DEVELOPMENTS IN THE CORPS OF ENGINEERS RIGID AIRFIELD DESIGN PROCEDURES

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ABSTRACT

In 1979 the U.S. Army Corps of Engineers introduced an airfield design manual that included design procedures for plain, reinforced, continuously reinforced, steel fiber reinforced, and prestressed concrete airfield pavements. In the intervening ten years additional work has led to changes in some of these procedures and to preparation of two updated design manuals published in 1988. This paper reviews the basis for these changes. Some of the topics covered include revised fatigue relationships for plain and steel fiber reinforced concrete pavements, acceptance of layered elastic design procedures as an alternative to Westergaard procedures, changes to thickness design for continuously reinforced concrete pavements, and revisions to the jointing criteria for steel fiber reinforced concrete pavements. Some work has not progressed to the point to allow its inclusion in the 1988 manuals and the current status of changes to prestressed and overlay pavement design are presented.

INTRODUCTION

In 1979 the U.S. Army Corps of Engineers approved a rigid airfield design manual (1) that included design methods for plain, reinforced, continuously reinforced, steel fiber reinforced, and prestressed concrete airfield pavements. This was the first time that complete design methods for all of these pavement types were presented together. The basis for these procedures was described at the Second International Conference on Concrete Pavement Design in 1981.(2) In the 10 years since the 1979 manual was published additional work has led to further changes in Corps of Engineer rigid airfield pavement design practice. A revised airfield design manual (3) was published in 1988, and it is an updated version of the 1979 manual. Also in 1988 the Corps of Engineers produced a design manual (4) based on the layered elastic analytical model and allows its use as an alternate design method to the revised 1988 manual.(3) This paper will review the basis for these changes in Corps of Engineers design practice and will discuss current work that may lead to further changes.

LOADS AND TRAFFIC

The Corps of Engineers designs on the basis of the static load of departing aircraft. Since 1955 a 15 percent increase in load was used with bicycle geared aircraft such as the B-52. This factor has been variously described as an impact factor, an aircraft growth factor, or simply a safety factor. Studies (2,5) of aircraft loads on pavements have repeatedly shown that static aircraft loads are the most severe loading; consequently, this bicycle gear impact factor has been removed from the new Corps of Engineers pavement design manuals.(3,4)

Aircraft traffic does not follow an identical trace on the pavement with each pass but rather wanders about the centerline in an approximately normal distribution. Based on field observations of aircraft wander, the Corps of Engineers converts passes of an aircraft to coverages using the method and assumptions described in Reference 6. Coverages on a rigid pavement are equivalent to the maximum accumulation of tensile stress repetitions at any one point on the pavement and reflect aircraft wander and gear configuration.

Previously, the Corps of Engineers designed military airfields on the basis of the dominant aircraft using the field (e.g. light load airfields were designed for F-4 aircraft; heavy load airfields were designed for the B-52; etc). However, in 1983 the Air Force Civil Engineering Center at Tyndall AFB, FL. reevaluated the actual traffic patterns at Air Force bases and selected a mixed traffic of various sized aircraft for design of Air Force light, medium, modified heavy, and heavy load airfields. These new mixed traffic requirements are incorporated in the new manuals.(3,4)

ANALYTICAL MODELS

The Corps of Engineers has long used the Westergaard edge loaded model (7) as the basis for its rigid pavement designs, (8) and this continues to be the basis for the revised 1988 design manual.(3) The Westergaard edge loaded model represents the concrete slab as an elastic plate with a modulus of elasticity and Poisson's ratio resting on a bed of springs characterized with a modulus of subgrade reaction k. This k value was referred to by Dr. Westergaard (7) as "an empirical makeshift, which however has been found to give usable results." The model calculates the tensile stresses caused by a load adjacent to a slab's free edge if the slab is infinite in the other three horizontal directions. Ioannides, Thompson, and Barenberg (9) present a detailed description and analysis of the origin and forms of Dr. Westergaard's slab solutions including edge, interior, and corner loading.

The Pickett and Ray (10) influence chart solutions to the Westergaard equations were a convenient method for engineering solutions and were used by the Corps of Engineers for developing design charts. A computerized solution (11) to these influence charts greatly simplified preparation of the airfield design manual charts and was used for the 1979 manual. A version of this program is also available now for microcomputers.

A regression equation to calculate the Westergaard free edge stresses was developed by Witczak, Uzan, and Johnson.(12) This was later modified slightly by Dr. Walter Barker at the U.S. Army Engineer Waterways Experiment Station and is

$$\sigma_e = \frac{P}{h^2}\left[a_0 + a_1 \ln \ell + a_2 (\ln \ell)^2\right]$$

where

σ_e = Westergaard free edge tensile stress (lb/in.^{2})

P = actual gear load, lb

h = slab thickness, in.

a_0, a_1, a_2 = regression constant from Table 1, calculated for a specific gear to match the computerized solutions of Pickett and Ray influence chart solutions

ℓ = radius of relative stiffness, in.

$$= \sqrt[4]{\frac{Eh^3}{12\,(1 - \upsilon^2)\,k}}$$

E = modulus of elasticity of the slab, lb/in.^{2}

υ = Poisson's ratio of the slab

k = modulus of subgrade reaction, lb/in.^{2}/in.

This regression equation provides a very rapid method of calculating the Westergaard edge stresses under a specific aircraft. Because of the speed of solution, this equation was used as the basis for a family of user-friendly, rigid pavement microcomputer design programs developed under the supervision of Dr. Walter Barker at the U.S. Army Engineer Waterways Experiment Station for use by Corps of Engineer and Air Force designers. These programs were used to develop the revised 1988 manual's design charts.(3)

While the Westergaard free edge analytical model continues to be the basis for the updated Corps of Engineers rigid airfield design manual (3), there are some recognized limitations with this model. The use of a single value k to represent all material under the slab cannot model the layered structure that actually exists. Some effective k on the surface of the base must be estimated on the basis of the k of the subgrade and quality and thickness of the base. Several methods have been published to accomplish this, but all suffer from theoretical limitations.

