

DESIGN METHOD FOR FLEXIBLE AIRFIELD PAVEMENTS

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A design method for flexible airfield pavements is discussed. The method is based on correlations of performance data of numerous full-scale accelerated traffic test pavements and computed critical stresses and strains of test pavements. The test pavements consisted of conventional flexible pavements and full-depth asphaltic concrete pavements. The loadings include single and multiple wheels. The stresses and strains in the pavement structures were computed by the finite-element technique incorporated with nonlinear stress-strain relations of pavement materials. Limiting criteria considered include radial tensile strains at the bottom of asphaltic concrete, maximum radial tensile strains and minimum ratio of radial tensile stress to vertical stress in the unbound granular layers, and vertical strains at the subgrade surface. The second criterion was developed only for single-wheel loads. The principle of superposition was used in the computations for multiple-wheel loads. The salient features of the design method are discussed, and limiting criteria are applied to the design of pavements and evaluation of pavement performance under mixed traffic and with overlays.

●COMMONLY USED methods for the design of pavements in engineering practice are the theoretical analysis approach and the experimental field approach. Much effort has been expended on both approaches, and in general there is poor correlation between them. Many theoretical methods, such as elastic layered systems and the finite-element method of analysis, are designed to compute the stress, strain, and displacement in the pavement structure but are not capable of evaluating the performance, i.e., cracking, distortion, etc., that is of primary interest to pavement design engineers. On the other hand, although an experimental, full-scale, field-test section approach produces reliable performance data, such test sections are both expensive and time-consuming and provide only minimum amounts of basic test data. In many cases, extrapolations must be used to estimate pavement behavior for cases not covered in field tests; consequently, criteria based on limited amounts of test data are generally conservative. Therefore, there is a definite need to bridge the gap between theory and performance. When such relationships, based on a broad spectrum of test conditions, can be established, design and evaluation of a pavement structure for a new aircraft can be accomplished by direct computations without full-scale tests being conducted for each new need. This method of combining theory and field performance is sound.

For years, the U.S. Army Corps of Engineers (CE) has gained its expertise in pavement design through the use of full-scale accelerated traffic tests. The observed pavement performance is expressed in terms of coverages to failure. Failure criteria are based on the cracking and rutting of pavements. Test results provide reliable and valuable information on the response of real pavements to actual traffic loads. From this information, failure criteria were established; however, it is the full-scale field test that is subject to criticism. The criticisms are generally technically unassailable because each test section represents but one test condition and extrapolation of the results into untested regions is dangerous. However, an improved pavement design methodology can and should be developed by taking full advantage of the valuable

information from full-scale traffic test results. [This can be done by correlating the field performance and the computed critical parameters (stresses and strains) of each test section, and then establishing limiting values of critical stresses and strains at different coverage levels (performance).]

With the limiting values, prediction of the performance of a pavement structure designed at a particular coverage level for a new aircraft that has never been tested becomes possible. Therefore, the design of new pavements that will be subjected to new aircraft can be done by computers without full-scale tests being conducted. This is theoretically justified because this methodology is based on results of numerous test pavements that were of different thicknesses, materials, and subgrade strengths and that were subjected to loads of different magnitudes and gear configurations. This method of combining theory and field performance not only has eliminated the disadvantages of full-scale testing but has fully exploited the data bank of full-scale testing results that are available at the U.S. Army Engineer Waterways Experiment Station.

Test pavements analyzed consisted of conventional flexible pavements and full-depth asphaltic concrete pavements. The loadings include single and multiple wheels. The stresses and strains in the pavement structures were computed by the finite-element technique incorporated with nonlinear stress-strain relations of pavement materials. The limiting criteria include radial tensile strains at the bottom of the asphaltic concrete, maximum radial tensile strains and minimum ratio of radial tensile stress to vertical stress in the unbound granular base course layers, and vertical strains at the subgrade surface. The second criterion was developed only for single-wheel loads. The principle of superposition was used in computing multiple-wheel load assemblies.

Although the results of the full-scale accelerated traffic tests used were limited only to aircraft loadings, it is believed that the criteria, after appropriate modifications, can be used for designing pavements subjected to highway loadings.

THEORETICAL COMPUTATIONS

Finite-Element Analysis

The nonlinear finite-element program (FEPAVE) used was obtained from the University of California at Berkeley. The program is suitable only for the analysis of axisymmetric solids (a single load with a circular loaded area). The surface load was applied stepwise so that the nonlinear stress-strain behavior and the modulus-stress dependency of the material could be included in the analysis. Ten and twelve load increments were used.

