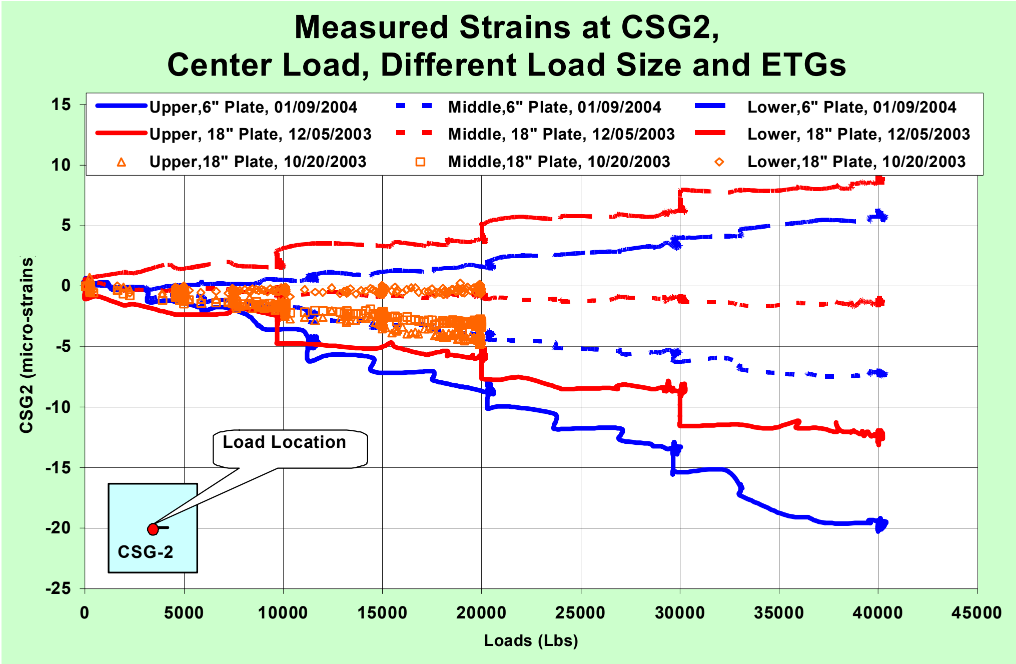


FIGURE 3-3. UNCERTAINTY OF THE INTERFACE BEHAVIOR UNDER A CIRCULAR LOAD AT THE SLAB CENTER

A second static test with an 18-inch plate was conducted (December 5, 2003) after the slab curling reduced to its lowest curling state (the four-corner curling displacements dropped from an average of 195 down to 79 mils after watering the surface). The load was applied at the slab center, again using an 18-inch plate. Figure 3-3 shows that the upper microstrain was -12.5 and the lower microstrain was about 9. The sensor at the middle of the slab thickness obtained a microstrain = -1.5, which indicates the interface friction had less effect than it had on October 20, 2003.

On January 9, 2004, the test was repeated by using a 6-inch plate. The results are also shown in figure 3-3. The maximum difference in microstrains between the measured upper and lower strains under a 6-inch plate (20 + 6 = 26) was slightly higher than under an 18-inch plate (12.5 + 9 = 21.5). However, the measured middle strain under a 6-inch plate was -7, which indicated stronger friction effects.

Figure 3-3 shows the difference of the magnitudes of peak strains measured at the upper and lower strain gages. The existence of friction reduces the magnitude of the measured lower strain. This indicates that the fully unbonded interface model used by the FAA is more or less overly conservative in estimating the maximum stress at the slab bottom.

The recent tests conducted at the NAPTF verify what had been observed in the test pavement at the Denver Airport: the interface is partially bonded and the bonded degree varies all the time, which increases the variation of the critical stress at the bottom of the PCC slabs. The variations in the critical stresses will impact pavement life.

3.3 SUMMARY.

The evolution of the FAA design standards over time has been discussed in this section. An understanding of the design theory and the fundamental difference in the failure mechanism between rigid and flexible pavements is important in evaluating how the structural lives of rigid and flexible pavements are influenced by the design procedures and governing assumptions.

For flexible pavements, the CBR-based design procedure, alpha factor, and the current layered elastic theory-based design procedures were discussed. The failure criteria for both conventional and mechanistic design methods were based on the results of full-scale tests at WES and the NAPTF. The results of the NAPTF tests were recently used to modify the flexible pavement failure criteria. It is believed that these revised criteria will result in more economical and reliable design procedures.

For rigid pavements, different structural models used over time were discussed. Different factors, such as load transfer at joints and layer interface effects on critical stresses, affect rigid pavement life. These factors and their relationship to the design method were summarized.

4. EFFECT OF FAILURE MODELS ON PAVEMENT LIFE.

As discussed in section 3, rigid and flexible pavements exhibit fundamentally different failure modes. For rigid pavements, failure is defined as a function of slab cracking; for flexible pavements, failure is defined with respect to permanent shear deformation in the subgrade, resulting in surface rutting. Each failure mode is reflected in the failure models that are imbedded in the design procedures. The models are generally described in terms of the number of load repetitions to failure, where failure is defined as an unacceptable degree of slab cracking and subgrade deformation for rigid and flexible pavements respectively.

This section reviews the failure models used in the FAA's conventional and mechanistic design of flexible and rigid pavement. The failure model algorithms described herein can be used to compute pavement life. Therefore, an understanding of the pavement models and their effect on pavement life is important.

4.1 FLEXIBLE PAVEMENTS.

As previously discussed, the results of full-scale pavement tests were used to develop failure models used in the FAA's conventional (CBR) and mechanistic (layered elastic) design methods. Details of the development and description of the flexible pavement design models are described in the following sections.

4.1.1 Failure Model in AC 150/5320-6D.

The design procedures in AC 150/5320-6D are based on the concept of ESWL. The pavement response related to failure is deflection. ESWL is the wheel load that yields the same maximum deflection as a multiple-wheel load. The contact area of ESWL is equal to the contact area of one of the wheels of the multiple-wheel assembly. In the CBR-based design procedure, the thickness factor (η) is a function of CBR/p. The earliest form of the relationship between η and CBR/p is derived from equation 3-2 in section 3 and is as follows:

$$\eta = \sqrt{\frac{p}{8.1 * CBR} - \frac{1}{\pi}} \tag{4-1}$$

(for 0.22 > CBR/p)

where

CBR	=	the strength test rating value
p	=	the contact pressure of ESWL

Based on the test results from the MWHGL study, the CBR equation was modified (equation 3-6 in section 3) and the relationship between η and CBR/p is shown below:

$$\eta = -0.0481 - 1.1562\left[Log\frac{\overline{CBR}}{p}\right] - 0.6414\left[Log\frac{\overline{CBR}}{p}\right]^2 - 0.4730\left[Log\frac{\overline{CBR}}{p}\right]^3 \tag{4-2}$$

In AC 150/5320-6D and the computer program F806, equation 4-1 is used when CBR/p is less than 0.22. The following relationship is used for 1 ≥ CBR/p ≥ 0.22:

$$\eta = 0.05 - 0.35187\left[Log\,\overline{\frac{CBR}{p}}\right] + 0.51492\left[Log\,\overline{\frac{CBR}{p}}\right]^2 \qquad (4\text{-}3)$$

For CBR/p > 1, equation 4-4 is used:

$$\eta = 0.05 - 0.005\left[Log\,\overline{\frac{CBR}{p}}\right] \qquad (4\text{-}4)$$

Figure 4-1 shows the relationship between the thickness factor η and CBR/p from different relationships.

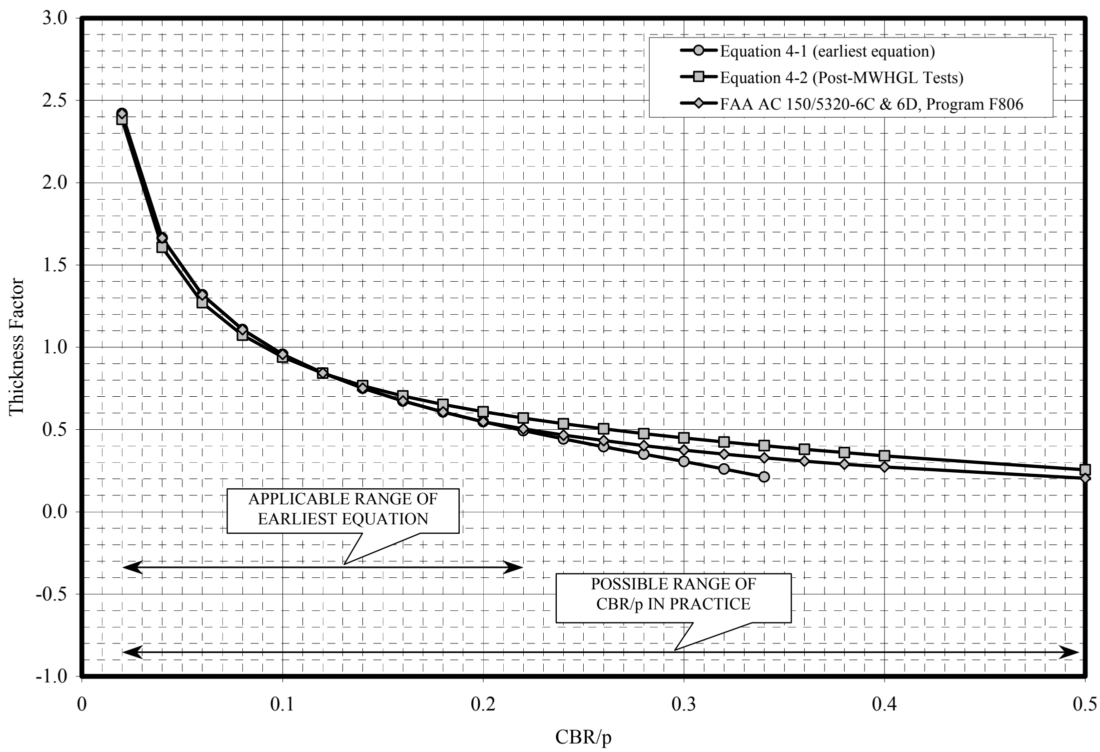


FIGURE 4-1. COMPARISON OF THICKNESS FACTOR FROM DIFFERENT RELATIONSHIPS

Values of CBR/p for design are typically in the ranges of 0.02 to 0.1 for protection of subgrades, 0.05 to 0.2 for protection of subbases, and 0.2 to 0.5 for protection of bases.

The number of allowable coverages is calculated by reorganizing the CBR equation as follows:

$$LogC = \left\{ \frac{t}{0.23 * \sqrt{A} * \left[-0.0481 - 1.1562 \left(Log \frac{\overline{CBR}}{p} \right) - 0.6414 \left(Log \frac{\overline{CBR}}{p} \right)^2 - 0.4730 \left(Log \frac{\overline{CBR}}{p} \right)^3 \right]} \right\} - 0.65 \qquad (4\text{-}5)$$

where

C	=	number of coverages
A	=	contact area
t	=	pavement thickness
CBR	=	strength test rating value
p	=	contact pressure of ESWL

Equation 4-5 provides the empirical link between predicted pavement life (coverages) and pavement response (p = ESWL/contact area of one wheel on the landing gear). ESWL is a true representation of pavement response because it is calculated as the single-wheel load that causes the same vertical deflection in the structure as the landing gear of interest. Both deflections are computed at depth "t" below the top of the pavement. In the normal nomenclature of pavement design, the CBR method for airport pavements is a true mechanistic-empirical model.

The MWHGL [5] tests indicated that the above equation does not apply for aircraft gears with multiple wheels. Based on the test results, an alpha factor or the load traffic factor was introduced in the MWHGL report. Equation 4-6 shows the relationship between alpha factor, pavement thickness, subgrade CBR, and contact pressure of ESWL.

$$\frac{t}{\sqrt{A}} = \alpha \left\{ -0.0481 - 1.1562 \left[Log \frac{\overline{CBR}}{p} \right] - 0.6414 \left[Log \frac{\overline{CBR}}{p} \right]^2 - 0.4730 \left[Log \frac{\overline{CBR}}{p} \right]^3 \right\} \qquad (4\text{-}6)$$

where

α	=	load traffic factor
A	=	contact area
t	=	pavement thickness
CBR	=	strength test rating value
p	=	contact pressure of ESWL

A standard chart of alpha factors with coverages for different numbers of wheels on the landing gear is shown in figure 4-2.

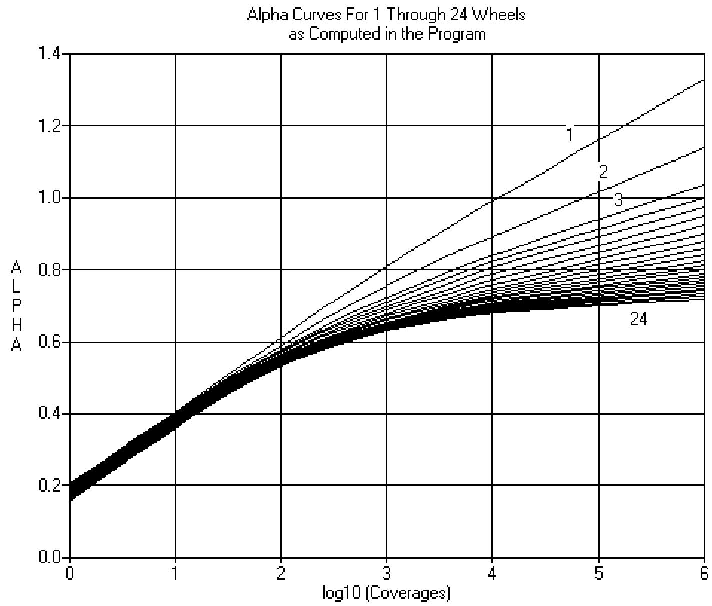


FIGURE 4-2. ALPHA FACTOR VERSUS COVERAGES
(From the COMFAA program)

With the advent of the Boeing-777 (six-wheel landing gear, 540,000 to 580,000 lbs projected to over 750,000 pounds), emphasis again shifted onto alpha factor because preliminary analyses indicated that aircraft with six-wheel landing gears would require significantly thicker flexible pavements than the existing fleet. The value of alpha factor at 10,000 coverages from inception to 1995 was 0.788. In 1995, as a result of reanalysis of the MWHGL data by the USACE, the International Civil Aviation Organization assigned an interim alpha factor value of 0.72 at 10,000 coverages for six-wheel gears. In all the analysis of data for the development of the alpha factor chart, alpha factor does not vary with CBR for a given gear configuration. Further information on these developments can be found in reference 38.

Recent analysis of combined MWHGL [5] and NAPTF [14] data showed that a least squares quadratic curve fit gives an alpha factor at 10,000 coverages for six-wheel gears of 0.718 to 0.729. The alpha factor should vary with CBR and gear geometry—the CBR design model, however, holds it constant. During the tests conducted at the NAPTF, the six-wheel gear test results did not show any unexpected levels of pavement damage compared to the four-wheel gears. Also, the FAA does not need alpha factors for landing gears with more than six wheels because of the transition to the layered elastic system.

4.1.2 Failure Model in AC 150/5320-16 (LEDFAA 1.2).

The FAA's LEDFAA design procedure based on layered elastic analysis is more mechanistic in nature. Critical pavement responses, such as the maximum tensile strain at the bottom of the HMA layer and maximum vertical strain at the top of the subgrade, are related to pavement life. JULEA (developed by the USACE) is the layered elastic computational program in LEDFAA-1.2 used for layered elastic analysis. The maximum horizontal tensile strain at the bottom of the HMA layer is computed and related to the number of coverages (life of HMA surface) considering asphalt fatigue failure. The Heukelom and Klomp model has been adopted and is as follows:

$$Log_{10}C = 2.68 - 5 * Log_{10}\varepsilon_H - 2.665 * Log_{10}E_A \tag{4-7}$$

where

C = number of coverages to failure
E_A = elastic modulus of asphalt concrete, psi
ε_H = maximum horizontal tensile strain at the bottom of the HMA layer

The concept of coverages as used in the FAA's mechanistic design procedures is discussed in more detail in section 4.2.2. The maximum vertical strain at the top of the subgrade is used to control subgrade rutting. The following equation is used to predict the number of coverages to failure and is based on the analysis of full-scale test data:

$$C = 10{,}000 * \left[\frac{0.000247 + 0.000245 * Log_{10}E_{SG}}{\varepsilon_V}\right]^{0.0658*E_{SG}^{0.559}} \tag{4-8}$$

where

E_{SG} = elastic modulus of subgrade soil, psi
ε_V = maximum vertical strain at the top of the subgrade

From equation 4-8, the number of coverages to failure depends on the subgrade elastic modulus and maximum vertical strain at the top of the subgrade. Figure 4-3 shows the subgrade failure criteria.

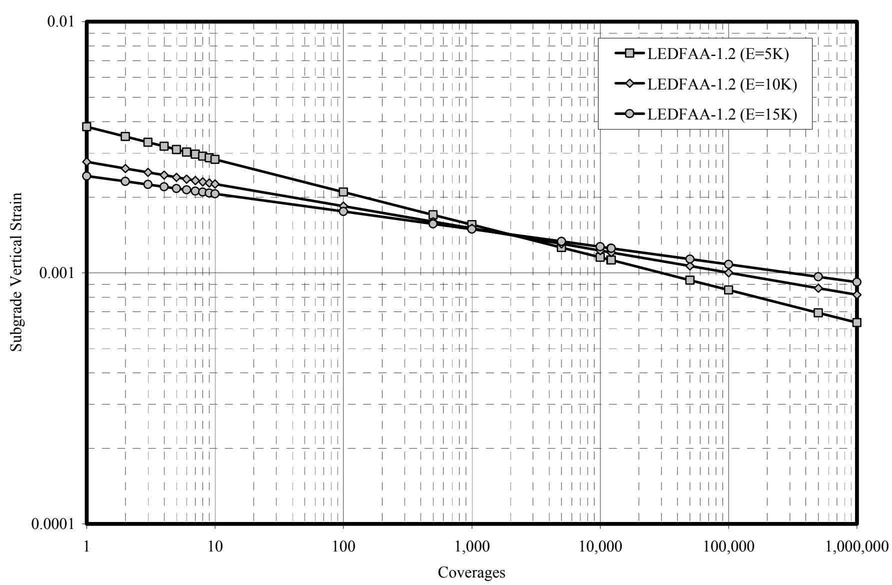


FIGURE 4-3. SUBGRADE FAILURE CRITERIA IN LEDFAA-1.2

4.1.3 Failure Model in AC 150/5320-6D Change 3 (LEDFAA 1.3).

As per AC 150/5320-6D Change 3, (issued in April 2004), LEDFAA 1.3 became an official FAA design standard and can be used in lieu of AC 150/5320-6D design charts for any aircraft mix. In the LEDFAA 1.3 computer program, LEDNEW was replaced by LEAF—a layered elastic computational program developed by the FAA. As far as the failure models are concerned, the failure model for HMA fatigue is the same as that in LEDFAA 1.2 (equation 4-7). The failure model for subgrade rutting was modified based on the availability of additional full-scale test data from the NAPTF and the reanalysis of all the previous full-scale test data. The dependency on the number of coverages to failure on subgrade modulus has been eliminated. The failure model is shown in equations (4-9a) and (4-9b).

$$C = \left[\frac{0.004}{\varepsilon_V}\right]^{8.1} \qquad \text{when } C \leq 12{,}100 \tag{4-9a}$$

$$C = \left[\frac{0.002428}{\varepsilon_V}\right]^{14.21} \qquad \text{when } C > 12{,}100 \tag{4-9b}$$

Figure 4-4 shows a comparison of failure models in LEDFAA 1.2 and LEDFAA 1.3.

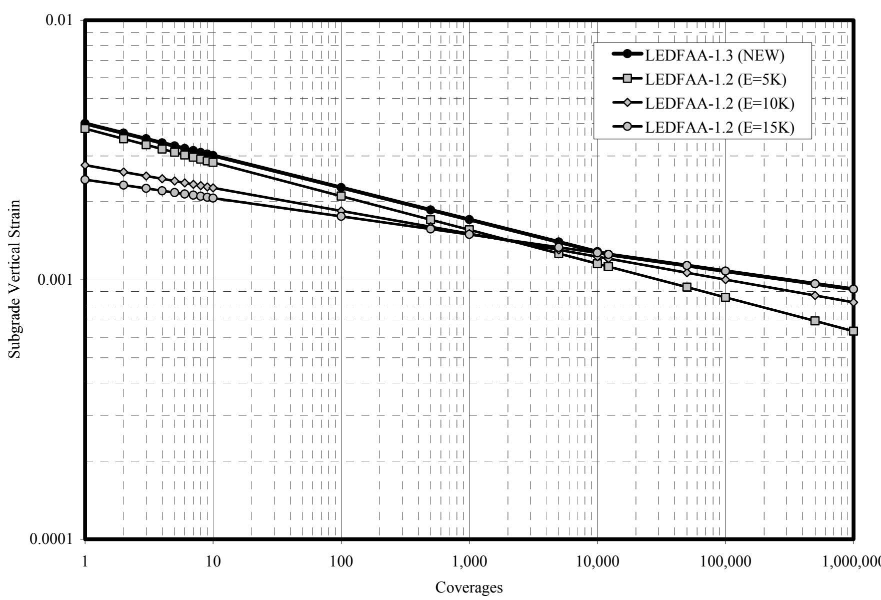


FIGURE 4-4. COMPARISON OF SUBGRADE FAILURE MODELS IN LEDFAA

4.2 RIGID PAVEMENTS.

4.2.1 Major Changes in Failure Models.

In AC 150/5320-6D [1], the following failure model equations are used:

$$COV = 5000 \times 10^{(\sqrt{\frac{R_F}{\sigma \times 1.3}} - 1) / 0.15603} \qquad (Coverage > 5000) \qquad (4\text{-}10a)$$

$$COV = 5000 \times 10^{(\sqrt{\frac{R_F}{\sigma \times 1.3}} - 1) / 0.07058} \qquad (Coverage \leq 5000) \qquad (4\text{-}10b)$$

where

σ = the working stress in the design
R = flexural strength
COV = coverage of the load

A table of the pass-to-coverage ratios used for rigid pavement design is provided in AC 150/5320-6D.