The revised 1988 design manual (3) continues to use a modification of an empirical overlay equation to try to account for the effect of high quality stabilized bases. This approach was originally introduced in the 1979 manual (1) and was described previously.(2) A comparison of the modified overlay and effective k methods for design of rigid pavements on stabilized bases found that both approaches could give conservative or unconservative results depending on the specific problem analyzed.(13)

Other analytical models offer potential advantages over the Westergaard edge load model, and these are discussed in more detail elsewhere.(14) One such model is the layered elastic analytical model. It can be solved on current generations of microcomputers, and this model is also used with current Corps of Engineers pavement evaluation methods using the falling weight deflectometer. This model represents the pavement as a series of continuous, horizontally uniform, homogeneous, isotropic layers; each characterized with a modulus of elasticity, Poisson's ratio, and either full slip, no slip, or some intermediate level of slip between each layer. Other analytical models such as finite element methods generally require a main frame computer system and are therefore less available and practical for conventional engineering design practice. The Corps of Engineers has published design methods using the layered elastic model, (4) and this design procedure is now accepted as an alternate to the Westergaard based model.(3)

FATIGUE RELATIONS

Pavement performance is generally related to the calculated stresses by means of a concrete fatigue relationship. Several fatigue relations developed from laboratory beam flexural fatigue tests are shown in Figure 1. In general, the fatigue performance of concrete is considered to be the same in compression, tension, and flexure, and the relation between the ratio of stress to strength is approximately linear with the logarithms of cycles. There is often considerable scatter in concrete fatigue data so probability of failure are sometimes also included as shown in Figure 1.

The ratio between the minimum (or "at rest" stress) and maximum stress (or fully loaded

Table 1. Corps of Engineers Pass to Coverage Ratios and Regression Constants

Aircraft	Typical Main Gear Load lb	Pass to Coverage Ratio		Regression Constants		
		Channelized*	Nonchannelized**	a_0	a_1	a_2
B-52	249,600	1.63	2.00	1.1249	-0.8449	0.1966
B-727	82,250	3.25	6.00	-0.8081	0.1698	0.1165
B-747	187,200	3.48	5.22	1.4096	-1.1067	0.2387
E-3/B-707	157,450	3.25	6.00	1.5490	-1.1773	0.2530
C-5A	361,900	1.62	2.20	1.1904	-0.7615	0.1351
C-130	78,750	4.18	8.10	0.5560	-0.6011	0.1936
C-141	162,150	3.44	6.34	1.4990	-1.1877	0.2634
A-10	22,500	7.82	15.52	-4.0850	2.4409	-0.1650
F-4	27,000	8.58	17.00	-4.0251	2.4894	-0.1775
F-14	34,500	7.14	14.29	-2.4290	1.4950	-0.0413
F-15 C/D	30,600	9.37	14.10	-4.3900	2.9452	-0.2649
F-111	54,000	4.92	9.80	-2.7582	1.4794	0.0322
KC-10-30	224,200	3.42	5.45	2.6351	-1.7184	0.3078
P-3	63,900	3.57	6.67	-1.1908	0.4615	0.0758
T-43/B-737	70,500	3.68	6.91	-1.1567	0.4012	0.0910
Shuttle	119,250	3.60	6.49	-1.0736	0.3180	-0.0986
Concorde	183,600	3.18	5.90	2.1465	-1.5308	0.3074
18 Kip axle	18,000	2.64	2.64	0.2440	-0.2172	0.1263

* 70 in. wander (e.g. taxiways or ends of runways).
** 140 in. wander (e.g. parking areas, runway interiors).

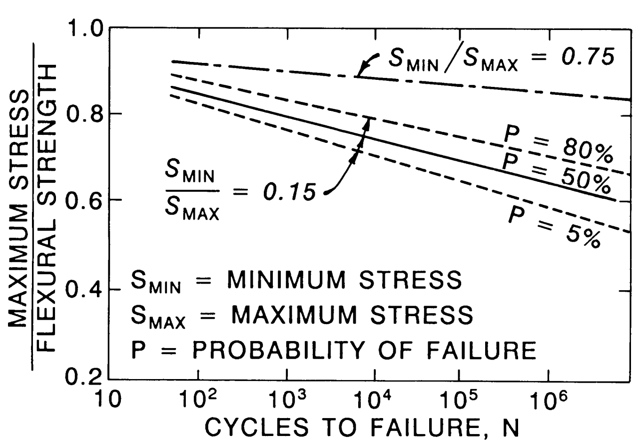

Figure 1. Representative Concrete Flexural Beam Fatigue Relationships (Reference 15)

stress) applied to the concrete during fatigue testing has a major impact on the final fatigue relation. As can be seen in Figure 1, the fatigue capacity of concrete decreases dramatically as this ratio decreases. A concrete pavement in the field is always under some fluctuating level of stress due to moisture and temperature gradients. According to the data presented by Domenichini and Marchionna (16) the ratio of minimum temperature stress to the sum of temperature and load stresses for AASHO road test slabs 6.5 to 9.5 in. thick varied from 0.16 to 0.60 depending on the time of day and season of the year. This implies that the appropriate fatigue relation for concrete pavement design is constantly varying with temperature.

The Corps of Engineers has relied on full scale field tests rather than beam tests to relate the calculated stress and strength to concrete pavement performance. Use of such a "field" fatigue relationship includes, to some extent, the effects of thermal and moisture gradients, nonuniform support, complex real states of stress, and actual joint conditions. Up through the 1979 manual (1) the Corps of Engineers used a bilinear fatigue relation rather than a straightline relation. The origin of this form was reviewed previously.(2) A reanalysis of the original Corps of Engineers test data found that for a failure criterion of one-half of the trafficked slabs having one or more structural cracks, a field fatigue relation for use with the Westergaard edge loaded model could be expressed as

$$DF_w = \frac{R}{(1 - \alpha)\ \sigma_w} = 0.50 + 0.25 \text{ Log } C$$

where

DF_w = Westergaard design factor

R = flexural strength of concrete measured by third point flexural beam test

α = load transfer across joints, taken as 0.25 for conventional Corps of Engineers joint construction standards

σ_w = maximum tensile stress calculated with the Westergaard edge model

C = coverages of traffic

This new fatigue relation was used for the 1988 manual (3) and generally requires a somewhat thicker pavement than the previous relation, particularly for high coverage levels.