Material Characterizations

Because pavement materials are subjected to repetitive applications of traffic loads and because the stress intensities in the pavements are generally small and well below failure strengths of the materials, it was felt that the resilient modulus could best characterize pavement material behavior under traffic loads as compared to other types of material characterizations. Therefore, better correlations between performance and computed values could be expected. A discussion of the various stress-strain relations for different pavement materials included in this study follows.

Asphaltic Concrete—Because of the thermo-viscoelastic nature of asphaltic materials, the most important factors influencing the stress-strain relationships of asphaltic materials are temperature and rate of loading. The resilient moduli of asphaltic materials should be evaluated in the laboratory at different temperatures and at different rates of loading. However, such data for the actual asphaltic mixtures used by CE were not available during the preparation of this report; therefore, resilient moduli of asphaltic mixtures developed by the Asphalt Institute were used (1). It is believed that the patterns of resilient moduli of asphaltic mixtures used by CE would be similar to those developed by the Asphalt Institute (1). For each test pavement, a mean temperature versus depth relation for the whole traffic period was determined and was used in the computations. Poisson's ratio of 0.4 was used for all asphaltic mixtures in this study.

Untreated Granular Materials—The characterizations of untreated granular materials were based on research conducted at the University of California (2). The resilient modulus, M_R , increases with the sum of principal stress, σ , according to

$$M_R = k_1 \sigma k_2 \quad (1)$$

where k_1 and k_2 characterize the resilient property of the granular material. Because laboratory results characterizing the resilient properties of granular materials used by CE were not available, representative values from other sources were chosen (2). For well-compacted base-course materials, k_1 and k_2 were 8,300 and 0.71 respectively, and for sand and gravel subbase materials, k_1 and k_2 were 2,900 and 0.47 respectively. The method for selecting the values of these constants will be described later.

The Poisson's ratio of granular material is also known to be stress-dependent. Because of lack of experimental results, a constant Poisson's ratio of 0.48 was used for both base and subbase materials. It was believed that, because granular materials under a thin asphaltic concrete surface course in a pavement structure tend to expand under the heavy aircraft load, a large Poisson's ratio would represent field conditions better than a small one.

Subgrade Soils—Extensive studies of the behavior of fine-grained materials in laboratory repeated-load tests have been made at the University of California (3). It was found that the resilient modulus did not depend on the confining pressure but was sensitive to deviator stress. At low-stress levels, the resilient modulus decreases rapidly with increasing values of the deviator stress, and as the deviator stress further increases, there is only a slight increase in resilient modulus. Because information on resilient modulus for subgrade soils used by CE was not available, this empirical equation (4) was used in the analysis:

$$E \text{ (psi)} = 1,500 \text{ CBR} \quad (2)$$

Poisson's ratio of 0.4 was used in the computations.

CORRELATION OF THEORY AND PERFORMANCE

This section applies the concept and computational procedures described to the analysis of performance data of full-scale accelerated traffic tests. Tables 1 and 2 give test data from many selected test sections for single- and multiple-wheel loads; the data cover a broad spectrum of loads, gear configurations, pavements, subgrade soils, and coverage levels. The failure criteria, from which the coverage levels were determined, were based on a subjective rating of pavement performance and primarily on rutting and cracking of the pavements. One coverage is defined to be sufficient passes of load tires in adjacent tire paths to cover a given width of surface area one time. The moving test loads were normally distributed across the surface of the traffic lane.

FEPAVE assumes that the pavement structure is an elastic medium with nonlinear stress-strain characteristics. The computed stress, strain, and displacement are, therefore, the response of the pavement to load before initial failure occurs or, preferably, during the early stage of traffic. Evaluation of the performance of full-scale test sections under traffic, however, is based on the integral pavement behavior up to the point of functional failure. Therefore, finite element programs can be used to predict pavement behavior during the early stage of traffic but cannot be used to predict failure coverage of the pavement. In fact, there is not a single computer program available to date that is capable of predicting functional failure of pavement in a simple and straightforward manner. Therefore, effort was confined to correlating computed values and actual observed coverages to failure of the test pavements from which the parameters controlling the pavement performance could be identified. This was based on the hypothesis that correlations must exist between performance of the pavement and stresses and strains actually developed in the pavement under load. Because the stresses and strains have to be theoretically computed, the success of the

correlation depends entirely on how good the stress-strain relations of pavement materials put into the computer program are. It was apparent that the more closely the input represented material properties, the better would be the correlations.