Pavement life, defined in section 2, can be expressed by the number of coverages. Therefore, in the failure model in AC150/5320-6D, the pavement life is a function of R/σ—the ratio between concrete flexural strength and the working stress used in design.

AC 150/5320-6D was recently revised to incorporate the contents of AC 150/5320-16, Airport Pavement Design for the Boeing 777 Airplane. The elastic multiple-layer model is used instead of the semi-infinite thin plate on the dense liquid foundation model. The failure model adopted was based on the following model developed by Rollings [39].

$$SCI = \frac{R_F/\sigma - 0.2967 - (0.3881 + 0.000039 \times SCI) \times Log_{10} COV}{0.002269} \qquad (4\text{-}11)$$

Where, R_F is still defined the same as in equation 4-10. σ is also the working stress in a design that equals the maximum interior stress calculated by using a multiple elastic layer model.

Equation 4-11 has been replaced by equation 4-12 in LEDFAA (AC 150/5320-16 and AC 150/5320-6D, Change 3, 2004):

$$SCI = \frac{R_F/\sigma - 0.2967 - F_S \times (0.3881 + F_{SC} \times 0.000039 \times SCI) \times Log_{10} COV}{0.002269} \qquad (4\text{-}12a)$$

$$F_{SC} = \frac{0.392 - 0.3881 \times F_S}{0.0039 \times F_S} \qquad (4\text{-}12b)$$

Where F_s and F_{sc} are two factors introduced to provide compensation for the new model. F_{sc} is defined by equation 4-12b to maintain the number of coverages for SCI = 100 using equations 4-11 and 4-12 for any given value of R/σ where σ is the working stress (rather than the critical stress directly calculated by the structural model).

Two major differences exist between equations 4-10 and 4-12:

- A different structural model is used. The elastic multiple layer system is used in calculating the critical interior stress in equation 4-12 rather than in a thin semi-infinite large plate on the dense liquid model in equation 4-10.

- The new failure model is adjusted by adjusting the working stress. The adjustments are normally implemented in three steps. The first step is to appropriately consider the contribution of the subbase layer under the PCC slab. A modification will be made if one of the following conditions is true: (1) a stabilized subbase with thickness thicker than 4 inches or with modulus other than 500,000 psi or (2) a crushed aggregate subbase with a thickness other than 8 inches. The second step is to consider the difference between edge and interior stress. The third step is for comprehensively compensating the different formats of failure models. This three-step adjustment is presented in figure 4-5.

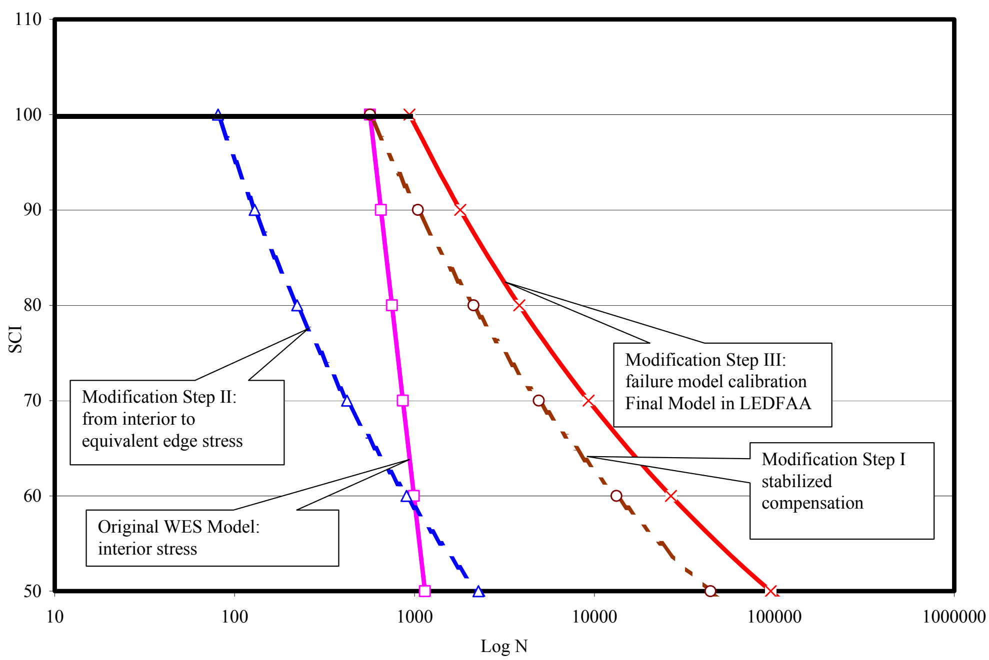


FIGURE 4-5. THREE-STEP ADJUSTMENT FROM WES TO LEDFAA FAILURE MODELS

One additional parameter, SCI, is introduced into the new model to define the pavement structural condition during its service. SCI is assumed to remain at 100 from a new pavement until the first full-depth crack is observed. After that, SCI is assumed to continuously decrease with six observed distresses. The six distresses for calculating the reduction of SCI proposed by Rollings [39], page 30, are

- Corner break
- Longitudinal, transverse, and diagonal cracking
- Shattered slabs
- Shrinkage cracks (cracking partial width of the slab)
- Spalling along the joints
- Spalling at the corner

Many distresses due to nonload factors, such as D cracking, joint seal damage, patching, scaling, crazing, and map crazing, are not counted in calculating the SCI reduction. When equations 4-11 and 4-12 are used for pavement thickness design, SCI = 80 is defined as the end of pavement life and corresponds to 50% of the slabs having been cracked. However, the comprehensive PCI would be more or less lower than 80 since the PCI of 100 would subtract the reduction values contributed from the listed six distresses, and from other distresses due to nonload factors (see ASTM D 5340).

It should be noted that when equation 4-10 was used (1978), the quantitative definition to define the end of pavement life was not clear. The new FAA design specifications (after 1995) introduced a quantitative definition of the end of pavement life with SCI = 80. The advantages of introducing SCI in the design procedure include

- The use of SCI differentiates the two types of distresses in design: one is structural-related due to loads, and the other is non-structural-related due to nonload effects. The main objective of the design specification is to determine the pavement structural components under the traffic loads.

- FAA design specifications provide design procedures for new PCC pavement and for overlays on an existing PCC pavement. Conceptually, the condition of the existing pavement prior to being overlaid must be defined before an overlay thickness is determined. AC 150/5320-6D uses an empirical method to define the existing pavement condition factors C_r and C_b. Experienced pavement engineers who observe the pavement surface distresses determine the values of these factors. Although the method is rough, more than 20 years of experience was accumulated. However, PCI rather than SCI was used to collect survey data for many years. Therefore, when the SCI began to be used in the LEDFAA program, formulae to determine SCI was recommended.

- The use of SCI possesses the potential to improve the design procedure in the future. This is particularly true when more SCI data becomes available from pavement surveys. It has been recognized for a long time that the same value of PCI from two deteriorated PCC pavements, one receiving much more structural-related distresses than the other, needs significantly different procedures to repair. It has also been understood that there is an optimal period for repairing a deteriorated pavement. The use of SCI in the design model will be helpful to better evaluate the pavement condition and get a cost-effective rehabilitation.

4.2.2 Calculation of Pass-to-Coverage Ratio.

The design procedure in AC 150/5320-16 [13] was adopted in the revised version of AC 150/5320-6D (April 30, 2004) for a traffic mix with and without B-777 aircraft. A pavement cross section is divided into 81 sections, each 10 inches wide. This procedure is also used for computing flexible pavement CDF. CDF is calculated for the center of each section using the following equation:

$$CDF = \frac{Applied\ Coverages}{Coverage\ to\ failure} \tag{4-13}$$

The value of coverage to failure is obtained by using the failure model equation 4-12, and the value of applied coverages is the number of applied load repetitions divided by the pass-to-coverage ratio.

Pass-to-coverage ratio is found by exactly the same procedure as was used to calculate the values in the table published in AC 150/5320-6D. The definition of coverage is: one coverage occurs at a point on the pavement when that point is covered by any tire print of the aircraft gear. Based on the definition, the following equation is used to calculate the pass-to-coverage ratio:

$$P_i / C_{j,i} = 1 / \sum_{k=1}^{m} \text{Prob}\left(X_{k,i} - \frac{W_i}{2} < X_j < X_{k,i} + \frac{W_i}{2}\right) \tag{4-14}$$

where

- P_i = Pass number of the ith type of aircraft
- $C_{j,i}$ = Coverage of the ith type aircraft at the jth strip
- W_i = Tire width of the ith type aircraft
- X_j = Distance between the centerline of the jth strip and the centerline of the pavement
- $X_{k,i}$ = Distance between the centerline of the kth tire of the ith type aircraft
- $Prob$ = Calculation symbol for probability of
- m = Number of tires of the ith type aircraft

Equation 4-14 indicates that each strip of the pavement (perpendicular to the transverse line) receives different amounts of the maximum stress, or different amounts of coverages, $C_{j,i}$. Therefore, pass-to-coverage varies along the transverse surface line of the pavement and leads to different CDF values at each strip. The failure model in AC 150/5320-6D has only one pass-to-coverage number for each type of aircraft.

These and other assumptions must be used to calculate the pass-to-coverage ratio. These also introduce uncertainties in pavement life prediction.

4.2.3 Uncertainty of the Concrete Flexural Strength.

Differences in the flexural strength of concrete (R_F) used for design and the flexural strength of the in-place concrete can be a significant factor in differences between the predicted life of a pavement and the observed life of the pavement. Possible sources of such differences are differences between design requirements and in-place acceptance requirements, allowances for statistical variations in the strength of in-place acceptance samples, environmental conditions at placement and during curing, and variations in construction practices.

AC 150/5320-6D requires that "Concrete flexural strength should be determined by ASTM C 78 Test Method. The design flexural strength of the concrete should be based on the age and strength the concrete will be required to have when it is scheduled to be opened to traffic." In contrast, AC 150/5370-10A requires that P-501 specification concrete be designed to achieve a 28-day flexural strength for acceptance. The engineer has the option to designate the time period if the specified strength is required earlier than 28 days. The specifications, therefore, require that the pavement thickness design engineer make allowances for differences between the strength of the concrete at the time of acceptance testing and the length of time that passes after placement before the pavement is put into service. It is not known if this typically leads to over- or underconservatism in the selection of the design strength of the concrete.

AC 150/5370-10A also states "Higher flexural strength can be specified when local materials make this economically feasible. However, it must be recognized that due to variations in materials, operations, and testing, the average strength of concrete furnished by a supplier must be higher than the specified strength to insure a good statistical chance of meeting the acceptance criteria throughout the duration of the job." This typically leads to a conservative selection of the design strength of the concrete.

With respect to differences between the strengths of laboratory samples and field samples or the in-place concrete, several projects conducted by the FAA at the NAPTF verify that the values of compressive strength (R_C) and R_F measured in the laboratory under the standard conditions required by the appropriate ASTM test methods are significantly different than those measured with field-cured specimens. The variations of the compressive and flexural strengths of the field-cured specimens are much larger than the variations of the compressive and flexural strengths of the specimens cured in the laboratory.

The findings from the FAA projects indicate that the strength of the in-place concrete in the field could be significantly different than the flexural strength used for thickness design. All the equations in section 4.2.1 show that the number of coverages to failure is very sensitive to the ratio of R_F and σ. Therefore, variations in pavement life must be influenced by the variation of the concrete strength in the field as well as by differences between the thickness design strength and the concrete mix design strength.

4.2.4 Control of Top-Down Cracking.

When the critical stress in a pavement is calculated for design, only the maximum stress at the bottom of the slab is considered in current FAA design specifications. However, unexpected top-down cracking has a significant effect on the pavement life. Corner cracking is initialized from the top of the slab due to a gear or tire load located at a corner of the slab, and if there were no restrictions on the joint spacing, top-down cracks would become the earliest major distresses observed under heavy aircraft traffic. Therefore, both AC 150/5320-6C [17] and -6D [1] set requirements for joint spacing to minimize the occurrence of top-down cracking.

A rule of thumb for joint spacing given by the Portland Cement Association was adopted in AC 150/5320-6C [17]. As a rough guide, the joint spacing (in feet) should not exceed twice the slab thickness (in inches). The ratio of slab length to slab width should not exceed 1.25 in un-reinforced pavements. All PCC slabs were divided into three thickness groups: less than 9 inches, between 9 to 12 inches, and thicker than 12 inches. The maximum joint spacing for transverse and longitudinal joints were recommended for each group (Table 3-7, AC 150/5320-6C, [17]).

The risk of top-down cracking was more noticeable by analyzing surveyed data [37]. It had also been noticed that smaller slabs reduced the risk of top-down cracking. However, large slabs (25 by 25 ft) were favored by construction techniques (requiring lower maintenance costs for repairing the joint-related distress). Therefore, a balance between the risk of top-down cracking and favorable slab size from a construction and maintenance viewpoint is necessary. It was also recognized that PCC pavements supported by a stabilized subbase are subject to higher warping and curling stresses than those supported on unstabilized foundations. Therefore, smaller slabs were recommended for pavements supported on a stabilized subbase.

Two minor modifications to AC 150/5320-6C were effected in AC 150/5320-6D. One more group was added to Table 3-7, requiring the use of smaller slabs (12.5 by 12.5 ft) for very thin slabs (6 inches). The requirements of Table 3-7 in AC 150/5320-6C were only recommended for PCC pavements without a stabilized subbase. For pavements with a stabilized subbase, it is required that the ratio of the joint spacing to the radius of relative stiffness be between 4 and 6. This was modified in Change 2 of AC 150/5320-6D to require that the ratio of the joint spacing to the radius of relative stiffness be 5.0 or less. The radius of relative stiffness (*l*) is calculated by the following equation.

$$l = \sqrt[4]{\frac{E \times h^3}{12 \times (1 - \mu^2) \times k}} \tag{4-15}$$

where

E = modulus of elasticity of the concrete, usually 4 million psi
h = slab thickness, inch
μ = Poisson's ratio of concrete, usually 0.15
k = modulus of subgrade reaction, psi

These requirements are the only provisions in the rigid pavement design procedure by which the problem of top-down cracking (such as corner cracking) caused by combined loading and curling is addressed, since the thickness design is based only on load-induced stresses calculated at the bottom of an assumed flat slab.

4.3 SUMMARY.

The failure model is an integral part of any pavement design procedure. The pavement responses are related to the allowable number of load repetitions using statistical and empirical transfer functions. Different failure models and their development over the years have been discussed. Generally, the predicted and actual pavement performance have not always compared favorably in the past. The full-scale test results and improvement in structural models (used for predicting critical pavement responses) are used for improving the failure models. The shift factors are used in various transfer functions to adjust the structural model calculated response to more realistically reflect field-observed pavement distress and performance. The pavement responses used in the failure models are for a given set of conditions (climate, pavement structure, material properties, aircraft loading, etc.). Pavement responses will vary as any of the above-mentioned conditions vary. The mechanistic design procedures must account for the effect of varying inputs to the structural model.

5. SENSITIVITY OF INPUTS TO PAVEMENT LIFE.

Pavement design is a stochastic process. The thickness design and resultant pavement life can be significantly affected by the choice of the design inputs. Since it is impossible to precisely define the absolute value of the primary design variables, there is always inherent uncertainty concerning the robustness of the design. This section discusses the sensitivity of the design inputs to computed pavement thickness and resultant pavement life for existing design models for airport pavements contained in AC 150/5320-6D and LEDFAA.

5.1 CONCEPTS AND FORMULAS.

A sensitivity study is a powerful tool to clarify, verify, quantitatively understand, and compare existing airport pavement design specifications. This tool was used in developing the computer program LEDFAA [29]. In 1995, a sensitivity study [41] was conducted to determine the input data for a model developed by WES, Corps of Engineers [42, 43, and 39] and made the design thickness comparable to those obtained by the FAA design procedure AC 150/5320-6C [17]. Two other FAA reports [44 and 45] have been drafted but not yet published. In these two reports, the concepts, derivation, and numerical results of the sensitivity for selected parameters in the FAA airport pavement design models up to 1994 were presented.

Each pavement design needs many input parameters. When a design model is determined, different input parameters have different influences on the pavement life. Some parameters are insensitive to the effects on pavement life; however, for some parameters; a small change will lead to a significant change in pavement life. The effects of many parameters on pavement life were qualitatively discussed in sections 3 and 4.

The sensitivity of pavement life, L, to any variable x is defined as

$$S_{x,L} = \frac{\partial L}{\partial x}\frac{x}{L} \qquad (5\text{-}1)$$

This equation is used for both HMA and PCC pavement sensitivity analysis. In cases where a pavement is subjected to only one type of aircraft, the pavement life is directly related to the number of passes of that aircraft. Therefore, *L* in equation 5-1 is related to the number of passes of aircraft N. However, for a pavement under a mixed traffic, the explicit relationship between the pavement life and the number of passes of an aircraft does not exist. Numerical analysis becomes necessary to calculate the sensitivity of *x*. In this case, the following approximate equation should be used:

$$S_{x,L} = \frac{\partial L}{\partial x}\frac{x}{L} \cong \frac{L[x+\Delta x]-L[x-\Delta x]}{2\times\Delta x}\times\frac{x}{L[x]} \qquad (5\text{-}2)$$

where

x = value of the variable

Δx = A very small quantity that is significantly smaller than the value of x

$L(x+\Delta x)$, $L(x,)$ and $L(x-\Delta x)$ = Value of L at $x = x+\Delta x$, x and $x-\Delta x$, respectively

After values of x and Δx are selected for a parameter, $L(x+\Delta x)$, $L(x)$, and $L(x-\Delta x)$ can be calculated. Then the sensitivity can be calculated by substituting the results into equation 5-2.

A positive value of $S_{x,L}$ indicates that the pavement life increases when x increases, and a negative value indicates that the pavement life decreases when x increases. A high magnitude of $S_{x,L}$ indicates that x is sensitive to L, no matter if it is positive or negative. The above definition of the sensitivity is a nondimensional quantity; therefore, it is independent of the unit of any variable and makes sensitivity of different variables comparable to each other.

From equation 5-1, the sensitivity is not only a function of the derivative of pavement life to the variable it is also a function of the values of the pavement life and the value of the variable. That is, the sensitivity varies with different pavement structures under different aircraft even if the pavement is designed using the same failure model.

5.2 FLEXIBLE PAVEMENTS.

Flexible pavement design is based on the vertical strain at the top of the subgrade. The number of coverages to failure is related to the subgrade strain, as follows:

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

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

where

C = the number of coverages to failure
ε_v = the vertical strain at the top of the subgrade

Subgrade vertical strain is computed using elastic layer theory and is affected by the stiffness (modulus) and thickness of layers above the subgrade. Equation 5-2 is used to compute the sensitivity of input parameters like P-401 HMA modulus, P-401 HMA thickness, P-209 base thickness, and the gross aircraft weight. The sensitivity of these parameters was studied for three pavement structures designed using LEDFAA for three types of subgrades—low strength (CBR-3), medium strength (CBR-8), and high strength (CBR-15). The aircraft mix (anticipated at John F. Kennedy International Airport (JFK)) used for the pavement design is shown in table 5-1.