The Westergaard edge load model and the layered elastic model calculate different numerical values for the stress. Since the uniform layer assumptions of the layered elastic model fail to account for the higher stresses at slab edges, the layered elastic model calculates a lower stress value than does the Westergaard. Consequently, the above fatigue relation cannot be used with the layered elastic model. Parker et al. (17) reanalyzed the original Corps of Engineer field test data, and developed the straight line fatigue relation for the layered elastic model as

$$DF_{EL} = \frac{R}{\sigma_{EL}} = 0.59 + 0.35 \text{ Log } C$$

where

DF_{EL} = layered elastic model design factor

R = flexural strength of concrete measured by third point flexural beam test

σ_{EL} = maximum tensile stress calculated using the layered elastic model

C = coverages of traffic

This layered elastic fatigue model inherently assumes that the joint construction provides the same load transfer across the joints as standard Corps of Engineers joint construction. In cases such as highways where loads traffic near a true free edge or in roller compacted concrete where joint construction fails to achieve levels similar to conventional Corps of Engineer practice, the above fatigue model is not valid. Reference 18 discusses design of roller compacted concrete with no load transfer in more detail.

Figure 2 shows several concrete fatigue relations plotted with the two Corps of Engineer relationships. The ACI relations for 5 and 50 percent probability of failure at a S_{min}/S_{max} ratio of 0.15 from Figure 1 are replotted in Figure 2. The Portland Cement Association relation is based on laboratory beam tests. In comparison with the ACI curves it provides a conservative limit for a low S_{min}/S_{max} ratio. Only a few of the Corps of Engineers tests could be conducted to 10,000 coverages or higher because of the high cost of testing. The extrapolation of these results to higher coverage levels is supported by the linear relation between design factor (or its inverse the stress ratio) and logarithms of coverages or cycles in beam tests.

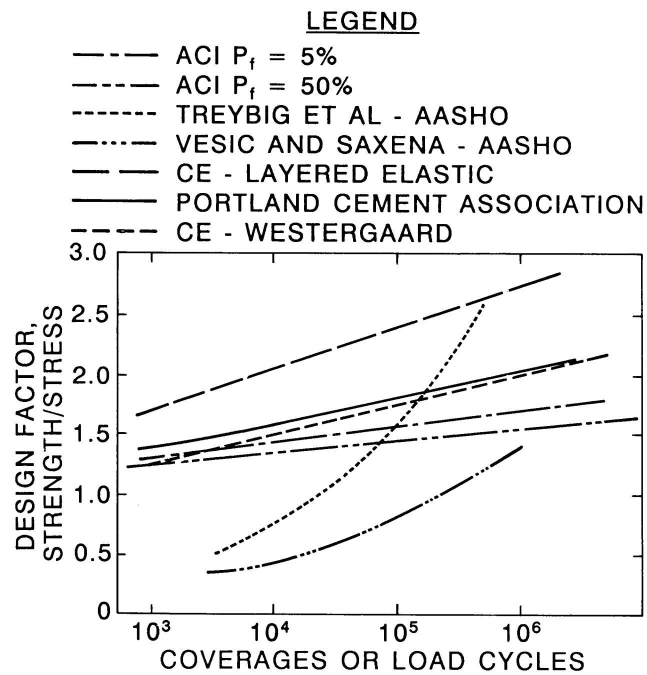

Figure 2. Comparison of Concrete Pavement Fatigue Design Relationship (Reference 14)

The relationships developed by Treybig et al. (19) and Vesic and Saxena (20) from the AASHO Road Test results are quite different from the Corps of Engineers relations, the beam tests, and one another. Part of this difference is due to the difference in failure criteria used. Vesic and Saxena (20) used a PSI rating of 2.5; Treybig et al. (19) used the development of "Type 3 cracking" defined in terms of crack opening or spalling to a width of 1/4-in. along at least half of the crack length; and the Corps used one-half of the trafficked slabs developing one or more structural cracks (PSI of about 3.0 to 3.3). Treybig et al. (19) and Vesic and Saxena (20) used different analytical models from one another so they, like the Corps of Engineers Westergaard and layered elastic approaches, end up with separate relationships from the same field data.

The most distinct difference between the AASHO test relations and the others is their curved rather than linear shape. AASHO rigid pavement test sections failed due to extensive pumping. Pumping is a function of soil type, moisture, and load and not of tensile stresses in the concrete. Consequently, the AASHO based relations include effects other than concrete fatigue. The Corps of Engineers requires that pumping protection be designed as a specific item separate and independent of the thickness design to protect against tensile stress fatigue.

The Corps of Engineers allows designers to use either the Westergaard edge loading model (3) or the layered elastic model.(4) The calculated stress values using either model are only nominal values reflecting the interaction of load, idealized material properties, and pavement thickness. The actual stress values in the field are highly variable due to factors such as specific load location on a slab, variations in material and support properties, changes in concrete modulus due to moisture changes or fatigue effects, stresses from constantly changing moisture and temperature gradients, changes in load transfer at joints due to temperature changes or fatigue, etc. The Corps of Engineers fatigue criteria for each model relate these nominal calculated stresses to performance using full scale accelerated load tests. Extrapolation of these relations beyond the actual coverage levels tested is supported by the approximately linear form of concrete fatigue relations in laboratory tests up to 10 (10) repetitions of load.