The rationale for determining the granular material constants k_1 and k_2 in Eq. 1 follows.

For all of the test pavements analyzed (Tables 1 and 2), there were generally three basic types of pavement structures:

1. Full-depth asphaltic concrete pavements resting on 4-CBR buckshot clay subgrade soil.
2. Conventional flexible pavements consisting of a 3-in.-thick (7.6 cm) asphaltic concrete layer, a 6-in.-thick (15.2 cm) base course, various thicknesses of subbase layer, and subgrade soils with various CBRs. (In most cases, the thickness of the subbase layer was much greater than the base course; therefore, the subbase layer practically controlled the performance of the pavement.)
3. Pavements with a 3-in.-thick (7.6 cm) asphaltic concrete layer, a thick, crushed-stone base course, and subgrade soils with various CBRs.

If improper stress-strain relations of pavement materials were used in the computations for these three different types of pavements, it is obvious that good correlations between computed values and observed performance of test pavements could not be obtained and, more than likely, three different correlations would result. On the other hand, if correct stress-strain relations were used, a single good correlation should be obtained. Values of k_1 and k_2 were carefully determined based on this logic.

Because all of the full-depth asphaltic concrete pavements were constructed over 4-CBR subgrade soil, good correlations between computed values and performance were first established. Computations for the conventional flexible pavements and pavements with thick, crushed-stone base courses were then made. In Eq. 1, k_1 and k_2 were initially assumed to be 2,900 and 0.47 for subbase materials and 3,933 and 0.61 for base-course materials (2). Correlations for conventional flexible pavements agreed well with those of full-depth asphaltic concrete pavements, but correlations for thick, base-course pavements did not; these values were much too large. These relationships indicated that the constants k_1 and k_2 chosen for subbase materials were reasonable but that the constants selected for the base-course materials were too small. Higher values of k_1 and k_2 were tried to obtain good correlations, and the final values selected and used were 8,300 for k_1 and 0.71 for k_2 .

Single-Wheel Test Data

Table 1 gives test data from 25 selected test pavements. The base-course material was well-compacted crushed stone, and the subbase material was gravelly sand. The traffic on the pavement for test point 14 was actually applied at two ambient temperature conditions: (a) 60 to 70 F (15.6 to 21 C) and (b) 90 to 115 F (32 to 45.5 C). The observed performances of the pavements were identical, indicating that the 3-in.-thick (7.6 cm) temperature-dependent asphaltic concrete surface layer had little effect on the performance of the pavement. The results for the cooler traffic period were used in this analysis.

Figure 1 shows the relations between vertical strain at the subgrade surface and performance. Figure 2 shows similar relations for radial tensile strain at the bottom of asphaltic concrete layers. Uncertainties exist in the computed values for pavements with subgrades having CBRs greater than 10; these are pavement test points 4, 5, 6, 7, and 8 shown in Figures 1 and 2 by open circles. These pavements were excluded from the final analysis but will be discussed separately.

In Figure 1, the best-fit line was drawn through the points for all pavements with greater than 100 coverages to failure. It seems that another straight line could have been drawn through the pavements with coverages less than 100. From the distribution of pavement points in Figure 2, it was concluded that, for pavements with coverages less than 100, the subgrade condition was not critical before initial failure had occurred; the critical factor was the radial tensile strain at the bottom of the asphaltic

Table 1. Single-wheel data.