TABLE 5-1. AIRCRAFT MIX USED FOR PAVEMENT DESIGN

No.	Name	Gross Weight (lb)	Annual Departures	Percent Annual Growth
1	A300-600	375,900	3,838	0
2	A320	162,000	15,101	0
3	A330	507,000	1,015	0
4	B-757	270,000	7,544	0
5	B-737-800	174,200	1,561	0
6	B-747-200	833,000	2,207	0
7	B-747-400	873,000	8,519	0
8	B-767-200	335,000	6,178	0
9	B-767-300ER	409,000	9,635	0
10	B-777-200	632,500	3,111	0
11	Concorde	410,000	406	0
12	Fokker F100	100,000	12,117	0
13	DC-9-32	121,000	569	0
14	DC-9-51	121,000	488	0
15	A340-500/600	750,000	2,441	0
16	A340-500/600 Belly	750,000	2,441	0
17	A380-800	1,340,000	5,475	0
18	B-747-SP	696,000	3	0
19	DC-8	358,000	504	0
20	MD-11	621,000	3,315	0
21	MD-11 Belly	621,000	3,315	0

Figure 5-1 shows the pavement designs for the three subgrade types. Table 5-2 and figure 5-2 summarize the results from the sensitivity analyses.

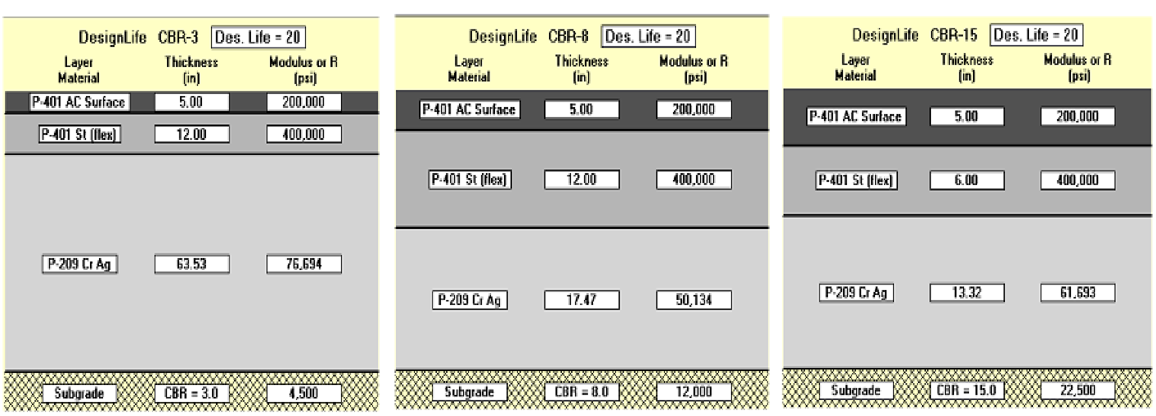


FIGURE 5-1. PAVEMENT SECTIONS FOR SENSITIVITY ANALYSIS

TABLE 5-2. RESULTS FROM SENSITIVITY ANALYSES

CBR	x	Δx	x-Δx	x+Δx	$L(x)$	$L(x+\Delta x)$	$L(x-\Delta x)$	$S_{x,L}$
Sensitivity to HMA Modulus (ksi)								
3	200	20	180	220	20	20.9	19.1	0.4500
8	200	20	180	220	20	22.3	17.8	1.1250
15	200	20	180	220	20	24.1	16.3	1.9500
Sensitivity to HMA Thickness (in.)								
3	5	0.5	4.5	5.5	20	22.0	18.2	0.9500
8	5	0.5	4.5	5.5	20	25.2	15.9	2.3250
15	5	0.5	4.5	5.5	20	31.7	12.4	4.8250
Sensitivity to P-209 Thickness (in.)								
3	63	1.26	61.74	64.26	17.9	23.2	13.9	12.9888
8	17	0.34	16.66	17.34	17.3	19.2	15.6	5.2023
15	13	0.26	12.74	13.26	16.1	19.2	13.5	8.8509
Sensitivity to B-737 Gross Weight (lb)								
3	173,000	17,300	1,55,700	190,300	20	7.5	88	-20.1250
8	173,000	17,300	1,55,700	190,300	20	8.1	83.5	-18.8500
15	173,000	17,300	1,55,700	190,300	20	9.0	74.9	-16.4750
Sensitivity to A380 Gross Weight (lb)								
3	1,300,000	130,000	1,170,000	1,430,000	30	8.4	124.4	-19.3333
8	1,300,000	130,000	1,170,000	1,430,000	30.2	8.3	127.9	-19.8013
15	1,300,000	130,000	1,170,000	1,430,000	28.9	9.1	105.7	-16.7128

5.2.1 Sensitivity of Pavement Life to HMA Modulus.

Of all the parameters studied in sensitivity analysis, the HMA modulus had the lowest sensitivity, as shown in figure 5-2. The sensitivity of pavement life to HMA modulus increases with the increase in subgrade strength (CBR).

5.2.2 Sensitivity of Pavement Life to HMA Thickness.

Figure 5-2 shows that the sensitivity of pavement life to HMA thickness is higher than the sensitivity to HMA modulus, but comparatively lower when compared to other parameters studied. For the pavements on high-strength subgrade, the sensitivity of pavement life to HMA thickness was higher when compared to the low-strength subgrade.

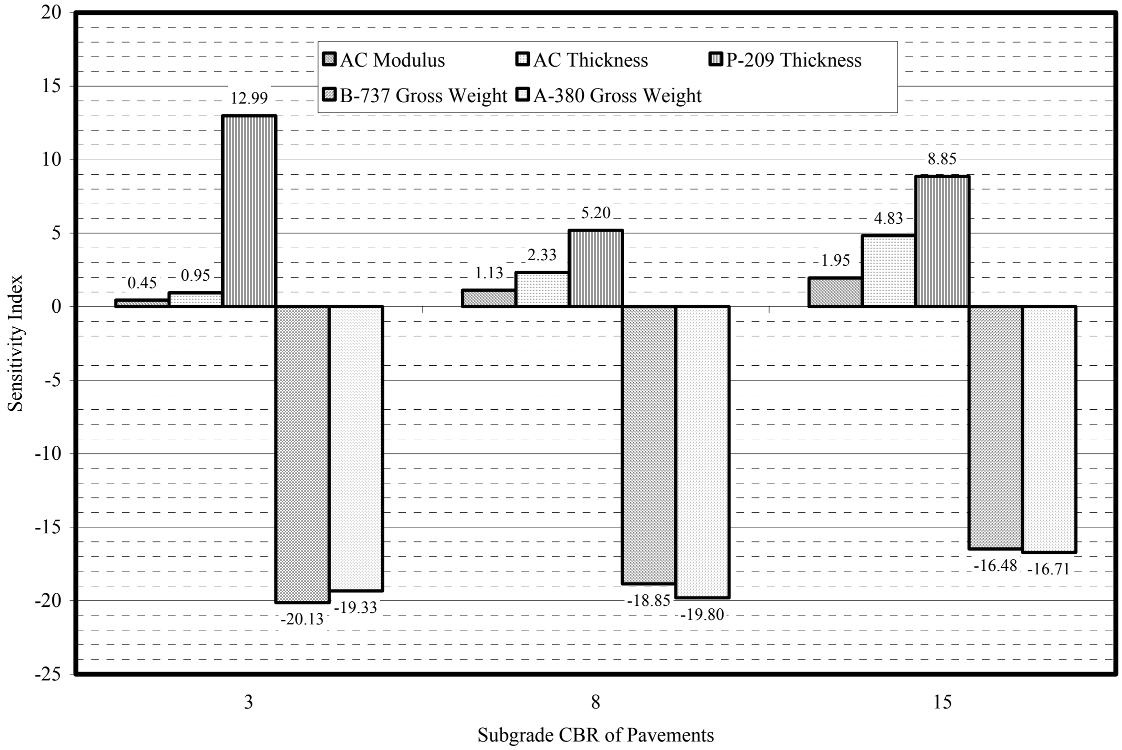


FIGURE 5-2. SUMMARY OF RESULTS FROM SENSITIVITY ANALYSES

5.2.3 Sensitivity of Pavement Life to P-209 Base Thickness.

Pavement life is more sensitive to P-209 base thickness when compared to the HMA modulus and HMA thickness. The sensitivity decreases with the increase in subgrade strength. Figure 5-2 shows that the sensitivity is higher for high-strength subgrade compared to medium-strength subgrade. The P-401 stabilized base thickness for the pavements on low- and medium-strength subgrade is 12 inches (thickness is based on minimum thickness requirements, as per AC 150/5320-6D). However, for the pavement on high-strength subgrade, the P-401 stabilized base thickness is 6 inches because of minimum thickness requirements for P-209 subbase. This is the reason for greater P-209 thickness sensitivity on high-strength pavement compared to the low-strength pavement.

5.2.4 Sensitivity of Pavement Life to Aircraft Gross Weight.

Two aircraft, one heavy A380 (gross weight 1,300,000 lb) and one light B-737 (gross weight 173,000 lb) were considered for the sensitivity analysis. Three pavements (one each on low-, medium-, and high-strength subgrade) were designed using LEDFAA 1.3 for each of the two aircraft, and the sensitivity of gross weight on pavement life was studied. As shown in figure 5-2, aircraft gross weight has the highest sensitivity on pavement life. Also, since the sensitivity is negative, it means that pavement life reduces with increase in aircraft gross weight.

5.3 RIGID PAVEMENTS.

Two types of sensitivities are of most interest in the sensitivity analysis for PCC pavement design. The first type uses the failure model in which the explicit relation between the pavement life indicator (coverage) and the pavement failure indicator (design factor R/σ) exists. Therefore, the closed-form solution of the sensitivity can be obtained. Since the failure model was developed based on full-scale tests plus engineer's experience, it is generally an empirical equation. Therefore, the sensitivity has to be affected by the format and values of parameters used in the model.

The second type of sensitivity was analyzed through the critical stress calculated using a specific response model (i.e., a mechanistic-based procedure). The sensitivity results will be affected by both failure and response models. Since the critical response of a pavement is calculated with a computer program, the sensitivity is also calculated with the design program, and only numerical results are available.

Though equations 4-12a and 4-12b are failure models, the parameters F_S and F_{SC} are functions of the material properties and thickness of the layer under the slab. They are also functions of the subgrade modulus. The sensitivity of design factor R/σ cannot be directly and simply derived by taking a derivative for equation 4-12. Therefore, the closed-form solutions of the sensitivity of R/σ based on equation 4-10 (model in AC 150/5320-6D) and equation 3-2 (the FAA mechanistic failure model) and the numerical results for equation 4-11 are presented separately.

5.3.1 Sensitivity of Design Factor R/σ.

Since the closed-form equation exists between the design factor and the number of coverages, the closed-form solution of sensitivities of the design factors R/σ on the coverage may be derived by using equation 5-1 for the design procedure in AC 150/5320-6D.

$$S_{R/\sigma,\,COV} = \frac{Ln\,(10)}{\sqrt{1.3}\times 2\times 0.15063}\times\sqrt{\frac{R}{\sigma}} \quad for \quad COV > 5000 \qquad (5\text{-}3a)$$

$$S_{R/\sigma,\,COV} = \frac{Ln\,(10)}{\sqrt{1.3}\times 2\times 0.07058}\times\sqrt{\frac{R}{\sigma}} \quad for \quad COV \le 5000 \qquad (5\text{-}3b)$$

Similarly, the closed-form solution of sensitivity based on the design procedure is

$$S_{R/\sigma,\,COV} = \frac{Ln\,(10)}{0.3881 + 0.000039\times SCI}\times\frac{R}{\sigma} \qquad (5\text{-}4)$$

Figure 5-3 illustrates the relationship between the sensitivity and the design factor where SCI = 80 has been used to indicate the end of pavement structural life.

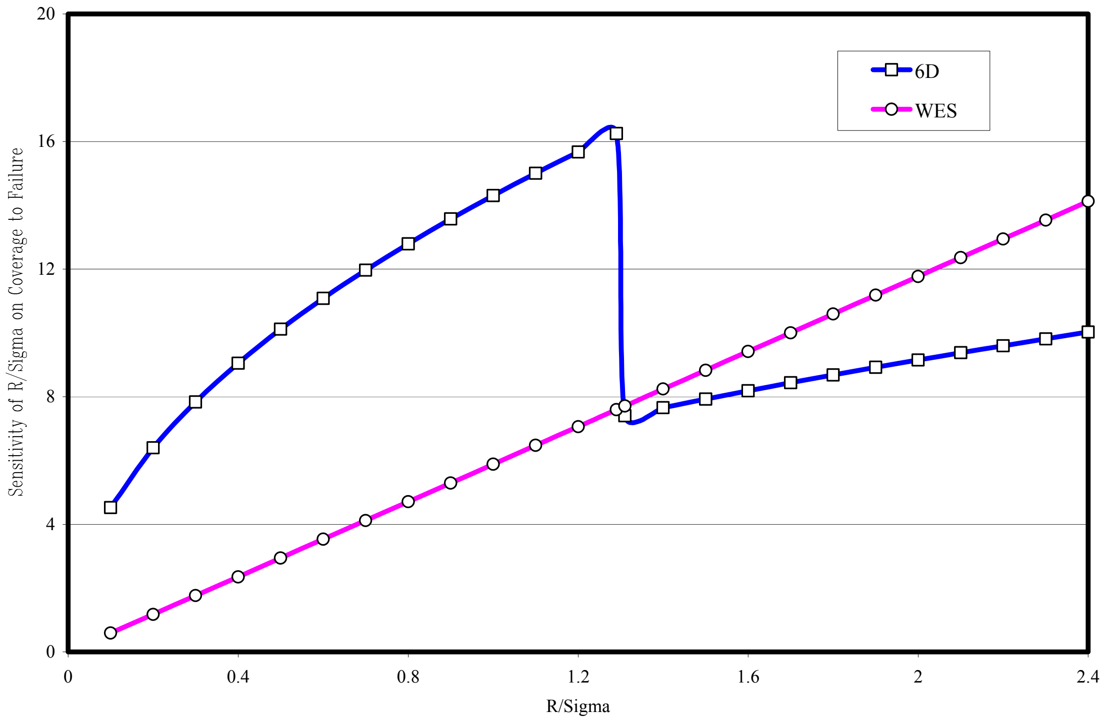


FIGURE 5-3. THE SENSITIVITIES OF DESIGN FACTOR R/σ VS R/σ

The two sensitivity curves of R/σ (calculated from equations 5-3a, 5-3b, and 5-4) are presented in figure 5-3; however, they are not completely comparable. The flexural strength *R* used in the two models follows the same standard test method ASTM C 78, the critical stress σ calculated by using two different models: plate on dense liquid foundation in AC 150/5320-6D and the elastic multiple-layer system for the WES models. The more important difference is that calculated critical stress is at different locations: free edge for AC 150/5320-6D and interior for the WES models.

The two failure models are presented in figure 5-4. When the failure model of AC 150/5320-6D was developed, only four full test points were available for coverages > 5000. Unfortunately, for two of them the test pavement never failed. Therefore, 5000 coverages were selected as a point where two failure curves were recommended. One is for coverages lower than 5000, the other is for coverage higher than 5000. The second curve was probably selected because insufficient full-scale data were available at the time. Though two curves are used, the failure model still is a continuous function at the point coverage = 5000. However, figure 5-3 shows that the sensitivity of the design factor R/σ has a significant jump at coverage = 5000.

It can be seen in figures 5-3 and 5-4 that the failure model and sensitivity curve in the design procedure, based on elastic multiple-layer theory, are continuous function (WES model [43 and 39]).

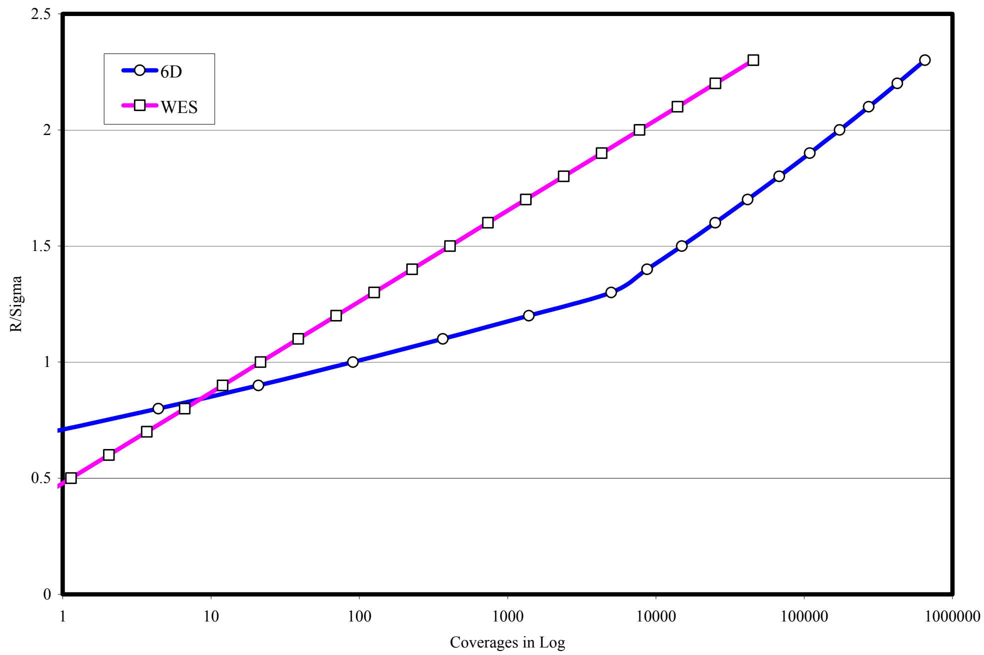


FIGURE 5-4. FAILURE MODELS IN AC 150/5320-6D AND WES

<u>5.3.2 Sensitivity to Slab Thickness, Aircraft Gross Weight, Subgrade Strength, and Stabilized Base</u>.

The sensitivities of the following six parameters are analyzed following equation 5-2.

- R = Concrete flexural strength
- H_c = Thickness of PCC slab
- E_{subg} = Modulus of subgrade
- H_{Stab} = Thickness of the stabilized subbase layer
- H_{Subb} = Thickness of the subbase layer
- Q_G = Gross weight of the aircraft

The analysis was conducted for PCC pavements built on two types of subgrade, with a modulus of 4500 psi, representing a very weak subgrade, and the other with a modulus of 30,000 psi, representing a very strong subgrade. The analysis was also conducted for three types of aircraft loads: a single tire with a 55,000-lb load, a B-727 aircraft with a gross weight of 200,000 lb, and a B-747 aircraft with a gross weight of 800,000 lb. The annual departures were set to 1200 for all. The LEDFAA program was used to calculate the slab thickness for 20 departure years for each aircraft. A rounded thickness value close to the required thickness was selected for the sensitivity analysis.

After a few numerical trials, it was found that pavement design life is very sensitive to concrete flexural strength, slab thickness, and aircraft gross weight; therefore, the values of Δx were selected as 1% to 1.5% of the values of the parameters. If the same level of Δx is used for analyzing the sensitivity of the thickness of stabilized subbase, the thickness of granular subbase, and the subgrade modulus, the difference between $L(x+\Delta x)$ and $L(x-\Delta x)$ will be too small to assure the accuracy of the calculation. Therefore, higher values are used for those three parameters. The detailed information on the input and output data is shown in table 5-3 for the weak subgrade and in table 5-4 for the strong subgrade.

The findings are summarized below.

- The numerical analysis of sensitivity had to be done case by case. The selected six cases covered subgrade strengths from weak to strong and covered the single, dual, and dual-tandem gears. The characteristics of each parameter's sensitivity were clearly seen in the numerical analysis.

- The three parameters with the highest sensitivity (the magnitudes of the calculated sensitivity) in each group were slab thickness, flexural strength, and aircraft gross weight.

- The sensitivity of subgrade strength of the weak subgrade was higher than the strong subgrade.

- The sensitivity of stabilized base thickness is always higher than granular base thickness. The difference between the two sensitivities for a pavement built on high-strength subgrade was more than for a pavement built on a weak subgrade.