PLAIN UNREINFORCED CONCRETE

Plain unreinforced concrete with short joint spacing (12.5 to 25 ft depending on slab thickness) has historically been the most economical and dependable rigid airfield pavement for the Corps of Engineers. Special reinforcement is useful primarily for special problems, thickness limitations, unusual economic situations, etc. The analytical models and fatigue criteria discussed in the previous sections are used to select a thickness to resist fatigue of the applied loads. The changes from the 1979 manual (1) are only the revised fatigue criteria and acceptance of the layered elastic model as an alternative to the Westergaard free edge model. For standard design problems the Westergaard solutions are simpler and faster so it will probably remain the most common in use. However, the layered elastic model has been effective for analyzing some complex rehabilitation projects. Its immediate use will probably remain limited to less standard problems with complex layered structures and material properties. In the future it will replace the Westergaard based design methods for Corps of Engineers pavement design.

The Westergaard stress solutions are not very sensitive to the k value used. However, Corps of Engineers field evaluations of in service airfield pavements during the 1940's and 1950's noted that higher strength subgrades (higher k value) were providing better functional performance than low strength subgrades. After initial pavement cracking on low strength subgrades, additional cracking progressed rapidly with spalling and raveling also developing along the cracks. On high strength subgrades deterioration after initial cracking (failure criterion used in Corps of Engineers testing and fatigue relations) was notably slower, and the pavement's functional life was therefore longer. To take advantage of this better post-cracking behavior, pavements built on subgrades with a k above 200 lb/in.^{2}/in. and designed with the Westergaard model are allowed to be thinner than required by the fatigue criteria. Allowable reductions used by the Corps of Engineers were reported previously.(2) More experience with the layered elastic design method is needed to see if these reductions or some modification of these reductions are appropriate with this model.

When using the Westergaard model, the Corps of Engineers allows a 25 percent reduction in the calculated free edge stress to account for load transfer if standard Corps of Engineers joint design and construction standards are used. This allowance for load transfer is based on results of model tests, full scale traffic tests, and measurements of in service pavements, and its use over the last 40 years with other Corps of Engineers criteria has given generally good performance.(21) Table 2 shows the results of some of the Corps of Engineers tests from full scale test sections and in-service pavements. Load transfer measurements will always vary due to factors such as construction standards or temperature changes causing the joint width to vary. Also load transfer deteriorates as load repetitions

Table 2. Representative Corps of Engineers Load Transfer Measurements for Full Scale Test Section and In-Service Pavements

Type of Joint	Number of Data Points	Load Transfer Range	Load Transfer Mean	Coefficient of Variation, %
Doweled Construction Joint	195	0.0-50.0	30.6	38.0
Doweled Expansion Joint	15	15.4-42.6	30.5	24.4
Contraction Joint with Aggregate Interlock	46	15.6-50.0	37.2	19.2
Tied Contraction	6	23.9-34.8	29.2	13.4
Doweled Contraction	4	28.2-42.8	35.1	17.3
Keyed Joint	61	5.6-49.0	25.4	41.4
Tied Key Joint	2	25.6-26.1	25.8	--
Lockbourne "Free" (butt) Joint	8	5.8-24.5	15.5	40.9

accumulate. Consequently, load transfer is a variable and not a constant. The doweled joints and the contraction joint with aggregate interlock in Table 2 have mean load transfer values well above the 25 percent design value; however, the keyed joint mean value barely meets the standard design value. Keyed joints do cause performance problems, (22) but they are economical joints for some applications. Because of their potential problems keyed joints are not allowed on pavements under 9 in. thick, under channelized traffic of heavy cargo or bomber aircraft, or over soft soils.

CONTINUOUSLY REINFORCED CONCRETE

Continuously reinforced concrete pavement has no joints. Instead continuous steel reinforcement at levels of 0.50 percent or more is used to keep all cracks that form from shrinkage of the concrete or environmental effects tightly closed. Ideally cracks form at 2 to 12 ft spacings, and provide a smooth, low maintenance surface. The design procedure used in the 1979 manual and described previously by Rollings (2) allowed some load induced cracking to occur on the assumption that the steel provided to keep cracks closed would also keep the load induced cracks closed and functional. This allowed a thinner continuously reinforced concrete pavement than plain, unreinforced concrete pavement.

Experience with continuously reinforced pavements for airfields is not extensive. Some early failures of continuously reinforced concrete pavement on airports were due to insufficient slab thickness, and thickness reductions for continuously reinforced concrete airport pavements probably are not justified with our current knowledge. This suggests that the 1979 manual's (1) approach to design is inadvisable, and the 1988 manual (3) requires continuously reinforced concrete to be the same thickness as plain concrete. The continuously reinforced pavement cost must be justified on the basis of smoother ride, savings on joint construction cost, and reduced joint maintenance cost.

CONVENTIONAL REINFORCED CONCRETE

Full scale accelerated traffic tests showed that conventional reinforcing did not change the coverage level at which cracking occurred in the pavement under traffic. However, in a manner somewhat analogous to the high strength subgrade effects discussed in the previous section, the functional performance after cracking improved. Additional cracking was slowed and spalling, and raveling of cracks progressed more slowly than with plain unreinforced concrete. An empirical, varying reduction in thickness is allowed for reinforcement between 0.10 and 0.50 percent. This reduction is used with both Westergaard and layered elastic models. The specific adjustments for different steel reinforcement ratios was given previously.(2) The original development of these concepts was presented by Philippe.(23)

STEEL FIBER REINFORCED CONCRETE

The addition of steel fibers to concrete can increase the flexural strength by 35 to 70 percent. In addition to increasing the strength to initial cracking, the steel fibers bridge across the crack and provide additional load carrying capacity after initial cracking. The 1979 manual used concepts developed by Parker (24) that took advantage of both the higher flexural strength and the additional load carrying capacity after initial cracking. These designs are appreciably thinner than conventional unreinforced pavements.

At the time of Parker's development work the only airfield application of steel fiber reinforced concrete was a small trial overlay at Tampa.(25) In 1983 the Corps of Engineers and the Federal Aviation Administration jointly sponsored a study to review the performance of steel fiber reinforced airfield pavements and to review design requirements for these pavements.(26)

The pavements inspected for this study are summarized in Table 3. With only one exception, slab curling and corner cracking were recurring problems at these sites. The curl can, in the extreme, be as large as 5/8-in. This curl occurs early in the life of the pavement and is permanent. At Denver curling was observed within one week of construction. When traffic is applied to slabs with upward curled corners, corner cracking develops rapidly (Figure 3).

Volume change characteristics of steel fiber reinforced concrete have not been adequately studied, but it is clear that differential shrinkage in the top and bottom of the slabs is the cause of the curling. The source of the differential curling is open to debate. Rollings (26) concluded from his study that autogenous shrinkage due to hydration resisted by frictional forces on the base of the slab was the most likely cause of the curl. The high cementitious content, small aggregate size, and low proportion of aggregate volume in the steel fiber reinforced concretes compared to conventional paving concrete probably led to higher autogenous shrinkage than normal. Other potential explanations such as temperature gradient, high heat of hydration coupled with surface cooling, moisture gradient, or carbonization fail to explain the early age of curling and the permanent nature of the curl encountered in the field.

As can be seen in Figure 4 the differential shrinkage between the top and bottom of a slab that will cause a 1/4-in. curl is a function of the slab length and thickness. As the slab length increases or the thickness decreases, the amount of differential shrinkage needed to cause curling decreases. Table 4 compares the differential shrinkage rates to cause a 1/4-in. curl for the allowable joint spacing in the 1979 design manual (1) with the differential shrinkage rates calculated for the pavements in Table 3. The in-service steel fiber reinforced pavements were encountering curling problems in the field at calculated differential shrinkage of 156×10^{-6} in./in. or less. The steel fiber joint spacings in the 1979 manual could have differential shrinkage rates as low as 8×10^{-6} in./in.,

Table 3. Selected Characteristics of Inspected Steel Fiber Reinforced Concrete Pavements (Reference 26)

Project	Date Constructed	Cement/ Fly Ash lb/yd^3	Steel Fiber, lb/yd^3	Underlying Layer	Bond to Underlying Layer	Slab Size Thickness ft × ft × in.	Severity of Corner Cracking
Norfolk apron	1977 to 1982	600/200	160 85	Asphalt	Unbonded	25 × 25 × 5	Severe
Denver apron	1981	575/210	83	Fabric	Unbonded	25 × 30 × 7 or 8	Severe
Las Vegas apron	1976 1979	600/250 650/252	160 85	Asphalt	Unbonded	25 × 50 × 6 25 × 50 × 8	Moderate
Fallon apron	1980 1981	788/0 766/0	82 81	Asphalt	Unbonded	25 × 40 × 5	Moderate
Reno apron	1975	658/216	200	Concrete	Fully bonded	25 × 20 × 4	Moderate
Salt Lake City apron	1980	583/203	83	Asphalt	Unbonded	25 × 30 × 7 or 8	Minor
	1981	620/215	85				
Tampa taxiway	1972	517/225	225	Concrete	Partially bond	25 × 75 × 4 25 × 175 × 6	None

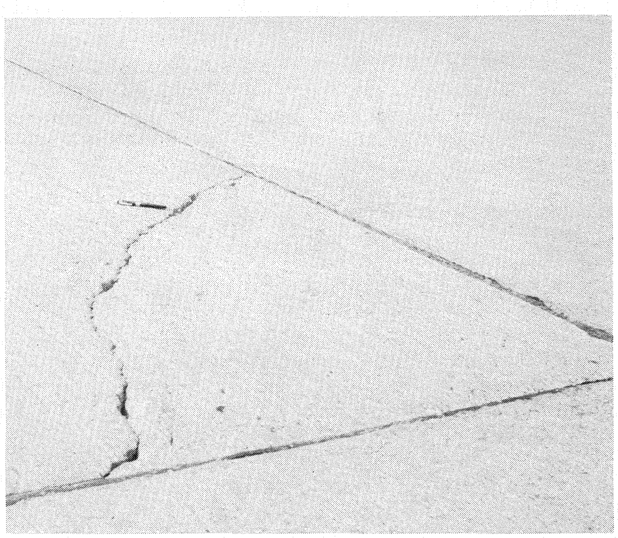

Figure 3. Corner cracking in a Steel Fiber Reinforced Concrete Pavement

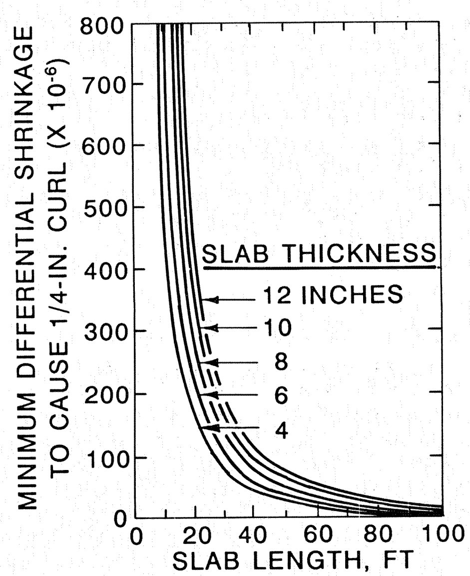

Figure 4. Effect of Slab Length and Thickness on Curling (Reference 26)

Table 4. Differential Shrinkage Rates to Cause Curl in Different Sized Slabs

Source	Concrete Type	Thickness in.	Slab Length ft	Calculated Differential Shrinkage $\times 10^{-6}$ in./in.*
1979 Design Manual	Plain	≧6, <9**	15	370
	Plain	≧9, <12**	20	313
	Plain	≧12	25	267
	Steel fiber	≧4, <6**	50	22
	Steel fiber	≧6	100	8
Norfolk	Steel fiber	5	25†	111
Denver	Steel fiber	7	25†	156
Las Vegas	Steel fiber	6	25†	133
Fallon	Steel fiber	5	25†	111
Reno	Steel fiber	4	20†	89
Salt Lake City	Steel fiber	7	25†	156
Tampa	Steel fiber	4	25†	89

* Differential shrinkage to cause 1/4-in. curl. Calculation method given in Reference 26.
** Smallest thickness used to calculate differential shrinkage.
† Shortest slab length used to calculate differential shrinkage.

well within range where curling trouble was encountered in the field. The joint spacing for plain concrete keeps the differential shrinkage at 267×10^{-6} in./in. or larger and has not encountered curling problems in the field. It is clear from this that the 1979 design manual (1) allowable joint spacing for steel fiber reinforced concrete in Table 4 are overly optimistic and would lead to curling problems. Therefore the 1988 design manual (3) requires steel fiber reinforced concrete pavement to have the same joint spacing as plain concrete until further research can establish better limits.