TABLE 5-3. SENSITIVITY OF DIFFERENT PARAMETERS, WEAK SUBGRADE, E_C = 4500 psi

Single-wheel load 55,000 lb, 1200 annual passes, design PCC thickness = 14.09″						
Parameters	x	Δx	$L(x\text{-}\Delta x)$	$L(x)$	$L(x\text{-}\Delta x)$	$S_{x,\,L}$
R	700	7	19.9	17.7	15.8	11.6
H_c	14	0.2	23.2	17.7	13.6	19.0
E_{subg}	4,500	225	18.5	17.7	16.9	0.9
H_{Stab}	6	0.5	18.9	17.7	16.6	0.8
H_{Subb}	10	1	18.5	17.7	17	0.4
Q_G	55,000	550	15.8	17.7	19.6	-10.7
B-727, 200,000 lb, 1200 annual passes, design PCC thickness = 16.46″						
Parameters	x	Δx	$L(x\text{-}\Delta x)$	$L(x)$	$L(x\text{-}\Delta x)$	$S_{x,\,L}$
R	700	7	43.6	38.3	33.6	13.1
H_c	17	0.2	46.9	38.3	29.5	19.3
E_{subg}	4,500	225	39.7	38.3	36	1.0
H_{Stab}	6	0.5	41.2	38.3	35.3	0.9
H_{Subb}	10	1	39	38.3	36.6	0.3
Q_G	200,000	2,000	33.9	38.3	43.4	-12.4
B-747, 800,000 lb, 1200 annual passes, design PCC thickness = 15.34″						
Parameters	x	Δx	$L(x\text{-}\Delta x)$	$L(x)$	$L(x\text{-}\Delta x)$	$S_{x,\,L}$
R	700	7	16.2	14.4	12.8	11.8
H_c	15	0.2	17.6	14.4	11.8	15.1
E_{subg}	4,500	225	15.9	14.4	13	2.0
H_{Stab}	6	0.5	15.3	14.4	13.6	0.7
H_{Subb}	10	1	15.1	14.4	13.8	0.5
Q_G	800,000	8,000	12.9	14.4	16.2	-11.5

TABLE 5-4. SENSITIVITY OF DIFFERENT PARAMETERS—STRONG SUBGRADE, E_C = 30,000 psi

Single-wheel load 55,000 lb, 1200 annual passes, design PCC thickness = 13.48″						
Parameters	x	Δx	$L(x+\Delta x)$	$L(x)$	$L(x\text{-}\Delta x)$	$S_{x,\,L}$
R	700	7	12.4	11.1	10	10.8
H_c	13	0.2	14.2	11.1	8.7	16.1
E_{subg}	3,0000	6,000	11.3	11.1	11	0.1
H_{Stab}	6	0.5	11.7	11.1	10.6	0.6
H_{Subb}	10	1	11.2	11.1	11	0.1
Q_G	55,000	550	10.2	11.1	12.1	-8.6
B-727, 200,000 lb, 1200 annual passes, design PCC thickness = 15.36″						
Parameters	x	Δx	$L(x+\Delta x)$	$L(x)$	$L(x\text{-}\Delta x)$	$S_{x,\,L}$
R	700	7	15.4	13.7	12.2	11.7
H_c	15	0.2	16.9	13.7	11.2	15.6
E_{subg}	3,0000	3,000	14	13.7	13.5	0.2
H_{Stab}	6	0.5	14.3	13.7	13.2	0.5
H_{Subb}	10	1	13.9	13.7	13.6	0.1
Q_G	200,000	2,000	12.4	13.7	15.3	-10.6

TABLE 5-4. SENSITIVITY OF DIFFERENT PARAMETERS—STRONG SUBGRADE, E_C = 30,000 psi (Continued)

B-747, 800,000 lb, 1200 annual passes, design PCC thickness = 12.67″						
Parameters	x	Δx	$L(x+\Delta x)$	$L(x)$	$L(x-\Delta x)$	$S_{x,L}$
R	700	7	31.6	28	24.9	12.0
H_c	13	0.2	34.3	28	22.9	13.2
E_{subg}	30,000	3,000	29.5	28	26.5	0.5
H_{Stab}	6	0.5	29.6	28	26.7	0.6
H_{Subb}	10	1	28.4	28	27.6	0.1
Q_G	800,000	8,000	25.3	28	31.1	-10.4

5.4 SUMMARY.

Designing the thickness of a pavement needs many input parameters and different input parameters have different influences on pavement life. Pavement life is insensitive to some parameters, i.e., a small change in the parameter has only limited effects on pavement life. However, for other parameters, pavement life is very sensitive to changes in their values, i.e., a small change will lead to a significant change in pavement life. The following can be summarized from the results of the parametric sensitivity analysis that was presented in this section.

For flexible pavements, pavement life is most sensitive to aircraft gross weight and subgrade strength. Pavement life is more sensitive to P-209 crushed stone base thickness when compared to HMA modulus and HMA thickness. For rigid pavements, pavement life is most sensitive to slab thickness, flexural strength of the concrete, and aircraft gross weight. To compare the sensitivities to thickness and stiffness effects between flexible and rigid pavements, the two pavement types (rigid and flexible) were evaluated using the aircraft mix provided in table 5-1.

The flexible pavement cross section (from LEDFAA 1.3) is shown in figure 5-1. Increasing the P-401 surface thickness from 5 to 6 inches increased the predicted pavement life from 20 to 24.2 years. Increasing the P-401 modulus from 200,000 to 300,000 psi (50 percent increase in modulus) at 5 inches thick also increased pavement life from 20 to 24.2 years.

The rigid pavement cross section (from LEDFAA 1.3) is 21.37 inches of P-501 PCC surface, 6 inches of P-306 econocrete base, and 10 inches of P-209 subbase. Rounding off the P-501 thickness from the 20-year design thickness of 21.37 inches to 22 inches (an increase of 0.63 inches) increased the predicted pavement life from 20 to 32.7 years. Increasing the flexural strength of the P-501 from 700 to 735 psi (an increase of 5 percent) increased the pavement's predicted life from 20 to 40.2 years. This shows that a slight increase in thickness and flexural strength of P-501 has a much larger effect on pavement life compared to the changes in P-401 thickness and stiffness.

The last three sections presented the factors affecting pavement life from theoretical concepts (structural models, failure models, and changes in input parameters to structural models). Generally, the predicted pavement life is different from actual pavement life because of the design assumptions (uncertainties in material properties, climatic conditions, changes in load characteristics, etc.). In section 6, the field performance data from different airports is analyzed to study the conditions of the pavements and the distresses as the pavements approach the end of their design lives.

6. PAVEMENT STRUCTURAL LIFE ANALYSIS.

The evaluation of FAA thickness design procedures was conducted in three major steps.

Step one was based on the models used for the thickness design. The effects of different parameters on the pavement operational life due to the response and failure models were discussed in sections 3, 4, and 5.

Step 2 focuses on the available pavement surveyed data published in technical reports and papers. The major sources of the published data were obtained from references 6, 37, and 46-49.

Step 3 takes advantage of unpublished data collected from pavement condition surveys of airport pavements. In the past 10 years, the FAA has supported projects for collecting and analyzing field-surveyed data for special purposes. The major sources of the unpublished data were obtained from references 50 through 57.

The analysis of steps 2 and 3 are presented in this section.

6.1 CONCEPTS AND ASSUMPTIONS.

After reviewing the FAA design specifications in sections 3, 4, and 5, it was found that although a failure model can quantitatively predict the pavement structural life, many factors can affect pavement life. Theoretically, pavement life is a random variable rather than a deterministic unknown that can be solved by a deterministic procedure. Therefore, an appropriate evaluation of the suitability and reliability of a design procedure to predict pavement life can be conducted based on a statistical analysis. A brief discussion follows of the two most important statistical parameters, the mean and the standard deviations, that influence the reliability and suitability of a design procedure.

Since pavement life is a random variable, the probability density function of pavement life can be used to conduct the analysis. For simplification, pavement life is assumed to be subject to Gaussian (or normal) distribution, and its probability density function is defined in equation 6-1.

$$p(L) = \frac{1}{\sqrt{2\pi} \times \sigma} \times e^{-\frac{(L-m)^2}{2\times\sigma^2}} \tag{6-1}$$

where

L = pavement life

m = mean (average) of the pavement life

σ = standard deviation of the pavement life. Conceptually, the value of σ indicates the degree of variation of pavement life.

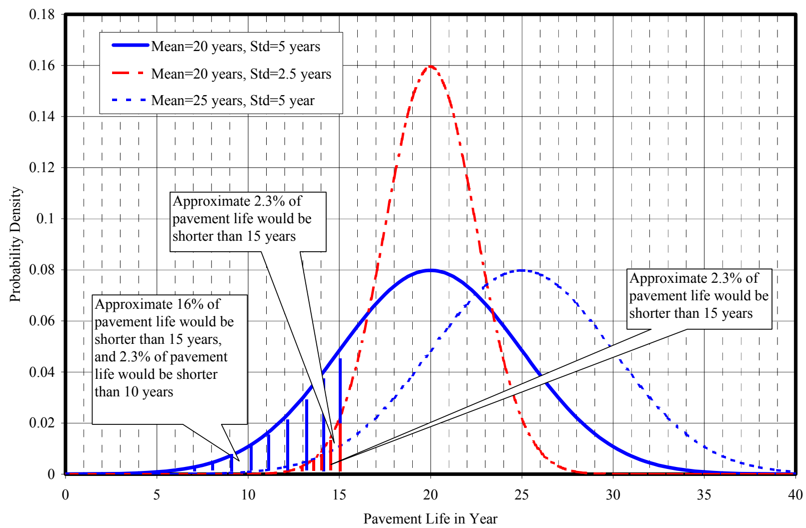
e = 2.71828

$p(L)$ = Probability density function of pavement life

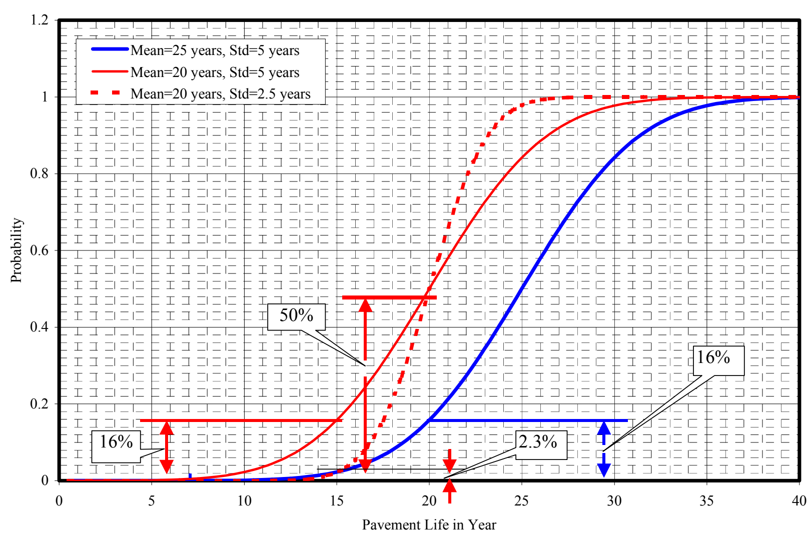
To illustrate the effects of the parameters, three groups were selected:

- Group I: m = 20 years and σ = 5 years
- Group II: m = 20 years and σ = 2.5 years
- Group III: m = 25 years and σ = 5 years

The probability density functions are presented in figure 6-1(a).



(a) Probability Density Function of Pavement Life



(b) Probability Function of Pavement Life

FIGURE 6-1. EFFECTS OF MEAN AND STANDARD DEVIATION ON PAVEMENT LIFE—CONCEPTUAL ANALYSIS

The probability of pavement life being shorter than *Le* (end of life) years can be calculated by following equation:

$$prob(L < Le) \quad = \quad \int_{-\infty}^{Le} p(L)dL \qquad (6\text{-}2)$$

Substituting equation 6-1 into 6-2, one can calculate the percentage of pavements shorter than *Le*. The probability functions for the above three groups are presented in figure 6-1(b).

The curve of pavement life from groups I and II have the same mean, 20 years, but different standard deviations. First, 50% of the pavements in these two groups will fail equal to or earlier than they are expected. Second, figure 6-1(b) indicates that about 16% of the pavements in group I will fail within 15 years and about 2.3% of the pavements will fail within 10 years. However, only 2.3% of the pavements in group II will fail within 15 years. Figure 6-1(b) also indicates that if the average of pavement life increases from 20 to 25 years and the standard deviation remains the same, the structurally failed pavements will drop from 50% to 16% at the end of 20 years. Two findings may be summarized:

- When the means of the two groups of pavements are the same, the pavements with smaller standard deviations (pavements in group II) are more reliable than those with larger standard deviations (in group I).

- When the standard deviations are the same, such as the pavements in groups I and III, the pavements with higher mean will be more reliable.

In general, the mean value of pavement life is controlled by the requirements of AC 150/5320-6D for design and by the materials preparation, construction, and maintenance quality control requirements of AC 150/5370-10A, AC 150/5320-12, AC 5360-8, and 14 CFR 134.

As discussed in sections 3, 4, and 5, pavement life is a random variable that has a relatively large standard deviation. Therefore, one should not expect that a single population containing pavements built in different environmental and geographic regions would have a very low standard deviation. The curve drawn from the pavement lives in group III shows that when the mean of pavement life increases to 25 years, only about 2.3% of the pavements fail within 15 years and 16% of the pavements fail within 20 years.

This is reasonable if the intent of the design procedure is to ensure that airport pavements substantially comply with the 20-year life requirement; the average life must be greater than 20 years. An increase in the average above 20 years will depend on the magnitude of the standard deviation (i.e., variability) of observed pavement life.

6.2 ANALYSIS PROCEDURES AND METHODS.

The evaluation of the suitability and reliability of pavement life determined by FAA design specifications cannot be conducted with theoretical analysis. Instead, a statistical analysis of field data must be used since

- many assumptions must be used in theoretical analysis, bringing unexpected errors into the analytical results. Statistical analysis does not use assumptions.

- factors which can neither completely nor appropriately be considered in a theoretical analysis can be considered in a statistical analysis because the data are collected from operational pavements experiencing the effects of the factors.

- statistical errors may be reduced to a low level if the number of samples is sufficiently large.

In 1990, 7964 airports in the United States had paved runways [52]. For such a large number of samples, one can estimate the probabilistic properties by analyzing the data in a selected sample group. Two parameters are often used to define the statistical properties of the data are

mean, expressed as

$$\mu_X = \frac{\sum_{i=1}^{n} X_i}{n} \tag{6-3}$$

where

x_i = values of samples selected
μ_x = sample mean
η = selected number of samples

and standard deviation, expressed as

$$\sigma_X = \frac{\sum_{i=1}^{n} (X_i - \mu)^2}{n - 1} \tag{6-4}$$

In the above two equations, n is the sample size selected from the population. The mean of the data indicates the average value of the data, and the standard deviation of the data indicates how the data are distributed around the mean.

If the total number of samples, N, in the population is greater than the selected number of samples n for analysis (if $N > n$), namely, only a portion of the pavement data are used, the mean of the selected samples will also be a random variable, Y (reference 58, section 10.1.3). The mean of Y is expressed as

$$\mu_Y = \mu_X \tag{6-5}$$

where

N = total number of samples
Y = random variable

This indicates that the mean of the selected data equals the mean of all the data.

The standard deviation of Y is expressed as

$$\sigma_Y = \sigma_X \times \sqrt{\frac{N-n}{n \times (N-1)}} \tag{6-6}$$

Equation 6-6 indicates that the more data used, the less variation and more reliable the statistical analysis will be.

As discussed in section 2, PCI is an index used to describe the pavement condition as affected by traffic loads, environment, and other sources. The PCI is calculated by equation 6-7 (reference 39, page 27):

$$PCI = 100 - a \times \sum_{i=1}^{m} \sum_{j=1}^{n_i} f\,(T_i, S_j, D_{ij}) \tag{6-7}$$

where

a	= adjustment factor (see ASTM D 5340)
m	= total number of distress type
n_i	= total number of severity levels for the ith distress
$f\,(T_i, S_j, D_{ij})$	= deduct value for distress type T_i, at the severity level S_j, existing at density D_{ij}

Another index, SCI, can be calculated by equation 6-8:

$$SCI = 100 - a \times \sum_{i=1}^{m_S} \sum_{j=1}^{n_i} f\,(T_i, S_j, D_{ij}) \tag{6-8}$$

The value of m in equation 6-7 is always equal to or greater than the value of m_S in equation 6-8 since m_S is the types of distresses related to the pavement structural failure and m is the number of all distress types. Therefore

$$PCI = SCI - all\ other\ deducts$$
$$all\ other\ deducts = D_E + D_M + D_C + D_O \tag{6-9}$$

where

D_E = deducts due to environment-related distresses such as raveling and weathering in HMA pavements

D_M = deducts due to material-related distresses such as popouts in PCC pavements

D_C = deducts due to construction-related distress such as bleeding in HMA pavements

D_O = deducts due to operations- and maintenance-related distresses such as patching or utility cuts in PCC pavement

The above statements may conceptually be used for both HMA and PCC pavements. The use of SCI = 80 to define the end of structural life of rigid pavements is well established. However, for flexible pavements, it is a new concept and has not been presented before. ASTM D 5340 uses 16 distresses for calculating the PCI value for HMA and 15 distresses for calculating the PCI value for PCC pavements. Most of the distresses are not solely structurally related, and they are contributed by multiple sources or by the interaction among two or more sources. Therefore, there is no unique definition of the distresses for SCI calculation.

In this study, the following six distresses used by Rollings [39] are used to calculate the deducts of SCI for PCC pavements.

- Corner break
- Longitudinal, transverse, and diagonal cracks
- Shattered slabs
- Shrinkage cracks
- Spalling along joints
- Spalling corners

Two distresses, alligator cracking and rutting, were used to calculate the SCI deducts for HMA pavements.

If an adequate quantity of surveyed pavements with detailed information on the above distresses is available and SCI = 80 is defined as the end of pavement structural life, curves similar to the curves in figures 6-1(a) and 6-1(b) can be generated.

Because of the time limitation for preparing this report, only distress survey data readily available to the authors was used. All of this data was collected before 1996, and although all of the distress survey reports contained PCI information, only a portion of them had detailed information on the types of distresses, with densities, suitable for calculating SCI values. Therefore, PCI had to be used for pavement operational life analysis in the following sections. However, equation 6-9 indicates that SCI is always greater than or equal to PCI. For example, if a group of pavements aged from 17 to 23 years has a mean PCI value of 80, one may easily conclude that the mean SCI value of the pavements must be greater than 80. Although the exact mean of pavement life based on SCI = 80 cannot be obtained using the existing data, the conclusion can still be made that the mean of pavement life is higher than 20 years for this example. Furthermore, by establishing a relationship between SCI and PCI from those survey reports containing sufficient information to compute SCI values, it is possible to make a better estimate of the mean pavement life for SCI = 80 than simply saying that it is higher than that given by PCI alone.

Theoretically, pavement structural life is defined as a pavement service period in years from the time the pavement is put into service until the time the value of SCI = 80 is reached. However, in practice, the end of pavement life is determined by the airport operator, or by other external factors, and may occur when SCI is significantly lower than 80, or when PCI has reached a threshold value requiring remediation even though the SCI is still significantly above 80. In addition, threshold values are different for pavements with different functional purposes (runway, taxiway, or apron) and when the pavements are located in different environmental or geographical regions. The threshold value also depends on the airport size: large, medium, or small hub, or general aviation airport.

6.3 DESCRIPTION OF SURVEYED HMA PAVEMENTS.

Eckrose/Green Associates provided all the field data used in this section of the report. The detailed procedures for surveying pavements may be found in reference 59. The data represent 30 airports from 10 states. There are a total of 2423 features with a combined area of 161 million square feet (msf), which is equivalent to the area of 107 standard runways (10,000 feet by 150 feet = 1.5 msf). Table 6-1 shows the distribution of surveyed pavements by state.

TABLE 6-1. GENERAL INFORMATION OF SURVEYED HMA PAVEMENTS

State	Airport ID	Number of Features	Airport Size	No. of HMA Runways	Total Area (msf)	Area of MHA Runways (msf)
MA	BOS	294	Large	5	18.94	5.52
TX	ELP	136	Medium	3	10.82	3.42
TX	HOU	54	Medium	1	2.38	1.14
TX	IAH	70	Large	3	4.98	3.51
FL	DAB	112	Small	3	6.4	2.79
FL	FLL	179	Medium	3	10.79	2.92
FL	GNV	76	Commuter	2	4.14	1.54
FL	JAX	8	Medium	1	0.3	1.5
FL	MLB	91	Small	3	6.3	2.55
FL	PBI	245	Medium	3	25.68	2.47
FL	PFN	88	Commuter	2	3.96	1.68
FL	PIE	114	Small	4	5.19	3.17
FL	PNS	80	Small	2	4.0	1.76
FL	RSW	35	Medium	1	4.35	1.80
FL	SRQ	63	Small	2	4.21	1.8
FL	TLH	71	Small	2	5.77	2.11
FL	TPA	63	Medium	3	4.4	2.3
IN	EVV	39	Commuter	3	2.73	2.32
IN	FWA	52	Small	2	3.04	3.08
MN	DLH	46	Commuter	1	2.66	0.85
TN	BNA	97	Medium	1	6.34	1.65
ND	BIS	49	Commuter	3	4.2	2.1

TABLE 6-1. GENERAL INFORMATION OF SURVEYED HMA PAVEMENTS (Continued)

State	Airport ID	Number of Features	Airport Size	No. of HMA Runways	Total Area (msf)	Area of MHA Runways (msf)
ND	FAR	13	Commuter	1	0.57	0.63
SD	FSD	26	Small	1	2.15	1.0
SD	RAP	64	Commuter	2	3.11	1.58
OK	TUL	59	Medium	1	2.77	0.92
WI	GRB	20	Small	1	0.83	0.24
WI	LSE	78	Commuter	3	5.18	2.87
WI	MKE	62	Medium	3	2.89	3.46
WI	MSN	39	Small	2	2.86	0.76
TOTAL		2423		67	161.94	63.44

Table 6-2 categorizes the surveyed data by airport size and area in relation to the total area in the United States.

TABLE 6-2. GENERAL INFORMATION ON HMA RUNWAYS IN THE U.S.

Airport Size	Total Area in U.S. (msf)	Surveyed Area (msf)	Percentage
Large Hub	88.7	9.03	10.2
Medium Hub	91.6	21.6	23.6
Small Hub	164.4	19.3	11.7
Total Hubs	344.7	49.93	14.5
Commuter	405.9	13.6	3.4

6.3.1 Analysis of Available Data for HMA Pavements.

6.3.1.1 General Information.

Figure 6-2 shows the average PCI for runways, taxiways, and aprons without differentiating the overlaid and non-overlaid pavements.

Figure 6-2 shows that a higher PCI is observed for younger pavements. The average PCI of pavements less than or equal to 15 years old are similar in all three groups. However, for the pavements older than 15 years, runways had the highest PCI and aprons had the lowest PCI.

If the pavements are separated into pavements with no overlay and pavements with an overlay, the average PCI seems to be independent of the number of overlays for the same functional pavement type (runway, taxiway, or apron) in different age groups. The results are shown in figures 6-3, 6-4, and 6-5 for runways, taxiways, and aprons, respectively, and are reported from the original (new pavement) construction date or from the date of last overlay (if the pavement was overlaid).

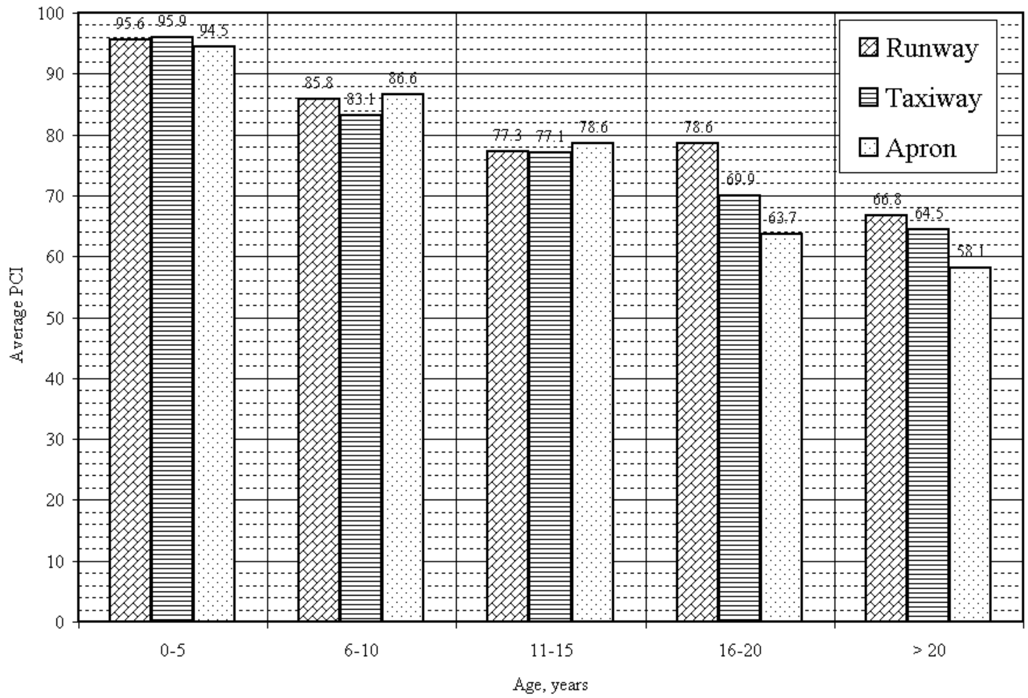


FIGURE 6-2. AVERAGE PCI FOR HMA RUNWAYS, TAXIWAYS, AND APRONS

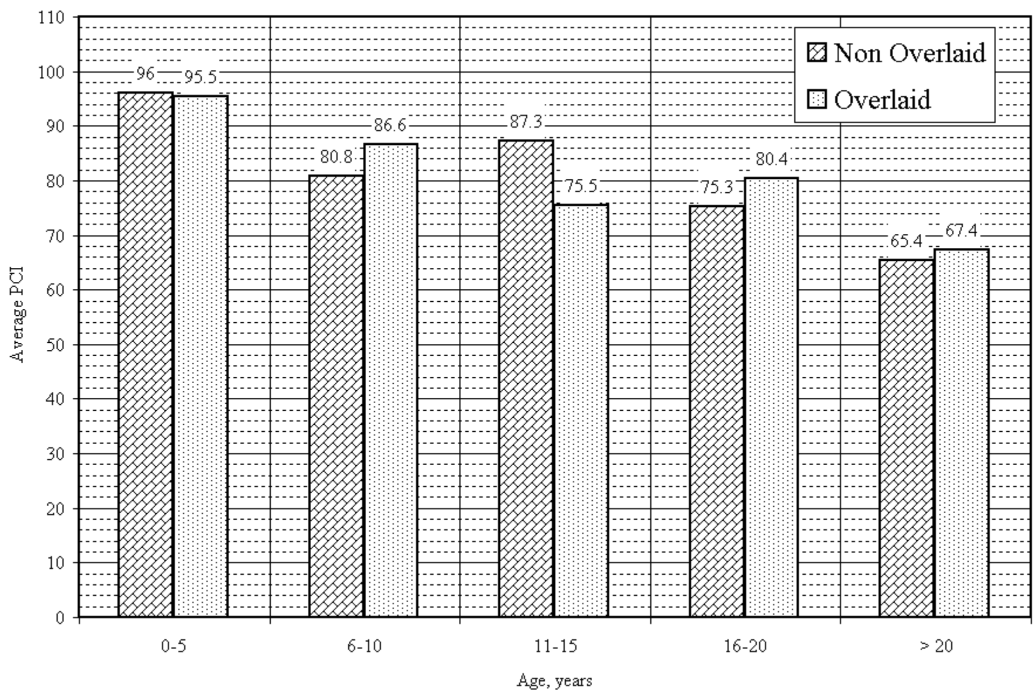


FIGURE 6-3. AVERAGE PCI FOR HMA RUNWAYS

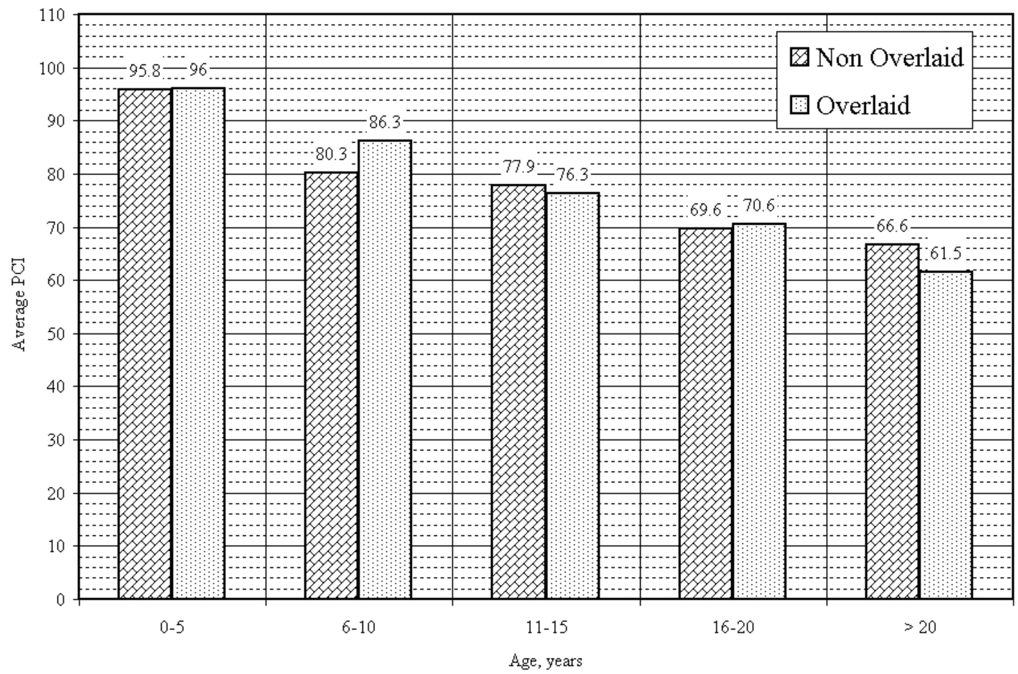


FIGURE 6-4. AVERAGE PCI FOR HMA TAXIWAYS

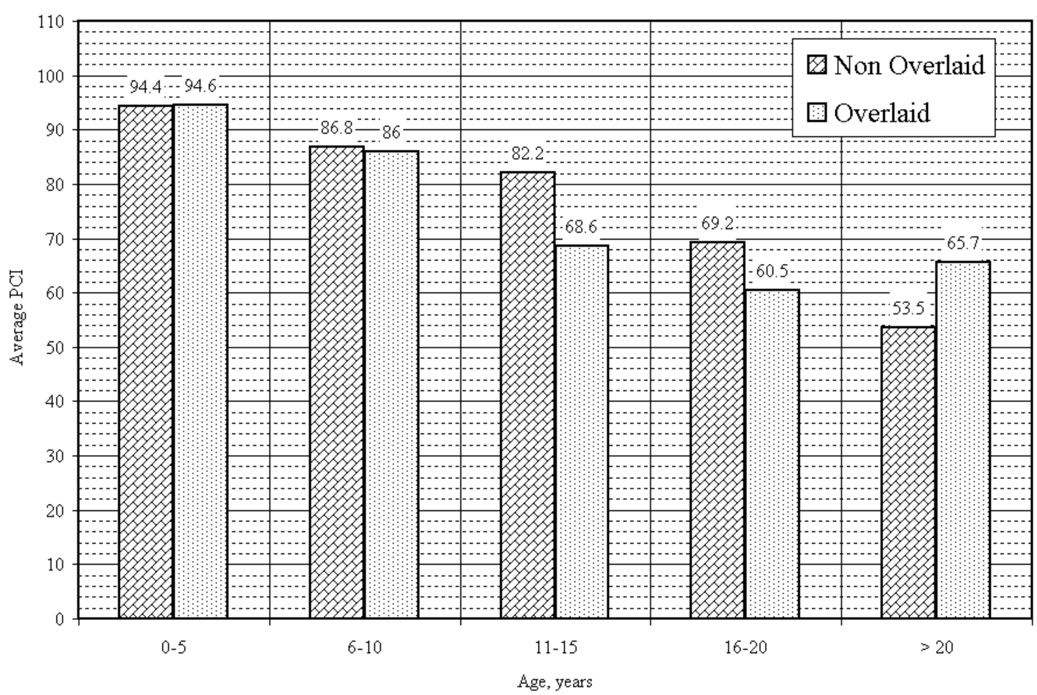


FIGURE 6-5. AVERAGE PCI FOR HMA APRONS

6.3.2 Distress Analysis.

6.3.2.1 Distress Groups.

AC 150/5380-6A [60] requires that PCI be determined based on the guidelines and procedures in ASTM D 5340 [3] for both PCC and HMA pavements. Sixteen distresses are identified for HMA pavements (discussed in section 2). For the purpose of this report, the distresses were divided into five groups as follows:

- Group-1: Cracking—Includes longitudinal and transverse cracks, alligator (or fatigue) cracking, block cracking, slippage cracking, and reflection cracking.

- Group-2: Disintegration—Includes raveling and weathering.

- Group-3: Distortion—Includes rutting, corrugation, shoving, depression, and swelling.

- Group-4: Loss of Skid Resistance—Includes bleeding, polished aggregate, and fuel spillage.

- Group-5: Others—Includes jet blast and patching distresses.

The accumulated deduct values due to distresses in a group are defined as the reduction of PCI for the group. Figure 6-6 shows the importance of different distress groups in HMA pavements. For example, about 82 percent of the group PCI reduction for runway pavements were contributed by distress group 1 (cracking), and 8.1 percent of it was contributed by distress group 3 (distortion).

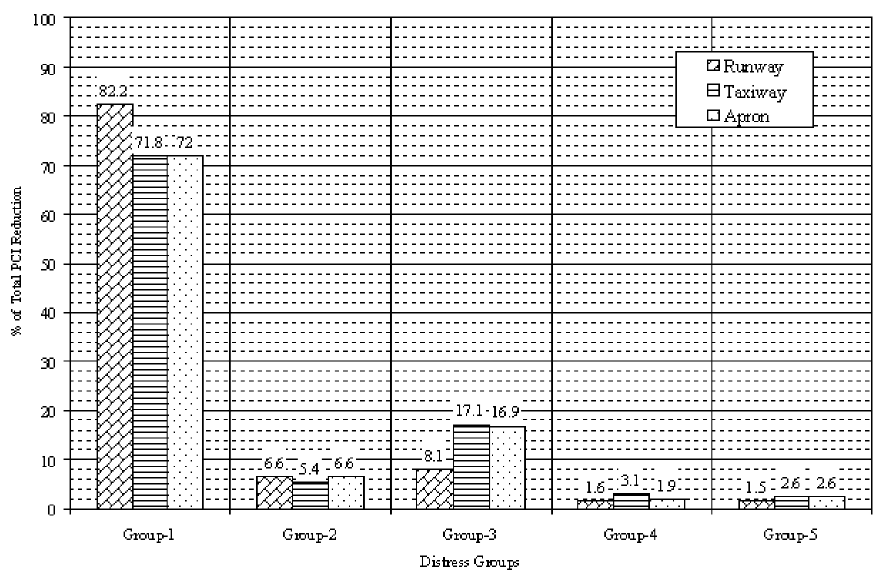


FIGURE 6-6. PAVEMENT CONDITION INDEX REDUCTION BY DISTRESS GROUPS FOR HMA PAVEMENTS

Figure 6-6 shows that

- Group 1 distresses (cracking) are the main source of PCI reduction for all three types of flexible pavements (runways-82.2%, taxiways-71.8%, and aprons-72%).

- Group 3 distresses (distortion) are the second most important reason for PCI reduction. The PCI reduction for runways, taxiways, and aprons are 8.1, 17.1, and 16.9 percent, respectively, or one-tenth to one-fifth of group 1 distresses.

- The runways have more cracks than taxiways and aprons, whereas taxiways and aprons have more distortion distresses, such as rutting.

- The loss of skid resistance (group 4) causes PCI reductions of 1.6, 3.1, and 1.9 percent for the runways, taxiways, and aprons, respectively.

- Disintegration distresses (group 2) PCI reductions for runways, taxiways, and aprons are 6.6, 5.4, and 6.6 percent, respectively.

- The PCI values reduced by jet blast and patching (group 5) distresses for the three types of pavements are 1.5, 2.6, and 2.6 percent, respectively.

6.3.2.2 Individual Distresses.

Table 6-3 shows the PCI reduction from the total deduct PCI value by the 16 distresses. Only 5 distresses among the 16 caused PCI reductions higher than 2. Each of the five distresses cause the PCI deduct values to be higher than 5 or 10 percent or more of the total surveyed area. They are longitudinal and transverse cracking, alligator cracking, rutting, block cracking, and raveling.

The deduct value used to calculate the PCI of pavements not only depends on the sum of the deduct values of all distresses observed, but also on the number of distresses that cause deduct values of PCI greater than 5. Therefore, the above five distresses may be defined as the most often observed and influential distresses among the sixteen. The total percent of PCI reduction attributed to these five distresses is 88.4 percent for all pavement types (runways, taxiways, and aprons).

Table 6-4 shows the effect of six major distresses on different pavement groups (runways, taxiways, and aprons).

TABLE 6-3. PAVEMENT CONDITION INDEX REDUCTION BY INDIVIDUAL DISTRESSES

Distress	PCI Deduct	PCI Deduct by Percent of Total PCI	Percent of Area with PCI Deduct > 5
Alligator Cracking	12.27	29.69	43.8
Bleeding	0.81	1.96	3.3
Block Cracking	2.91	7.04	10.6
Corrugation	0.01	0.02	0.1
Depression	0.81	1.95	5.1
Jet Blast	0.01	0.02	0.0
Joint Reflection Cracking	1.61	3.89	6.8
Longitudinal and Transverse Cracking	14.16	34.25	62.1
Oil Spillage	0.13	0.32	0.2
Patching	0.93	2.26	5.2
Polished Aggregate	0.01	0.03	0.2
Raveling and Weathering	2.52	6.08	11.5
Rutting	4.62	11.17	20.7
Shoving from PCC	0.0	0.0	0.0
Slippage Cracking	0.05	0.12	0.4
Swell	0.49	1.18	3.2

TABLE 6-4. PAVEMENT CONDITION INDEX REDUCTION BY SIX MAJOR DISTRESSES

<table>
<tr><th colspan="2">PCI Reduction</th><th>Long./ Tran. Cracks</th><th>Alligator Cracking</th><th>Rut-ting</th><th>Block Cracking</th><th>Rav-eling</th><th>Joint Reflection Cracks</th><th>Area (msf)</th><th>Ave. PCI</th><th>Ave. Age</th></tr>
<tr><td rowspan="2">All Runway</td><td>Value</td><td>15.5</td><td>8.6</td><td>2.0</td><td>2.0</td><td>2.2</td><td>1.3</td><td rowspan="2">54.1</td><td rowspan="2">83.2</td><td rowspan="2">10.7</td></tr>
<tr><td>Percent</td><td>46.4</td><td>25.8</td><td>5.9</td><td>6.0</td><td>6.6</td><td>3.9</td></tr>
<tr><td rowspan="2">All Taxiway</td><td>Value</td><td>12.5</td><td>13.0</td><td>6.0</td><td>2.4</td><td>2.2</td><td>1.2</td><td rowspan="2">63.0</td><td rowspan="2">81.3</td><td rowspan="2">11.0</td></tr>
<tr><td>Percent</td><td>30.9</td><td>32.0</td><td>14.7</td><td>5.9</td><td>5.4</td><td>2.9</td></tr>
<tr><td rowspan="2">All Apron</td><td>Value</td><td>15.3</td><td>17.7</td><td>6.7</td><td>5.8</td><td>3.8</td><td>3.1</td><td rowspan="2">28.3</td><td rowspan="2">76.0</td><td rowspan="2">14.1</td></tr>
<tr><td>Percent</td><td>26.2</td><td>30.4</td><td>11.5</td><td>10.0</td><td>6.6</td><td>5.4</td></tr>
</table>

Figures 6-7 and 6-8 show the effects of six distresses on different functional pavements.

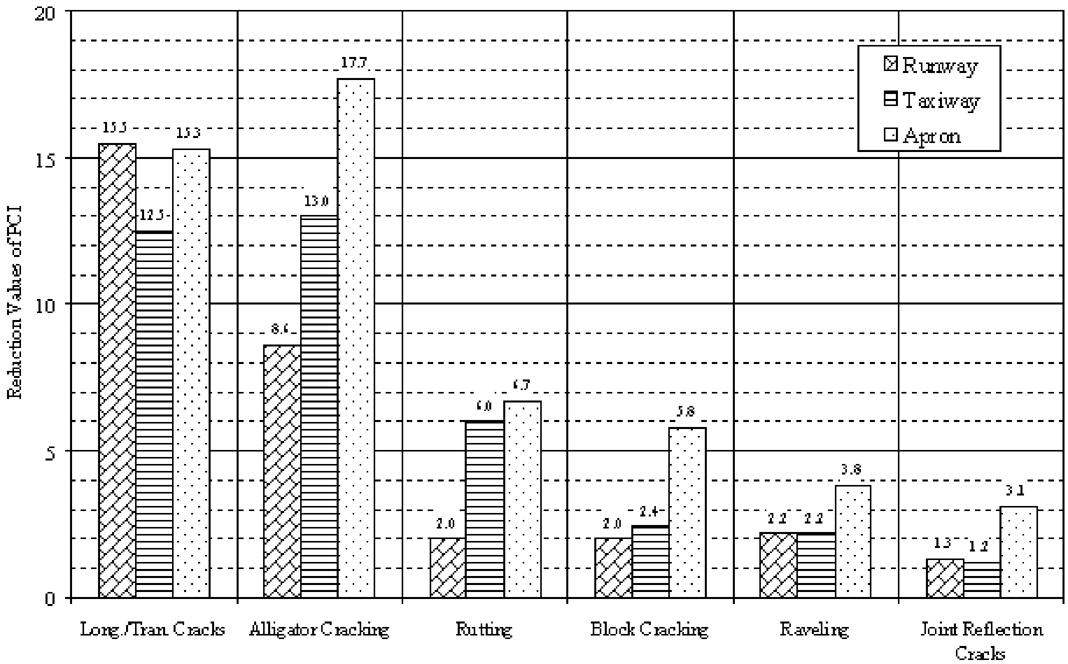


FIGURE 6-7. PAVEMENT CONDITION INDEX REDUCTION VALUES FOR DIFFERENT FUNCTIONAL HMA PAVEMENTS

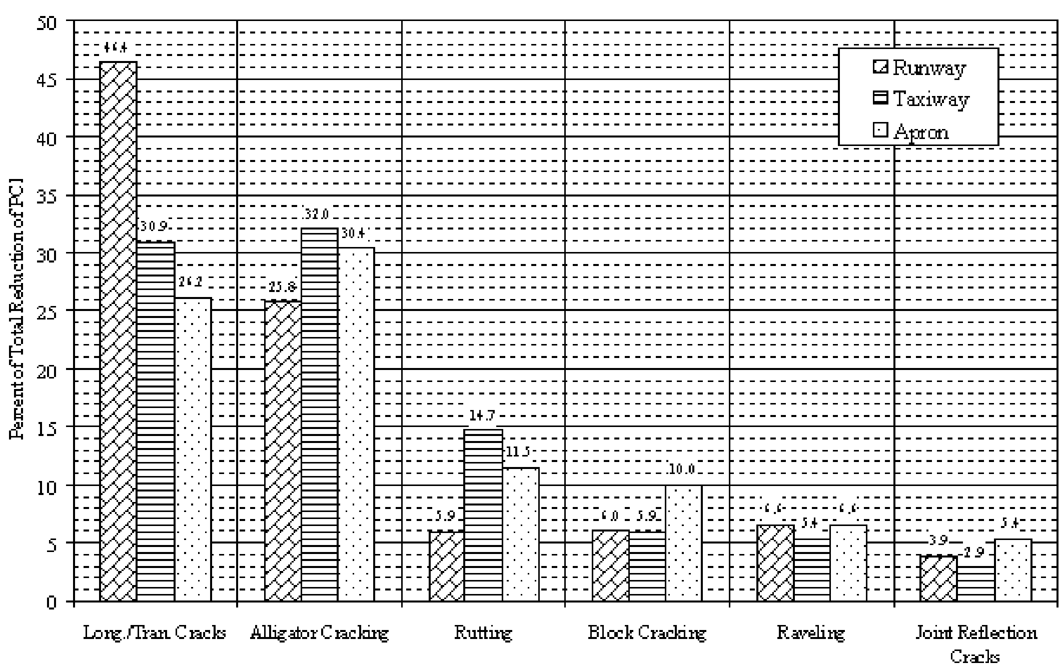


FIGURE 6-8. PERCENT OF TOTAL REDUCTION OF PCI FOR DIFFERENT FUNCTIONAL HMA PAVEMENTS

Figures 6-7 and 6-8 show that alligator cracking (load-induced distress) caused the highest reduction of PCI for aprons, followed by taxiways and runways. In other words, aircraft load-induced problems are more serious on aprons and taxiways compared to runways. Longitudinal and transverse cracking is the major source of PCI reduction on runways. This type of distress is mainly caused by construction and environmental factors. Less rutting was observed on runways compared to taxiways and aprons.