Parker (24) developed his thickness design fatigue criterion for steel fiber reinforced pavement which defined failure to be the opening of a crack leading to failure or debonding of the fibers bridging the crack. Some questions were raised concerning the potential for corrosion of the steel fibers and the long term validity of this concept. However, an American Concrete Institute state-of-the-art report (27) finds that fiber corrosion is limited to surface fibers even when the steel fiber reinforced concrete is exposed to deicing salts or seawater. Testing by Morse and Williamson (28) found that as long as a crack in steel fiber reinforced concrete did not exceed 0.01 in., corrosion of the fibers bridging the crack did not occur. At wider crack openings fiber corrosion at the crack and consequent strength loss is rapid. This work supports Parker's criterion that tight hairline cracks are not failure of the pavement, but rather opening of these cracks is the proper failure concept.

Parker (24) established his steel fiber reinforced fatigue criterion in terms of percent standard thickness. Rollings (26) used Parker's original failure concept to develop new fatigue relationships for the Westergaard and layered elastic models using design factor rather than percent standard thickness. Only six accelerated traffic test results were available for steel fiber reinforced concrete pavements. Therefore it was assumed that the fatigue relation for steel fiber reinforced concrete should be parallel to that of conventional plain concrete. The six test sections were used to locate the fatigue relation as shown in Figures 5 and 6. Because of the lack of data on steel fiber reinforced concrete performance, the fatigue relation was located as a conservative interpretation of the data rather than a "best fit." The suggested steel fiber reinforced concrete design fatigue relationships in Figures 5 and 6 for the Westergaard and layered elastic models are

$$DF_W = 0.35 + 0.25 \text{ Log } C$$

$$DF_{EL} = 0.11 + 0.38 \text{ Log } C$$

with the variables as defined previously in the section on fatigue.

There is more scatter of the data with the Westergaard model in Figure 5. Four of the six test items had stabilized base layers and one used a membrane encapsulated base. The layered elastic model probably did a better job of analyzing these structures with distinct stiff

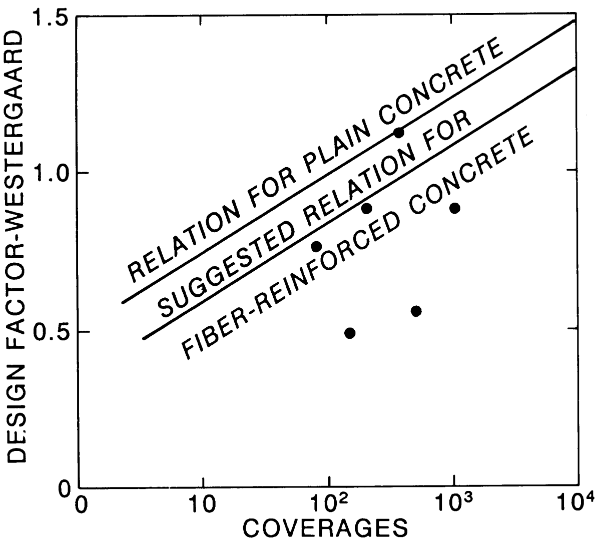

Figure 5. Steel Fiber Reinforced Concrete Pavement Fatigue Relationship for the Westergaard Model (Reference 26)

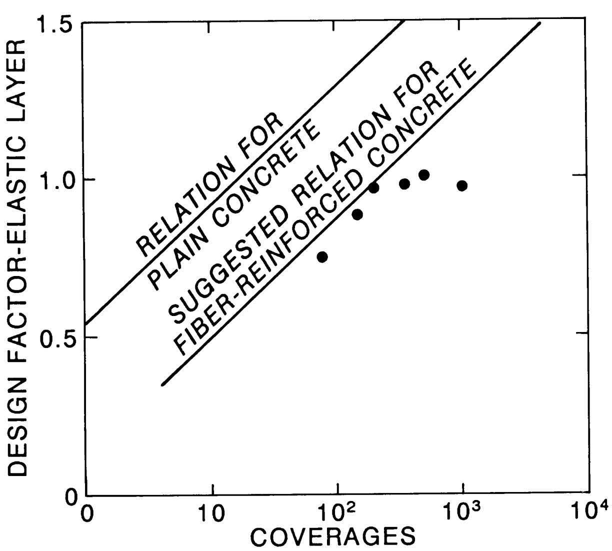

Figure 6. Steel Fiber Reinforced Concrete Pavement Fatigue Relationship for the Layered Elastic Model (Reference 26)

layers than did the Westergaard model that had to try to approximate their effect with a single k value.

There is some danger that large deflections of thin fiber reinforced concrete could introduce new modes of failure in the base or subgrade rather than fatigue failure of the concrete surface. Consequently, the 1979 manual (1) included a deflection check to keep deflections of the steel fiber reinforced concrete in the same range as encountered with conventional pavements. This deflection check was kept in the 1988 manual;(3) and the basis for the procedure was described previously.(2) Generally, it only controls for heavy aircraft on soft subgrades.