Further analyses of the data showed that non-overlaid pavements experienced more rutting and alligator cracking compared to overlaid pavements. The longitudinal and transverse cracking and reflection cracking cause more PCI reduction on overlaid pavements than on non-overlaid pavements.

As far as pavement age is concerned, longitudinal and transverse cracking is the number one source of PCI reduction for younger pavements (1 to 8 years), while alligator cracking becomes more serious for older pavements (17 to 23 years). The importance of rutting on PCI reduction decreases when the pavement gets older.

6.3.2.3 Structural Condition Index.

SCI is defined as the structural component of the PCI. For rigid pavements, SCI is clearly defined, and there is a procedure to calculate it [39]. Currently, for flexible pavement, there is no definition for SCI or a procedure to calculate it. This report used two distresses, alligator cracking and rutting, to calculate deducts of SCI for flexible pavements. Alligator cracking is load-related and is a structural problem. Rutting is a structural problem but could be caused by non-load-related mechanisms (such as poor asphalt mix design, inadequate compaction in the asphalt, base, or subbase layer). The procedure developed to calculate SCI for flexible pavements is described below.

Figure 6-2 shows the PCI values for runways, taxiways, and aprons. Table 6-4 lists the PCI reduction by six major distresses for all types of pavements (runways, taxiways, and aprons). It is assumed that the PCI deduct for a given pavement type (runway, taxiway, or apron) is the same for the pavement of all ages. For runway pavements, from table 6-4, the SCI deduct (DSCI) is calculated as the sum of PCI reduction due to alligator cracking and rutting (25.8+5.9 = 31.7 percent). For taxiways and aprons, the DSCI is calculated as 46.7 percent (32+14.7) and 41.9 percent (30.4+11.5) respectively.

The SCIs were calculated for flexible pavements using equation 6-10.

$$SCI = 100 - (100 - PCI) \times \frac{DSCI(\%)}{100} \qquad (6\text{-}10)$$

Where *DSCI(%)* is the area-weighted distress deducts due to the load related distresses.

The results are shown in table 6-5 and figure 6-9.

TABLE 6-5. STRUCTURAL CONDITION INDEX COMPUTATION FOR HMA AIRFIELD PAVEMENTS

Pavement	Age (years)	PCI	DSCI (%)	SCI
Runway	0-5	95.6	31.7	98.6
	6-10	85.8		95.5
	11-15	77.3		92.8
	16-20	78.6		93.2
	> 20	66.8		89.5
Taxiway	0-5	95.9	46.7	98.1
	6-10	83.1		92.1
	11-15	77.1		89.3
	16-20	69.9		85.9
	> 20	64.5		83.4
Apron	0-5	94.5	41.9	97.7
	6-10	86.6		94.4
	11-15	78.6		91.0
	16-20	63.7		84.8
	> 20	58.1		82.4

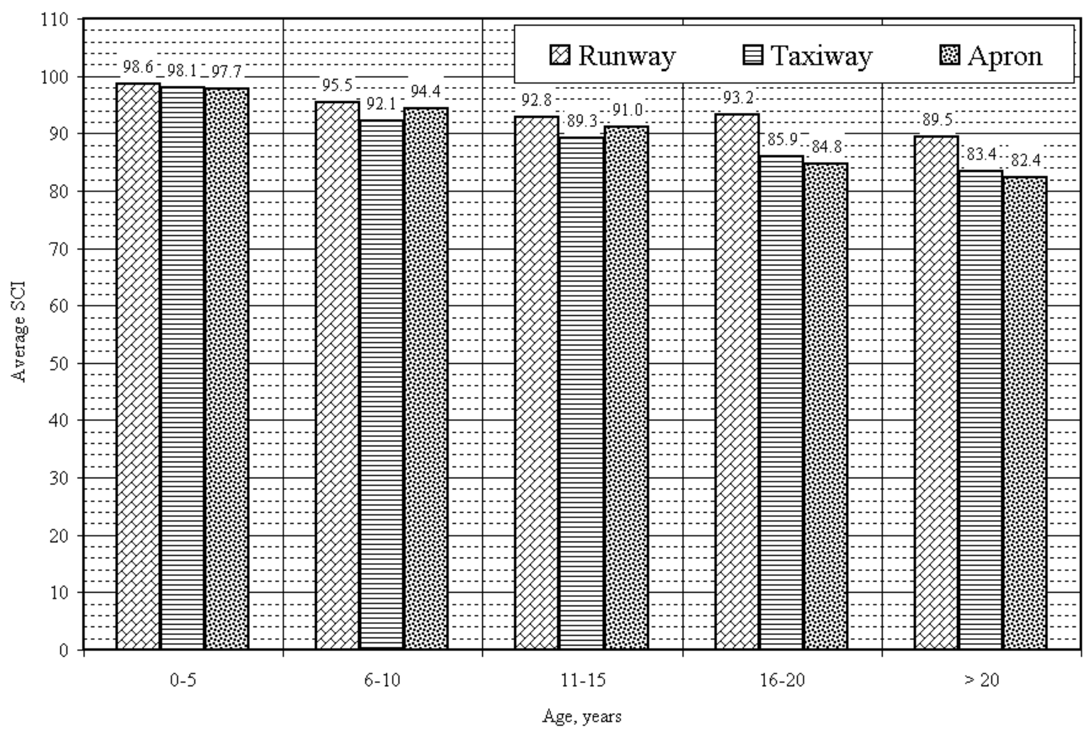


FIGURE 6-9. AVERAGE SCI FOR HMA RUNWAYS, TAXIWAYS, AND APRONS

Only load-related distress deducts were considered for computing the SCI. The load-related distresses for flexible pavements are alligator cracking and rutting (table 2-1). Figure 6-9 shows that the average SCI for pavements older that 20 years is higher than 80.

The average SCI for age > 20 years for runways is approximately 90. This is higher than the taxiways (≈ 83) and aprons (≈ 82). Therefore, in general, the SCI of runways is higher than the SCI of aprons and taxiways. The slow speed of aircraft on taxiways and aprons results in lower HMA stiffness (since HMA is a visco-elastic material) and longer load durations. Both these factors are related to HMA fatigue (alligator cracking) and pavement rutting.

6.4 AVAILABLE DATA FROM PUBLISHED REPORTS.

Kohn [46] evaluated the FAA design procedures for high-traffic volume pavements. The airports studied, with flexible pavements, were Phoenix Sky Harbor International Airport (PHX) in Arizona and JFK in New York. The information collected from these sites was construction data (materials, thickness, strength, and physical properties), PCI, traffic history, and nondestructive test (NDT) data (deflection basins, dynamic stiffness modulus, etc.).

6.4.1 Phoenix Sky Harbor International Airport.

All the pavement sections at PHX were conventional flexible pavements varying in age from 2 to 16 years (at the time of the study) with a mean age of 8.5 years. A pavement section consisted of asphalt surface, aggregate base course, select material subbase, and natural subgrade. One section was constructed with a soil-cement subbase. Table 6-6 summarizes the PCI data.

TABLE 6-6. PAVEMENT CONDITION INDEX SUMMARY FOR HMA PAVEMENTS AT PHX

Section No.	Age (years)	Total No. of Sample Units	Average PCI	Standard Deviation	Percent Deduct Values Based on Distress Mechanism		
					Load	Environment	Other
1	7	16	29	7.9	55.2	1.4	43.2
2	16	17	54	10.4	72.1	17.6	10.4
3	2	19	65	11.5	45.9	0.0	54.1
4	14	13	94	8.4	59.2	40.8	0.0
5	4	17	43	4.3	24.0	3.5	72.5
6	4	8	48	1.7	0.0	6.2	93.8
7	5	9	70	24.1	52.3	2.8	44.9
8	15	19	40	14.1	91.1	8.9	0.0

The condition of the pavements ranged from poor to excellent with an average PCI of 55.4 and a standard deviation of 20.5. The large range was due to the age of the pavements (2 to 16 years). Two types of problems were reported in the pavements: (1) fatigue or alligator cracking of the asphalt surface in the older pavements (load-related) and (2) excessive bleeding in the newer pavements (construction- or material-related). FAA thickness designs were reported in reference 46 using NDT-determined material properties and design values. The designs based on NDT properties were very close to existing designs (except for pavement sections 2 and 5, where the

FAA required granular material thicknesses based on NDT properties were larger than the existing thicknesses). The existing surface thickness was equal to or greater (1 to 2 inches) than the required thickness. Using the design values, the thicknesses obtained from the FAA design procedures were generally less than the existing pavement thicknesses (except for pavement section 3, which required an additional 6 inches of granular material to match the FAA design based on NDT properties).

The pavement condition was assessed using USACE failure criteria (rutting, upheaval, and cracking), and none of the pavement sections were declared failed. Considering the age of the pavements, it was reported that the design procedure was adequate for rutting criteria; however, a criterion was needed for asphalt surface fatigue cracking. The report revealed a limitation to the construction specifications used in conjunction with the design method. It recommended inclusion of a method of using local material specifications when environmental conditions may have a significant effect on the pavement's performance.

6.4.2 John F. Kennedy International Airport (identified as KIA in reference 46).

There were three flexible pavement sections at JFK. Two of the sections had an HMA surface and a bituminous macadam base course. The third section consisted of an HMA surface over a lime-cement-fly ash base course. Table 6-7 summarizes the PCI data.

TABLE 6-7. PAVEMENT CONDITION INDEX SUMMARY FOR HMA PAVEMENTS AT JFK

Section No.	Age (years)	Total No. of Sample Units	Average PCI	Standard Deviation	Percent Deduct Values Based on Distress Mechanism		
					Load	Environment	Other
1	5	11	92	7.1	67.5	14.4	18.1
2	20	25	50	9.9	69.4	27.6	3.0
3	13	12	76	3.2	0.0	97.9	2.1

The average PCI for the three flexible pavement sections was 72.6 (very good) with a standard deviation of 21.9. The primary distress observed in the flexible sections was random cracking and some small areas of fatigue cracking. No rutting caused by shearing of subgrade was observed. The actual pavement thicknesses compared well with the design thicknesses using NDT data, except for pavement section 3 where the thickness difference was 24 inches (actual thickness provided was more than the design thickness).

The general conclusion stated in the report was that the FAA design procedure was adequate for design against subgrade rutting but needs a method to control fatigue cracking.

6.4.3 Additional Data and Information.

A 1982 study by the General Accounting Office (GAO) [61] looked at the condition of runways at 46 small airports. The study concluded that the condition of the runways at these airports was deteriorating because of deferred maintenance. The report does not go into detail about the type of distresses observed. It does mention some surface deficiencies such as cracks. Any functional distresses in a pavement surface, if left unrepaired, may lead to the structural failure of the pavement. The report highlights the need of timely repair to prevent rapid deterioration of runways.

In 1998, the GAO performed a study on 1154 National Airspace System Airports [62] and analyzed the PCI data. The study reported that the runways were in good condition. Seventy-seven percent of the runways in the GAO's runway pavement database were ranked in the three highest categories—excellent, very good, and good. Eleven percent were rated fair, and twelve percent were ranked poor, very poor, or failed. The report does not specify whether the pavements in poor condition were due to load-related distresses or functional distresses. The PCI data were not available for further analysis in this study.

Grogan [47] undertook a study to evaluate the performance of stabilized base courses. All the pavement sections studied with asphalt-stabilized base had an HMA surface. They reported that all the pavements except one had major rehabilitation or reconstruction in the past 10 years (from date of study). Most of the rehabilitation work was limited to the surface course, such as grinding off and replacing with a like or greater thickness of HMA. Distresses observed were all mix design (materials problem) related, such as bleeding and rutting of HMA rather than structural distresses.

6.5 DESCRIPTION OF SURVEYED PCC PAVEMENTS.

An FAA project was conducted in 1996 to evaluate the influence of slab size on the performance of PCC pavements by analyzing the airport survey data in conjunction with theoretical analysis. A technical report [63] was published in April 2000. The data from this report included 288.4 msf of pavement data from 2820 features (for a definition see AC 150/5380-6 [60], the same as pavement section defined in ASTM D 5340) of 174 airports. These airports are located in 23 states (six FAA regions and Hawaii) and one in Japan, three of which are military airports: Marine Corps Air Stations Futenma, Okinawa, Japan; Kaneohe Bay, Hawaii; and El Toro, California. The number of features and corresponding areas of the pavements in each state and FAA region are listed in table 10 of reference 63.

The percentages of pavements grouped by function are shown in figure 6-10. The percentage of pavements surveyed from runway, taxiway, and apron are almost the same. The percentages of pavements grouped by the years having been serviced are presented in figure 6-11. About 38% of all surveyed PCC pavements older than 20 years were still in service during the survey. It should be noticed that the real lives of the pavements in this group are longer than 20 years. Of the pavements younger than 20 years, about one-third are between 16 to 20 years. The surveyed pavements with age between 11-15 years are less than all other groups (only 10% of total pavements). The data for preparing the FAA report [63] were also used for the analysis of this report.

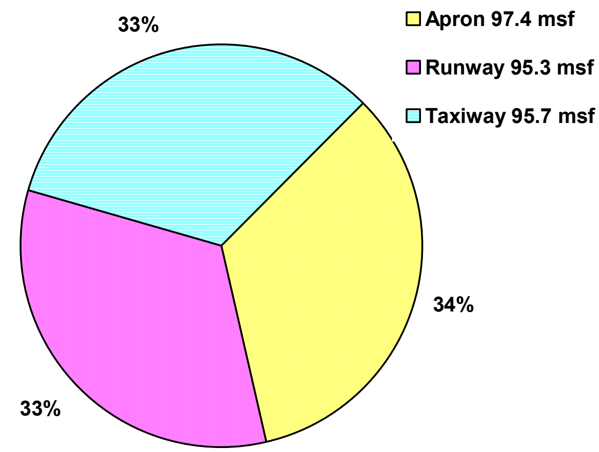


FIGURE 6-10. SURVEYED PCC PAVEMENTS GROUPED BY FUNCTION

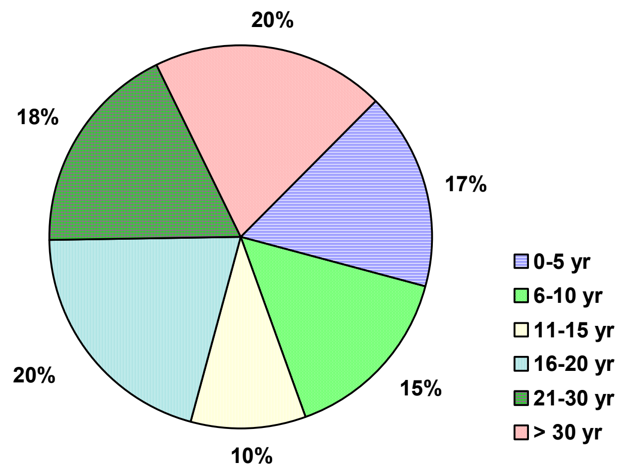


FIGURE 6-11. SURVEYED PCC PAVEMENTS GROUPED BY YEARS OF SERVICE

6.6 ANALYSIS OF SURVEYED PCC PAVEMENTS.

The average PCI values of all surveyed pavements grouped by age are shown in figure 6-12. The average PCI decreases when the age increases. The average PCI in the pavement group between 21 to 30 years old is 79. As discussed previously, the SCI of a pavement is always equal to or greater than the PCI of the pavement since the deducts for PCI are attributed to all 15 types of distresses and the deducts for SCI are only attributed to 6 distresses. Therefore, the average SCI of the pavements in this group should be greater than 80. Though the data are sufficient to show the average pavement life, figure 6-12 still shows that a large amount of pavements (57.6 msf among the 288 msf, about 20% of the total surveyed pavement) still had an SCI greater than 80 after 21 to 30 years of service. This implicitly indicates that the average structural life of PCC pavements is longer than 20 years.

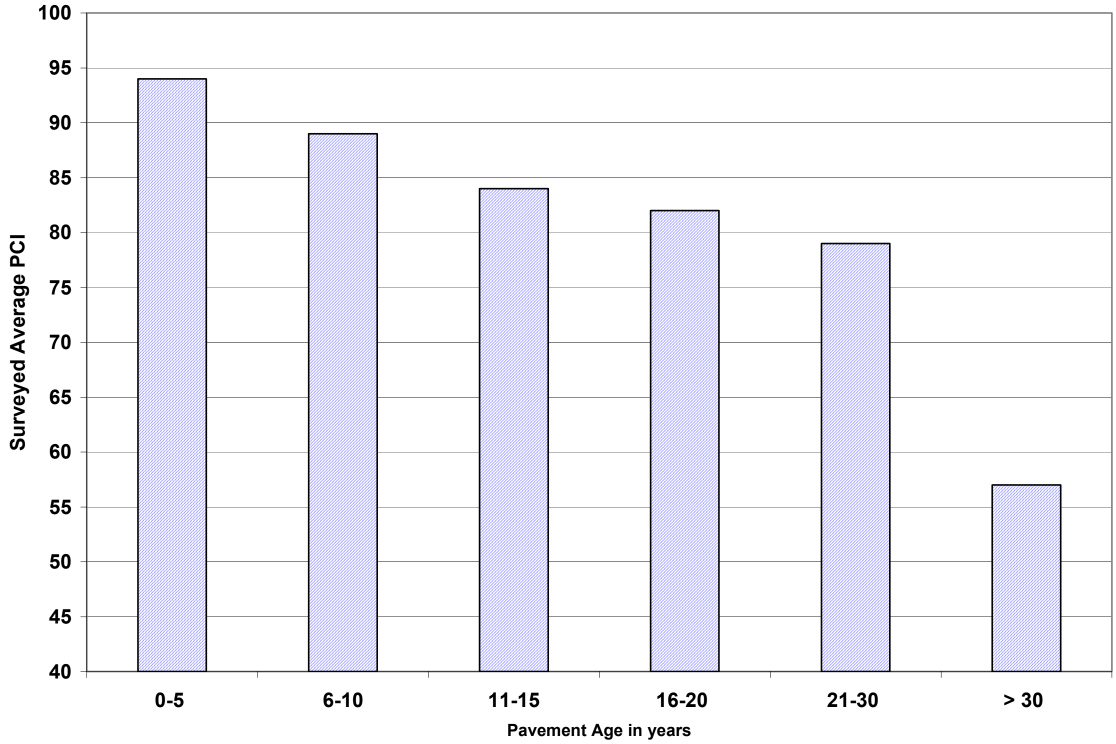


FIGURE 6-12. PAVEMENT CONDITION INDEX OF THE PCC PAVEMENTS BY AGE

Figure 6-13 presents more detailed PCI values for different functional pavements: runways, taxiways, and aprons. The PCI values of surveyed apron pavements are the lowest in each age group. This may be attributed to different maintenance procedures used for runways, taxiways, and aprons. This may also be caused by the loading position of the aircraft on the aprons. One wheel at a slab corner is frequently observed only on pavement aprons and is most critical to the development of top-down corner cracks.