PRESTRESSED CONCRETE

A crack in a prestressed pavement acts like a plastic hinge. Because of the prestress the slab redistributes the bending moment, and failure of the slab does not occur until the negative radial bending moment around the load causes tensile cracking in the surface. The prestressing allows use of the concrete in the inelastic range and consequently greatly increases the load carrying capacity of the pavement. The Corps of Engineers developed an empirical design equation for prestressed pavements from model and limited full scale tests in the 1950's, and this design procedure was included in both the 1979 and 1988 manuals.(1,3) This procedure was described previously.(2)

The Corps of Engineers was forced to use empirical procedures for prestressed pavements because the elastic models such as Westergaard or layered elastic cannot be used to analyze inelastic behavior. Extrapolation of this empirical design based on model testing to new aircraft gear configurations was of uncertain validity and a better basis for design was needed. In 1986 Dr.'s Barenberg and Ioannides at the University of Illinois developed the framework for a new prestressed design procedure for the U.S. Army Engineer Waterways Experiment Station. The analytical basis for this work is a finite element model that represent the concrete pavement as an assemblage of four node plate elements on a subgrade of springs. The stiffness of each element composing the slab can be varied to model the development of the plastic hinge and redistribution of moments.

This new prestress design concept shows considerable promise and has given good agreement with the 1950's Corps of Engineers prestressed pavement test results, but more experimental work is needed to better define the reduced stiffness of the cracked concrete. The results of the work to this point will hopefully be published shortly. Further progress on this promising work is unfortunately stalled by a general lack of pavement research funding within the Corps of Engineers.

OVERLAYS

The Corps of Engineers rigid and flexible overlay design methods are empirical and based on testing conducted in the 1940's and 1950's.

Development of these empirical design equations is described by Mellinger (29), Hutchinson and Wathen (30) and Mellinger and Sale.(31) The basic overlay design equations are

$$h_o^n = h_{eq}^n - C_1 h_b^n$$

$$t = 2.5 \left(Fh_{eq} - C_2 h_b\right)$$

where

h_o = thickness of concrete overlay

h_{eq} = thickness of equivalent new concrete required to support the design traffic if no base pavement existed

C_1 = condition factor for base pavement under a rigid overlay
= 1.00 if little or no cracking
= 0.75 some initial structural cracking but little spalling or multiple cracks
= 0.35 if badly cracked with shattered slabs, spalling, or faulting

h_b = thickness of base pavement to be overlaid

n = 1.0 for fully bonded overlays
= 1.4 for partially bonded overlays where no attempt to bond or to prevent bond is made
= 2.0 for unbonded overlays where a bond breaking layer is used

t = thickness of flexible overlay

F = a factor less than or equal to 1.0 that allows cracking in the base slab; charts for factor can be found in the design manuals (3,4)

C_2 = condition factor for base pavements under a flexible overlay
= 1.00 if no cracking or small amount of structural cracking
= 0.75 if base pavement contains limited multiple cracks
= 0.50 if base pavement contains extensive multiple cracks

These equations were developed when the U.S. Air Force was first bringing jet aircraft into widespread use, and there was considerable concern over the potential for spalled concrete to be ingested by the jet engines. For this reason failure for rigid overlay tests was defined to be initial cracking in the overlay. No such concern existed for flexible overlays so failure for these overlays was defined as large visible deflections indicating impending shear failure.

It is clear that the rigid and flexible overlay equations were developed for very different levels of performance. The F factor in the flexible overlay design equation allows the base slab to shatter into approximately 35 pieces. This difference in design philosophy can result in anomalies such as calculating negative flexible overlay thicknesses and also makes it impossible to compare rigid and flexible overlay designs on any rational or economic basis. Because of the generally unsatisfactory nature of the flexible overlay equation the constant 2.5 was increased to 3.0 in the 1988 design manual (3) simply to require thicker and presumably better performing flexible overlays. The layered elastic design manual (4) also uses these empirical equations.

The Federal Aviation Administration sponsored a study at the U.S. Army Engineer Waterways Experiment Station to find a mechanistic overlay design procedure to replace the 30 year old empirical Corps of Engineers overlay design equations. This study (14) developed a new proposed design procedure based on the layered elastic model and a complete reanalysis of the Corps of Engineers accelerated traffic test results. The major facets of the proposed design procedure and existing Corps of Engineers design procedure are compared in Tables 5 and 6 for rigid and flexible overlays.

These design procedures were compared by preparing representative designs for F-4, B-727, C-141, and B-747 aircraft and randomly selected traffic, soil, and concrete properties. As can be seen in Figure 7 the Corps of Engineers unbonded overlay design procedure provided a conservative design in all cases. This was not true, however, for the partially bonded equation which gave results that were sometimes conservative and sometimes not. Figure 8 shows the unbonded calculated results from Figure 7 and the partially bonded calculated results plotted together with the Corps of Engineers rigid overlay design equations. The proposed design procedure shows less advantage for the increased friction/bonding of the partially bonded overlays compared to unbonded overlays than do the existing Corps of Engineers equations. In fact, the Corps of Engineers partially bonded equation would fit the unbonded calculated results as well as it does the partially bonded calculated results. Figure 9 compares the thicknesses required by the Corps of Engineers flexible overlay equation and the proposed design procedure. In general, the Corps of Engineers equation allows thinner overlays than the proposed method up to about 14 in. after which it requires appreciably thicker overlays.

The existing Corps of Engineers overlay equations are hampered by their empirical basis, different failure definitions, and general inability to evaluate the complex interactions that occur in overlay pavements. Evaluation with a more powerful analytical model suggests that the Corps of Engineers unbonded equation is a simple conservative upper bound solution, but sometimes it is overly conservative. The Corps of Engineers partially bonded rigid overlay and flexible overlay equations, however, can give either conservative or unconservative results, and their validity is uncertain. Some additional analytical work is being funded on this topic by the Federal Aviation Administration. Reliable field performance data on overlays are scarce, and more data need to be collected.

Table 5. Comparison of Rigid Overlay Design Methods (Reference 14)

Design Considerations	Corps of Engineers	Proposed Method
Analytical model	Empirical equation	Layered elastic
Failure	Cracking in 50 percent of slabs	Predict deterioration in terms of Structural Condition Index
Interface conditions between overlay and base pavement	Adjust power in equation	Variable between full bond and and unbonded
Material properties	Equivalent required h needed as input to empirical equation	E , ν for all materials and flexural strength of concrete
Difference in strength/modulus of overlay and base pavement concrete	Adjust thickness of base pavement	Included directly in calculation of stresses and design factor
Cracking in base pavement before overlay	Reduce thickness of base pavement	Reduce E value of base concrete
Fatigue effects of traffic on uncracked base pavement	Not included	Included in terms of equivalent traffic
Cracking of base after overlay	Not applicable	Reduced E value of base as cracking progresses under traffic

Table 6. Comparison of Flexible Overlay Design Procedures (Reference 14)

Design Considerations	Corps of Engineers	Proposed Method
Analytical model	Empirical equation	Layered elastic model
Failure	Visual deflection in test sections	Onset of cracking in base slab or minimize additional cracking in base slab Limit tensile strain in AC Limit vertical strain in subgrade
Material properties	Equivalent required h needed as input to empirical equation	Include elastic modulus and Poisson's ratio for each layer Include flexural strength of concrete used to limit cracking Include allowable AC and subgrade strain as functions of modulus and repetitions
Cracking in base pavement before overlay	Existing pavement thickness is reduced	Modulus of concrete reduced in calculations
Reflective cracking and rutting in the flexible overlay	Not included	Not included

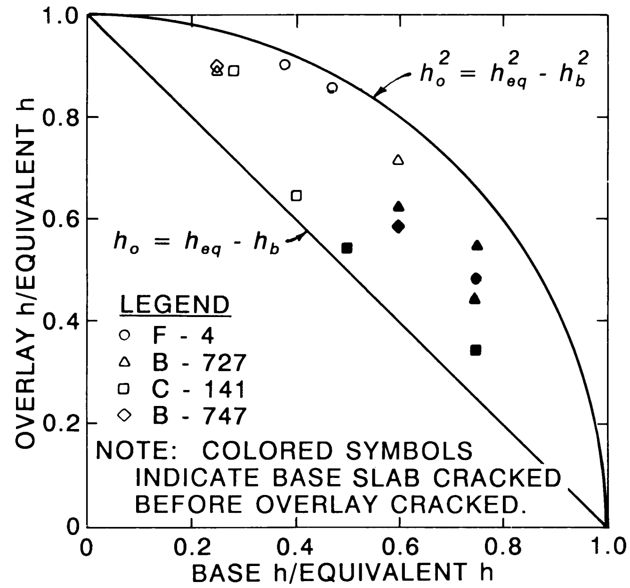

Figure 7. Unbonded Rigid Overlay Results (Reference 14)

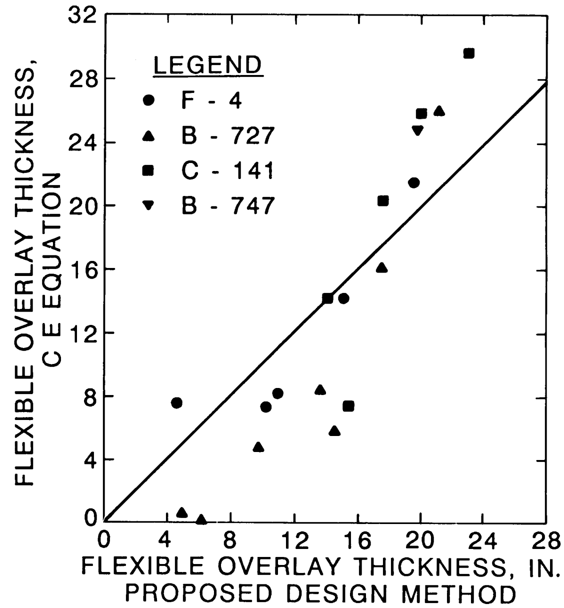

Figure 9. Flexible Overlay Results (Reference 14)

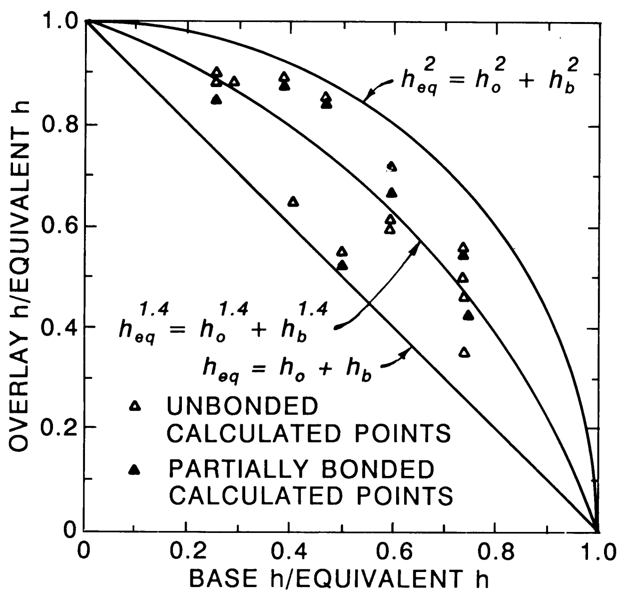

Figure 8. Partially Bonded Rigid Overlay Results (Reference 14)

CONCLUSIONS

Pavement design is in a constant state of evolution. The 1979 rigid airfield design manual (1) represented a significant step forward with its presentation of design methods for five different types of concrete pavement. Further work found that some provisions such as the continuously reinforced concrete pavement thickness design or allowable joint spacing for steel fiber reinforced concrete were overly optimistic and had to be adjusted for the 1988 design manual.(3) Other improvements such as the revised fatigue criteria for plain concrete were developed for the 1988 manual,(3) and additional work added further confidence in other concepts such as the steel fiber reinforced concrete failure definition. Also, the venerable Westergaard analytical model and some of the empirical approaches were proving inadequate for some problems. This led to a design procedure (4) based on the layered elastic design model being accepted as an alternative to the Westergaard based design method. Some promising work with finite element methods for prestressed pavements and layered elastic theory for overlays may put these currently empirical designs on a sounder basis in the near future.

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