As concluded in reference 63, slab size is a sensitive factor to the PCI value. Figure 6-14 clearly verifies that a large slab size leads to quicker deterioration.

The PCI distributions for three functional pavements from 16 to 23 years old are presented in figure 6-15. About 80% of the runway pavements in that age group had a PCI ≥ 80. However, only 45% of the apron pavements in that age group had a PCI ≥ 80. Different functional pavements would have different requirements for operation. Therefore, different threshold values are needed for determining the need for major rehabilitation or reconstruction of a PCC pavement.

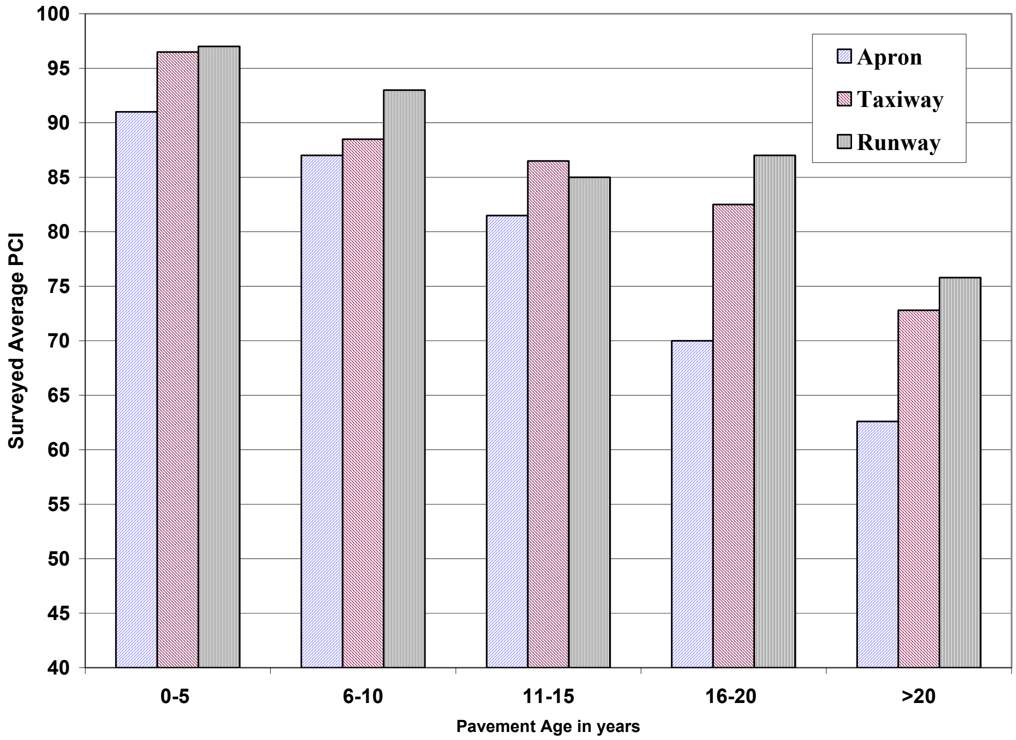


FIGURE 6-13. PAVEMENT CONDITION INDEX OF PCC PAVEMENTS BY AGE AND FUNCTION

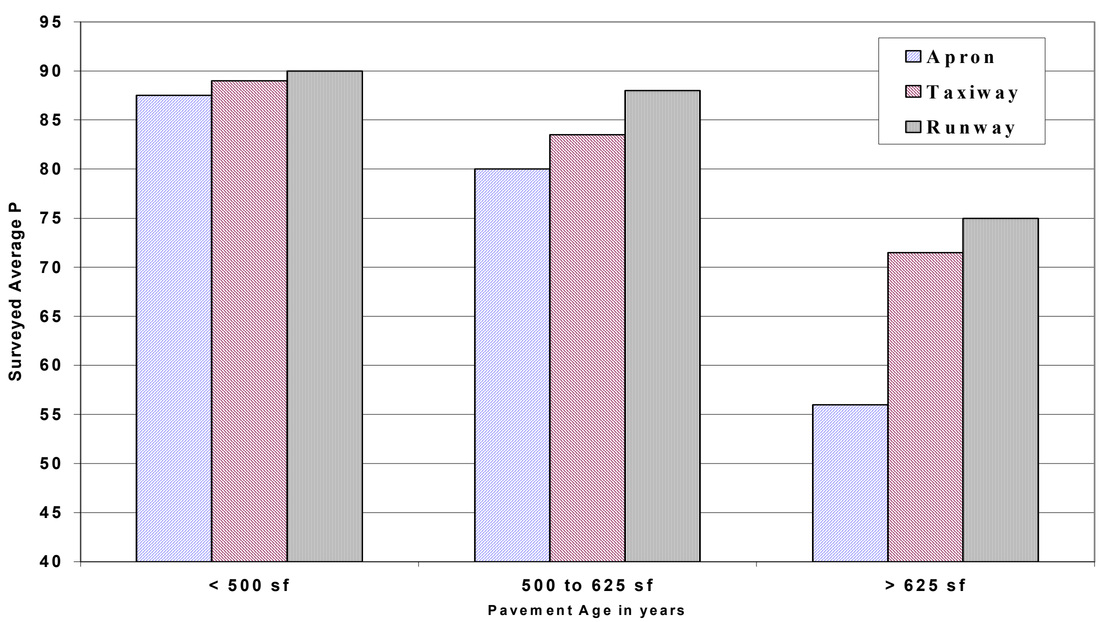


FIGURE 6-14. PCI_{AVG} OF 0- TO 30-YEAR PCC PAVEMENTS GROUPED BY SIZE

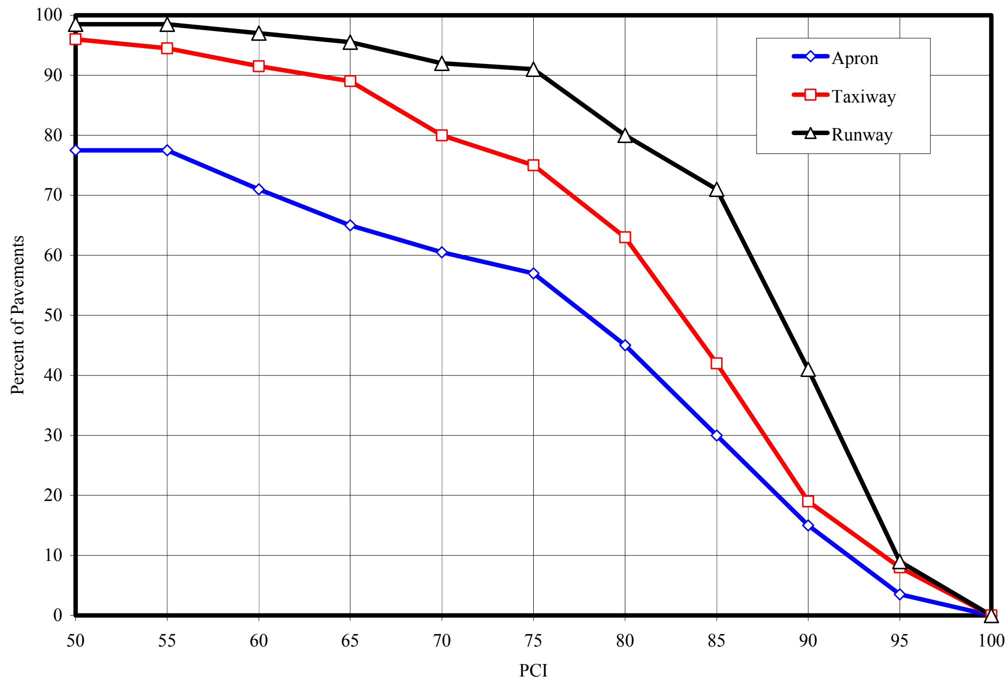


FIGURE 6-15. DISTRIBUTION OF PCI FOR ALL PCC PAVEMENTS BETWEEN 16 AND 23 YEARS

A rough procedure was used in this report to approximate the SCIs from the surveyed PCI values. Pavement survey data, where the load-related and non-load-related distresses were available were analyzed. Equation 6-10 was used to estimate the SCI value from the surveyed PCI values.

Two additional assumptions were also used:

- The DSCI(%) depends on the function of the pavements and is different for runways, taxiways, and aprons.

- The DSCI(%) is the same for pavements in different age groups so the SCI of pavements in different age groups may be estimated by using the same DSCI(%) if the pavement types are the same.

R.D. McQueen and Associates collected the data for the projects at the Memphis International Airport (MEM) and Washington Dulles International Airport (IAD). All distresses are divided into load-related (six distresses) and non-load-related (all other) distresses. The reorganized data are listed in table 6-8. By using the DSCI(%) in table 6-5 and combining the last two groups (age between 21 to 30 and > 30) into one group (age > 20), the SCI of runways, taxiways, and aprons for all age groups are plotted in figure 6-16.

TABLE 6-8. AREA-WEIGHTED DSCI BY FEATURE

Airport	Section	Area (sf)	Age (years)	PCI	DSCI in Percent
Runway: The area-weighted DSCI = 35.9					
MEM	RW18R-01	466,875	N/A	70	41
MEM	RW18R-01A	934,375	N/A	90	50
IAD	RW1R-01	170,000	44	90	6
IAD	RW1R-1L	170,000	44	75	22
IAD	RW1R-1R	170,000	44	69	45
IAD	RW1R-02	235,000	44	83	10
IAD	RW1R-2L	235,000	44	74	44
IAD	RW1R-2R	235,000	44	70	43
IAD	RW1R-03	171,000	44	86	8
IAD	RW1R-3L	171,000	44	69	31
IAD	RW1R-3R	171,000	44	75	27
Taxiway: The area-weighted DSCI = 38.5					
MEM	TWM-02	66,250	N/A	64	56.68
MEM	TWM-03	225,000	N/A	68	26.98
MEM	TWM-04	57,0000	N/A	44	28.13
IAD	TLD-03	457,500	25	21	37
IAD	TLD-04	67,500	4	90	70
IAD	TLE-01	168,777	14	52	63
IAD	TLE-02	90,437	14	55	62
IAD	TLE-03	145,716	14	60	46
IAD	TWA-02	27,580	15	63	44
IAD	TWB-02	27,232	15	78	40
IAD	TWE-02	82,180	9	96	17
Apron: The area-weighted DSCI = 20.3					
IAD	APRBR-01	357,500	9	65	8
IAD	APRBR-02	385,000	9	74	11
IAD	APRBR-03	319,688	9	73	42
IAD	APRBR-04	649,300	7	73	22

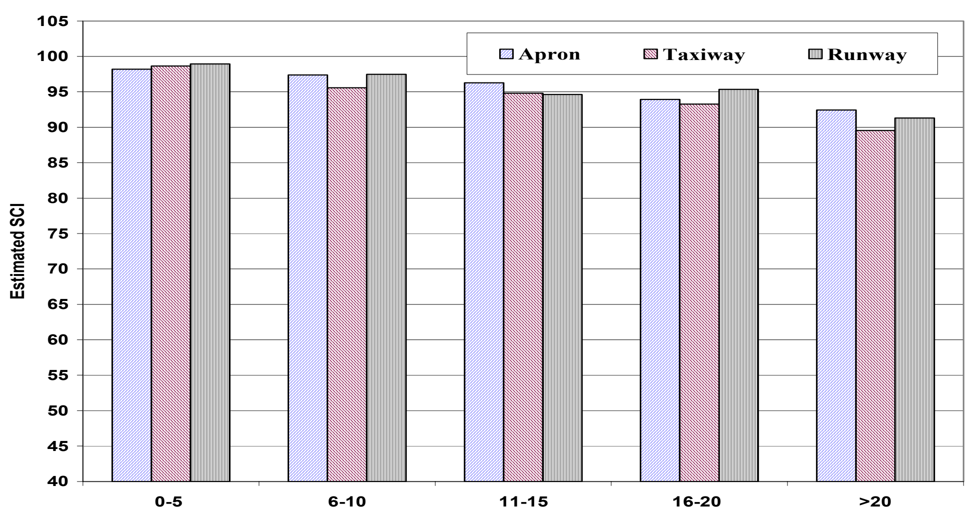


FIGURE 6-16. ESTIMATED SCI OF PCC PAVEMENTS BY AGE AND FUNCTION

The findings are summarized below.

- The average structural life of the surveyed PCC pavements are higher than 80 for the pavements in all age groups.

- Since the PCC airport pavements were designed for SCI = 80 at 20 years service, it can be concluded that the average PCC pavement structural life is significantly longer than 20 years.

- Comparison of figures 6-13 and 6-16 indicates that the differences of PCIs are higher than the differences of SCIs in each age group. The differences are higher in the older age group than in the younger age group. This comparison shows that the variations of pavement conditions are mainly caused by the non-load-related effects.

6.7 SUMMARY.

This section summarizes the findings (e.g., quotes, findings, and conclusions) extracted from published and unpublished data sources, reports and papers. Generally, PCI data is available from airfield pavement surveys. An attempt was made to separate non-load-related distresses from load-related distresses. The structural condition for PCC pavements is clearly defined by SCI. In this section, SCI was also defined for flexible pavements. It is a new concept that has not been used before but something that warrants further study and evaluation. The PCI deduction due to load-related distresses (alligator cracking and rutting) was considered for computing the SCI for flexible pavements. Figure 6-17 shows that the SCI near the pavement design life (20 years old) for both flexible and rigid pavements and for all types of pavements (runways, taxiways, and aprons) is higher than 80. Slightly more structural damage for flexible taxiways and aprons is observed compared to rigid taxiways and aprons. The flexible pavement SCI includes rutting, a portion of which was probably contributed by rutting in the surface HMA layer. This would represent material and construction defects rather than structural defects. HMA rutting is difficult to extract from the total rutting obtained from routine condition survey data. Consequently, the SCI values for flexible pavements are probably understated. Figure 6-18 shows the comparison between flexible and rigid pavement PCIs for pavements older than 20 years.

Differences between flexible and rigid SCI and PCI for pavements older than 20 years are shown in figure 6-19.

As shown in figure 6-19, the difference between flexible and rigid pavement PCIs is higher than between flexible and rigid pavement SCIs. Generally, flexible PCIs are lower than rigid PCIs. This difference indicates that factors other than structural are more prevalent in the deterioration of flexible pavements. This could argue for the review of material and construction standards and maintenance practices for flexible pavements.

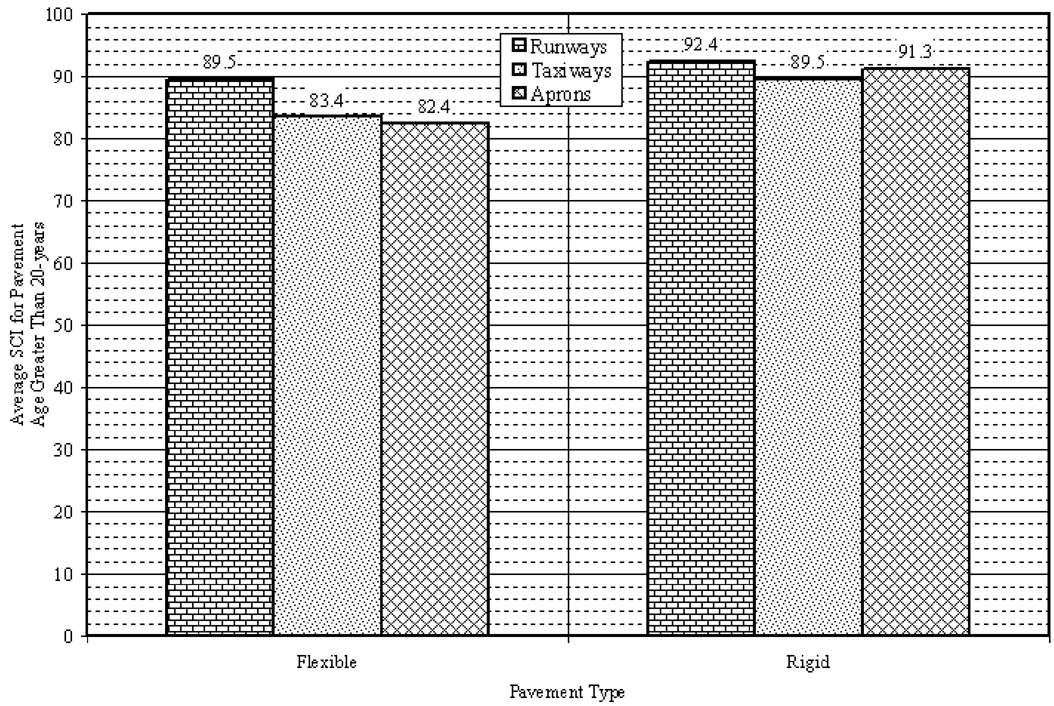


FIGURE 6-17. AVERAGE SCI FOR PAVEMENTS OLDER THAN 20 YEARS

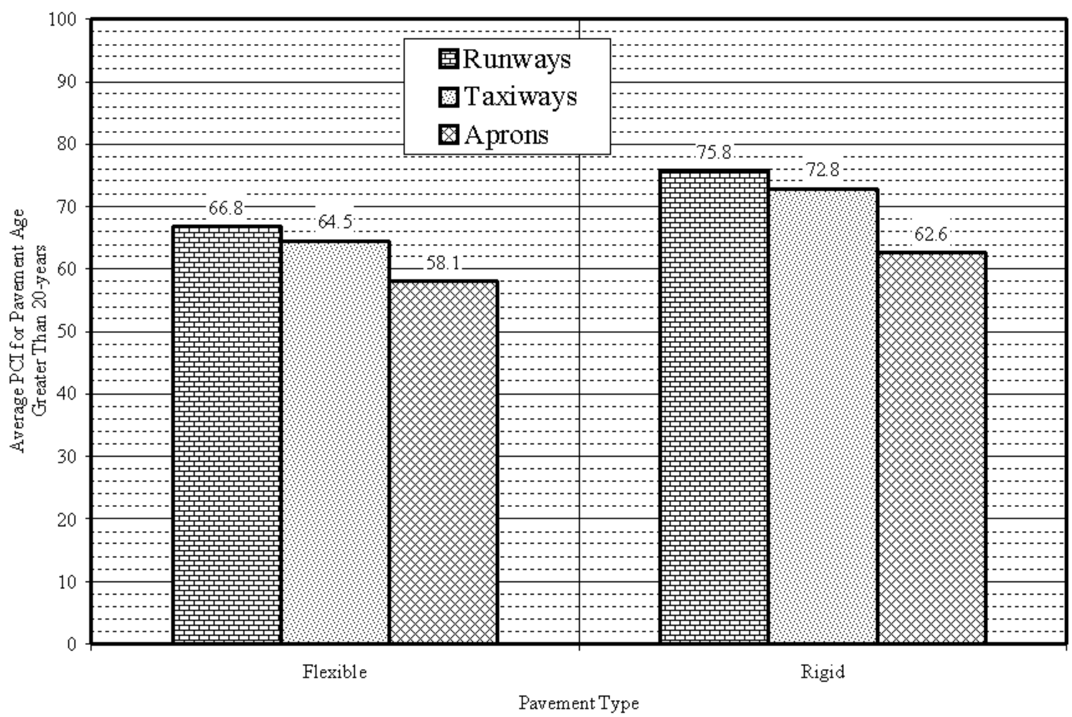


FIGURE 6-18. AVERAGE PCI FOR PAVEMENTS OLDER THAN 20 YEARS

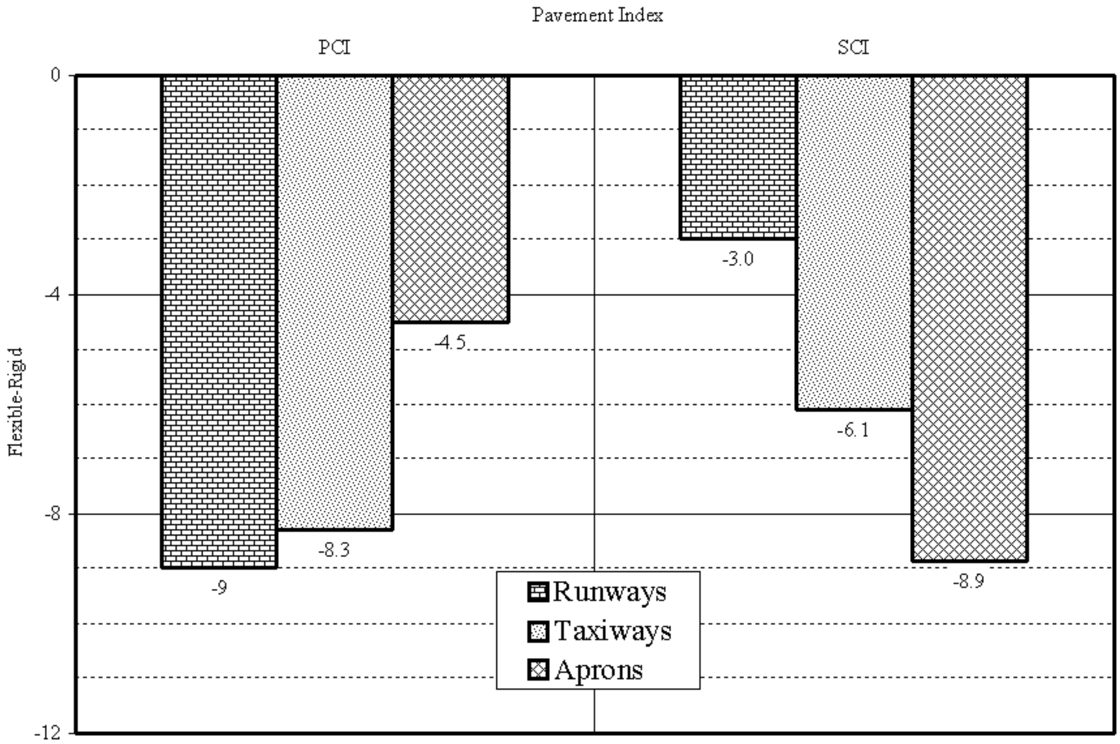


FIGURE 6-19. DIFFERENCE IN FLEXIBLE AND RIGID PCI AND SCI FOR PAVEMENTS OLDER THAN 20 YEARS

The findings of previously published reports on flexible pavement performance are quoted below:

- Kohn [46]:

  "The analysis of the data collected during this phase of the study did tend to indicate the general adequacy of the current design procedures in terms of obtaining a design thickness. Factors were discovered which indicate improvements are needed in areas other than the determination of thickness. In general, it can be said that the FAA 6C design method is adequate from the standpoint of thickness design."

- Kohn [46]:

  "The flexible pavement sections surveyed were found to be much closer to design thickness when the NDT properties were used in the design procedure. None of the sections was found to be failed using the rutting criteria. Some of the sections were failed under the cracking criteria. This fact presents a problem in the design method since the cracking encountered was alligator- or fatigue-type cracking. The current method specifies minimum thicknesses for critical pavement sections, but no method is present to check the fatigue life of the surface for a given set of conditions."

- Grogan [47]:

  As previously identified in section 6.4.3 the study to evaluate the performance of stabilized base courses concluded for cement-stabilized bases that: “The overall performance of cement-stabilized bases can be considered good. All of the pavements containing cement-stabilized bases have at minimum, performed for their design life. Some of the more heavily used pavements have required replacement after more than 20 years in service. However, these pavements have likely supported a great deal of more traffic than they were originally designed for. The good performance of these pavements even with the probable large increase in operations over original design, indicates that the pavements were over designed, that the stabilized base helps provide good performance, or that, as is known, the loads are more critical than the actual number of operations and that as long as the load magnitude designed for is accurate, the number of operations only needs to be within an order of magnitude. Usually the pavements are designed to carry the maximum gross weight of the design aircraft and most aircraft rarely if ever reach their maximum gross allowable weight during normal daily operations.” This is applicable for rigid pavement performance also.

- GAO [61]:

  The report states that: “(1) runways that were constructed at small airports with ADAP funds are deteriorating faster than necessary because airport owners are deferring critical pavement maintenance; (2) in addition to shortening a runway’s useful life, deferring maintenance generally results in serious damage to the basic runway structure, thus increasing the cost to rehabilitate the runway.”

- GAO [62]:

  “The airports in the national system have runways that are in generally good condition. Seventy-seven percent of the runways in GAO’s runway pavement database were ranked in the three highest categories – excellent, very good, and good. Eleven percent were rated fair; the remainder were rated in the three lowest categories – poor, very poor, and failed.”

The findings of previously published reports on rigid pavement performance are quoted below.

- Kohn [46]:

  “In summary, a majority of the rigid pavement sections are as thick as or thicker than the current FAA procedure would require to carry the current traffic. None of the sections was found to be in any condition close to the failure condition used in the design procedure. ………… Thus, the design procedure appears to be adequate from a thickness point of view.”

- Grogan [47]:

  “The overall results of this study indicate that Portland cement concrete pavements with cement stabilized layers are performing well and are supporting aircraft at a level of performance at or above the design level for the life of the pavements.”

- Ahlvin [6]:

  “The survey program led conclusively to the finding that rigid pavements are performing much better than design criteria would indicate.”

The current research study and the review of previously published studies indicate and demonstrate that the FAA thickness design procedures for airport pavements (which are based on the structural failure) are adequate for the 20-year design life requirements.

7. CONCLUSIONS.

The objective of this study was to evaluate whether the Federal Aviation Administration (FAA) thickness design standards for flexible and rigid pavements are consistent with the FAA's standard for 20-year pavement design life requirement. Numerical analysis using structural and failure models and the analysis of surveyed pavement data (available from published and unpublished resources) was performed to achieve the objective. Based on the analysis of the surveyed data of airport pavements and using a Structural Condition Index (SCI) value of 80 as the minimum acceptable index for structurally sound pavement, it was concluded that the FAA thickness design standards for flexible and rigid pavements are adequate and are consistent with the 20-year design life requirements, if other related standards for material preparation, construction, and maintenance are appropriately applied.

Available data from airport pavement condition surveys indicated that the average SCIs of both hot mix asphalt (HMA) and Portland cement concrete (PCC) pavements in all age groups were higher than 80. Based on the definition adopted in this report, the average pavement structural lives for runways, taxiways, and aprons were greater than 20 years. In other words, airport pavements designed following Advisory Circular (AC) 150/5320-6D have sufficient thickness to provide the required 20-year structural life.

It is necessary to differentiate the concepts and effects of the Pavement Condition Index (PCI) and SCI indexes. Sixteen and fifteen distresses are currently used to define the service condition of HMA and PCC pavements, respectively, and to calculate the values of PCI. Two pavements surveyed with the same PCI value may have very different service conditions and may need different maintenance and rehabilitation procedures for repair. Therefore, the value of PCI alone should not be used to evaluate the standards for pavement structural (or thickness) design. The structural, or load-related, SCI is also necessary to establish pavement rehabilitation requirements.

In addition to the SCI data, the PCI data for flexible and rigid pavements were also compared. Although the SCIs for flexible and rigid runways were nearly the same, the differences between the PCI values for rigid and flexible pavements were larger, as well as those for aprons and taxiways. The PCIs for flexible pavements over 20 years of age were generally lower than the corresponding PCIs for rigid pavements. This indicated that, while the structural performance of flexible and rigid pavements were comparable, a difference in functional (e.g., materials and construction) performance was noted. Therefore, while the data evaluated during the study confirmed the adequacy of the FAA's structural design procedures, the data indicated that improvements in the construction and material standards for flexible pavements should be considered.

The surface layer thickness and stiffness effects were examined. A slight increase in thickness and flexural strength of P-501 has a much larger impact on pavement life compared to the changes in P-401 thickness and stiffness.

Since the importance of each functional pavement type (runway, taxiway, and apron) is different for each airport, different criteria may be applied for their maintenance and rehabilitation. Therefore, different thresholds of PCI may be used by an airport authority for defining the end of service life of runway, taxiway, and apron pavements. These differences do not necessarily directly relate to differences in structural performance.

The development of FAA design procedures and models has kept pace with technology. The procedures have evolved to become more mechanistic in nature. The design model now considers the contribution of all layers above the subgrade for HMA pavements and beneath the surface layer for PCC pavements to separately evaluate the contribution of each layer. The new physical model is closer to the real pavement. The traffic model for both the HMA and the PCC pavements has also changed from empirical to more mechanistic. The latest model determines the pavement thickness using a cumulative damage factor concept. The damaging potential of each aircraft is computed and added using Miner's hypothesis. This concept simulates traffic loading in a more appropriate way.

The condition SCI = 80 is defined as the end of pavement structural life. The point at which the SCI values drops below 100 is defined as the appearance of the first full-depth cracks in rigid pavements.

The concept of SCI for flexible pavements was introduced in this study. Two distresses, alligator cracking and rutting, are recommended in this report to calculate SCI for HMA pavement and to evaluate the FAA design standards for HMA pavement thickness determination. The flexible pavement SCI includes rutting. Rutting may include a portion that was probably contributed by rutting in the surface HMA layer. This distress was related to material and construction defects rather than structural defects. HMA rutting was difficult to extract from the total rutting obtained from routine condition survey data. Consequently, the SCI values for flexible pavements were probably understated. SCI provides the potential to further improve airport evaluation in the future, and further study and evaluation of the concept is recommended.

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9. INDUSTRY COMMENTS.

For the preparation of this report, qualified industry experts from the Asphalt Institute and the American Concrete Pavement Association were recruited to review the work plan and the final draft. Although opportunities to review the work in progress did not become available, comments from the representatives of the respective organizations on the final draft report follow.

9.1 COMMENTS FROM THE ASPHALT INSTITUTE.

The conclusion that the FAA thickness design procedures adequately provide for a 20-year operational life appears to be valid based on the methodology in this report. While we generally concur with the conclusions, we recognize that the methodology of analysis to reach its stated conclusions, specifically, the use of structural condition index (SCI) to define failure in hot mix asphalt (HMA) pavements is a new concept that may or may not be valid. Therefore, this concept and methodology of analysis needs to be investigated further.

With regard to the general topic of FAA thickness design, the consensus throughout the HMA industry is that the FAA procedure for flexible pavements could be substantially improved in the following important ways:

a) HMA Modulus—Temperature Dependence: in the current procedure in FAA AC 150/5320-6D using LEDFAA, the modulus for P-401 surface layers is fixed at 200,000 psi and the modulus of P-401 base layers is fixed at 400,000 psi. This is a shortcoming that conflicts with the fact that HMA stiffness is dependent on temperature. All else being the same, we should expect that a P-401 mix in Miami will perform differently than a P-401 mix in Bangor simply due to the difference in climate. The current FAA method does not allow us to account for this important temperature dependency.

b) HMA Modulus—Material Properties: Another fundamental issue with the FAA design method is that by establishing fixed values for stiffness, the FAA does not take complete advantage of high quality materials available for HMA mixes, such as improved asphalt binders that provide improved high temperature stiffness to resist rutting while maintaining low temperature flexibility to resist cracking.

c) Calculation of Horizontal Strains/Stabilized Bases: The current FAA procedure sums the accumulated horizontal strains at the bottom of the P-401 surface layer. While this is the correct procedure for traditional flexible pavement designs using HMA over aggregate base, the FAA method does not adjust to account for the presence of an asphalt stabilized base P-401 layer, which is required for airport pavements that support heavy aircraft (greater than 100,000 lb.). When one is present, horizontal strains should be calculated at the bottom of the P-401 base in order to accurately analyze the effect of fatigue in deep HMA structures.

d) Full-Depth Asphalt: If the improvements listed above were to be enacted, the FAA would be in a position to evaluate full-depth asphalt pavements in which standard P-401 mixes are used exclusively above a prepared subgrade. Full-depth asphalt pavements have been in use for more than 30 years in the highway industry. Furthermore, this strategy fits well with the FAA stabilized base requirement for heavy aircraft operations, and takes it a step further by using P-401 in place of all aggregate layers. By analyzing temperature-dependency in HMA, allowing for improved HMA material properties in the design, and accurately analyzing the fatigue failure in the pavement section, the FAA would be able to assess whether full-depth P-401 pavements are a viable alternative for commercial airport applications.

e) Perpetual Pavements: The most significant new development in structural design of HMA pavements is the perpetual pavement concept. This design concept is essentially a full-depth asphalt pavement that uses non-standard HMA mixes at particular layers in the pavement structure. From the bottom up, a typical perpetual pavement structure consists of a well-prepared subgrade to provide firm foundation support, a flexible base layer to resist fatigue cracking, a rut-resistant intermediate layer, and a renewable, rut-resistant surface course that can be milled and replaced periodically. The general concept is that distresses in a perpetual pavement are driven to the surface, which can be renewed as needed, say every 15-20 years. The firm foundation support, appropriate material selection, and adequate thickness design prevent the structure from failure due to deformation at the top of the subgrade. The flexible base layer resists the accumulation of fatigue damage. The renewable surface layer allows for the management of top-down cracking. Perpetual pavements are the future of the HMA industry and the FAA should begin to study their effectiveness for airport applications.

The HMA industry concurs with the conclusion that "the data indicate that improvements in the construction and material standards for flexible pavements should be considered." The following recommendations are offered as areas of further study:

a) Superpave Mix Design: the FAA may want to evaluate the use of the Superpave mix design technology as an improved method to build rut-resistant mixes for airfield pavements.

b) Stone Matrix Asphalt (SMA): SMA has been used successfully in the construction of rut-resistant mixes for heavy highway applications and should be considered for use on commercial airports.

c) Materials Standards and Construction Practices: As is demonstrated in your report, construction and material deficiencies contribute to the overall functional performance of airfield pavements. These issues can range from the longitudinal joint construction and obtaining adequate mat density to material segregation (mechanical and thermal), moisture susceptibility and other issues that apply to the entire HMA industry, not just airports. We share the FAA's concern about construction and material quality and we would be happy to discuss ways in which the FAA and the HMA industry could approach research into these issues.

9.2 COMMENTS FROM THE AMERICAN CONCRETE PAVEMENT ASSOCIATION.

Section 705 of **Vision 100 – Century of Aviation Reauthorization Act** provides the Federal Aviation Administration (FAA) with an opportunity that is un-matched by any recent event in the history of airfield pavements. Specifically, the provision requires FAA to review the standards used to determine the appropriate thickness for airfield pavements are in accordance with the standard 20 – year-life requirement and make appropriate adjustments if the Administrator determines that such standards are not in accordance with that requirement. The industry advocates that adjustments to the standards are necessary, to advance to the state of the art.

This report does not answer the mandate of Section 705. It does, however, support industry's concern that "[G]enerally the predicted and actual pavement performance have not always compared favorably." We appreciate that the FAA now has additional and recent support that clearly shows the problem with its many pavement standards. But, we still must address the mandate of Section 705.

The discrepancy in the relationship between pavement design thickness and other related FAA standards, as related to the 20-year design life is evident, for example, in two documents.

- Advisory Circular 150/5320-6D provides for designing pavement thickness based upon the number of aircraft departures, weight of the aircraft, design strength of concrete and the value of sub-base support.
- Advisory Circular 150/5370-10 (Item P-501) requires that the constructed strength exceed the design strength.

These FAA documents provide one example and show that identification of the problem is relatively easy and direct. And as this report states "[I]ncreasing the flexural strength of P-501 from 700 to 735 psi (an increase of 5-percent) increases the pavement predicted life from 20 to 40.2 years." ("Increasing the flexural strength of P-501 from 700 to 735 psi (an increase of 5 percent) increased the pavement's predicted life from 20 to 40.2 years" after editing.)

Determining a solution to the interrelated FAA standards problem is a more complex matter, but we believe answers can be obtained through proper research and consultation with the industry.

This report provides three substantiating reasons why the FAA design, materials and construction standards must be reviewed and adjustments made.

1. This report substantiates that the FAA definition of pavement failure is unreasonable. However, the introduction of an arbitrary standard, as this report suggests, does not solve the problem. We reference the following to illustrate the conflicts in the draft report on this issue.

    This report presents the failure criteria for pavement in terms of several different variables and takes a position "[W]hen the pavement is incapable of performing the task it was designed for, it has failed." Subsequently, it states however that "[A] unique definition of pavement failure does not exist." The report then proceeds to create a new definition of failure for AC pavements based upon the Structural Condition Index (SCI). The stated relationship between SCI and pavement failure is "an SCI of 80 is consistent with the current FAA definition of initial failure of a rigid pavement" (50% of slabs in traffic area exhibit initial cracking). This definition is the basic problem inherent in the FAA standards. If 50% of the slabs in a pavement feature have initial cracking (low intensity cracking) then the correlation between SCI and failure definition is consistent. Beyond that a correlation is not expected. The SCI is a design variable used for determining the thickness for rigid pavement overlay calculations. Additionally, and more important, creating an SCI for asphalt pavements goes beyond current practice. I am not aware of any situation or practice where SCI is associated with flexible pavement performance.

2. This report concludes that "airport pavements designed following AC 150/5320 6C and 6D have sufficient thickness to provide the required 20-year structural life." ("airport pavements designed following Advisory Circular (AC) 150/5320-6D have sufficient thickness to provide the required 20-year structural life" after editing.) This statement, however, does not address the intent of the question posed by Section 705 and it conflicts with another statement in the report. Specifically, the report "confirm(s) the adequacy of the FAA structural design procedures" ("confirm(s) the adequacy of the FAA structural design procedures" after editing) but then states that "the data indicate that improvements in the construction and material standards (not the design standards) for flexible pavements should be considered" ("the data indicated that improvements in the construction and material standards for flexible pavements should be considered" after editing).

3. This report identifies that possibly the FAA materials and/or construction standards could be improved, but does not define what adjustments to the standards should be considered. This report implies that the standards for concrete pavement may be conservative but could not quantify this issue. The draft report also clearly indicates a need to improve standards for flexible pavements. There is a strong recommendation for a definition of pavement failure consistent with field performance.

While each of these three major points in this report shows that possibly some of the FAA standards should be adjusted, it does not recommend doing the necessary research to accomplish the tasks that are necessary to answer the questions raised. And, the desktop computer model assessment used to support the report does not close the widening gap between "predicted and actual pavement performance." Additional specific comments are provided as separate correspondence.

We present the following research plan for concrete pavement that the FAA should adopt as a roadmap to answer the questions that the report raises. With this research, we can obtain a valid and technically correct answer to the Section 705 mandate.

The following research projects are required. The most up-to-date information on the life of airfield pavements is not adequate to answer the outstanding questions related to design assumptions and pavement performance. These projects will be accomplished and the results used to (1) develop a reasonable definition of pavement failure, and (2) examine the variables inherent in the design assumptions that have the most significant influence on pavement life.

Project 1 – Define Pavement Failure in Terms of Performance. The project will involve doing a survey of domestic service airports in each of the four levels of size and making a determination about the condition of rigid airfield pavement when owner decisions are made regarding replacement. Estimated time to complete is 18 months. Estimated cost is $450,000.

Project 2 – Evaluate the Range of the Concrete Modulus (E_c) for Typical Concrete Placements and Impact on Pavement Performance. Typically, the value of the concrete modulus is assumed to be 4,000,000 psi. However, higher strength concretes have higher values of modulus. Investigate the variability and determine the impact on pavement performance. Estimated time to complete is 24 months. Estimated cost is $250,000.

Project 3 – Investigate the Load Transfer Efficiency of Joints on Both Aggregate and Stabilized Subbase. Load transfer efficiency is dependent upon ambient environment and subbase support. Stabilized subbase is required for those airfield pavements that are used to support airplanes weighing in excess of 100,000 pounds. Accomplish studies that determine the joint load transfer efficiencies for both subbase types on different subgrades. Investigate the impact of load transfer efficiency on pavement performance. Estimated time to complete is 18-months. Estimated cost is $800,000.

Project 4 – Investigate Subbase Restraint on the Performance of Concrete Pavements. Mechanistic design procedures have allowed for the modeling of the different layers in a pavement system and the interaction of those layers in supporting loads. Assumptions have been made about the amount of friction developed between the subbase materials and the concrete for rigid pavements. A sensitivity analysis suggests that the character of the friction can impact pavement performance. The impact has not been characterized. Estimated time to complete is 12 months. Estimated cost is $250,000.

Project 5 – Evaluate the Accuracy of Predicted Pavement Stress versus Actual Pavement Stress. Pavement models assume that stress concentrations occur at various locations within a pavement structure depending upon the location and distribution of loads. Accomplish full scale testing that will validate the assumptions related to Cumulative Damage Factor (CDF) and the accuracy of models. Estimated time to complete is 18-months. Estimated cost is $900,000.

9.3 COMMENTS FROM THE NATIONAL ASPHALT PAVEMENT ASSOCIATION.

Given the restraints of the study, the National Asphalt Pavement Association (NAPA) believes the FAA did a good job putting the report together. The Asphalt Institute responses are constructive and reflect our views. In addition, NAPA has the following two items for consideration:

a) Any change to structural design procedures or assumptions regarding pavement structural life needs to be based on a study that is specifically designed to address the issue. In other words, it is acceptable to review the structural performance of pavements based upon existing pavements for exploratory purposes, but substantive changes will require more careful investigation and documentation.

b) Section 5 of the report seemed to be based entirely upon an analytical study of the effects of changes in input to changes in performance. This requires experimental validation before it is put forward. If one changes the thickness or modulus of HMA or PCC, the resulting change in pavement response will be more or less dramatic, depending upon the "baseline" values that are assumed.