| Test Point | Reference | Wheel Load (kips) | Tire Contact Area (in. ²) | Thickness (in.) | | | Subgrade CBR ^a | Coverages at Failure |
|------------|-----------|-------------------|---------------------------------------|-----------------|------|---------|---------------------------|----------------------|
| | | | | Surface | Base | Subbase | | |
| 1 | 5 | 200 | 1,500 | 7 | 13 | 19 | 6 | 150 |
| 2 | 5 | 200 | 1,500 | 7 | 15 | 21.5 | 9 | 1,700 |
| 3 | 5 | 200 | 1,500 | 6 | 13 | 30 | 8 | 1,300 |
| 4 | 5 | 200 | 1,500 | 6 | 12 | 0 | 16/6 (41.5 in.) | 10 |
| 5 | 5 | 200 | 1,500 | 6.5 | 14 | 0 | 18/6 (45.5 in.) | 60 |
| 6 | 5 | 200 | 1,500 | 6.5 | 14.5 | 25 | 15.5/6 (39.5 in.) | 360 |
| 7 | 5 | 200 | 1,500 | 7 | 14 | 9 | 17.5/6 (34 in.) | 1,500 |
| 8 | 6 | 30 | 150 | 3 | 6 | 6 | 14 | 216 |
| 9 | 7 | 15 | 250 | 2 | 8 | 0 | 8 | 3,760 |
| 10 | 7 | 15 | 250 | 2 | 8 | 0 | 9 | 3,760 |
| 11 | 8 | 50 | 285 | 3 | 6 | 6 | 3.7 | 6 |
| 12 | 8 | 30 | 285 | 3 | 6 | 6 | 3.7 | 120 |
| 13 | 6 | 30 | 150 | 1.5 | 10.5 | 0 | 7 | 178 |
| 14 | 6 | 30 | 150 | 2 | 10 | 0 | 6 | 203 |
| 15 | 9 | 10 | 91 | 0 | 5 | 0 | 6 | 40 |
| 16 | 7 | 50 | 667 | 3 | 23 | 0 | 5 | 3,000 |
| 17 | 7 | 20 | 334 | 3 | 14.5 | 0 | 5 | 5,000 |
| 18 | 7 | 15 | 250 | 2 | 0 | 10 | 5 | 582 |
| 19 | 7 | 25 | 416 | 2 | 4 | 12 | 5 | 385 |
| 20 | 10 | 75 | 270 | 15 | 0 | 0 | 4 | 6 |
| 21 | 10 | 75 | 270 | 15 | 0 | 0 | 4 | 8 |
| 22 | 8 | 50 | 285 | 3 | 6 | 15 | 4 | 200 |
| 23 | 10 | 75 | 270 | 9 | 0 | 15 | 4 | 12 |
| 24 | 10 | 75 | 270 | 24 | 0 | 0 | 4 | 90 |
| 25 | 11 | 75 | 270 | 3 | 21 | 0 | 4 | 50 |

Note: 1 kip = 4.448 222 kN. 1 in.² = 6.451 600 cm².

^aSingle entries indicate a CBR that is representative of the entire depth of test; double entries indicate a change of CBR at depth (e.g., for test point 4, the upper 41.5 in. of subgrade had a CBR of 16 and below that depth, a CBR of 6 was representative).

Table 2. Multiple-wheel data.

| Test Point | Reference | Aircraft Type | Assembly Load (kips) | Tire Contact Area (in. ²) | Thickness (in.) | | | Subgrade CBR | Coverages at Failure |
|------------|-----------|---------------|----------------------|---------------------------------------|-----------------|-----------------|---------|--------------|----------------------|
| | | | | | Surface | Base | Subbase | | |
| 1 | 8 | Boeing 747 | 240 | 290 | 3 | 6 | 24 | 3.8 | 40 |
| 2 | 8 | Boeing 747 | 240 | 290 | 3 | 6 | 24 | 4 | 40 |
| 3 | 8 | Boeing 747 | 240 | 290 | 3 | 6 | 32 | 4 | 280 |
| 4 | 8 | C-5A | 360 | 285 | 3 | 6 | 6 | 3.7 | 8 |
| 5 | 8 | C-5A | 360 | 285 | 3 | 6 | 24 | 3.8 | 1,500 |
| 6 | 8 | C-5A | 360 | 285 | 3 | 6 | 24 | 4 | 1,500 |
| 7 | 10 | C-5A | 360 | 285 | 3 | 12 ^a | 0 | 4 | 98 |
| 8 | 10 | C-5A | 360 | 285 | 15 | 0 | 0 | 4 | 425 |
| 9 | 8 | C-5A | 360 | 285 | 3 | 6 | 15 | 4 | 104 |
| 10 | 10 | C-5A | 360 | 285 | 9 | 0 | 15 | 4 | 734 |
| 11 | 10 | C-5A | 360 | 285 | 9 | 15 ^a | 0 | 4 | 2,198 |
| 12 | 11 | C-5A | 360 | 285 | 3 | 21 | 0 | 4 | 5,000 |
| 13 | 11 | Boeing 747 | 200 | 285 | 3 | 21 | 0 | 4 | 890 |

Note: 1 kip = 4.448 222 kN. 1 in.² = 6.451 600 cm².

^aAsphalt-stabilized.

Figure 1. Relationship between vertical strain at subgrade surface and performance of pavement under single-wheel loads.

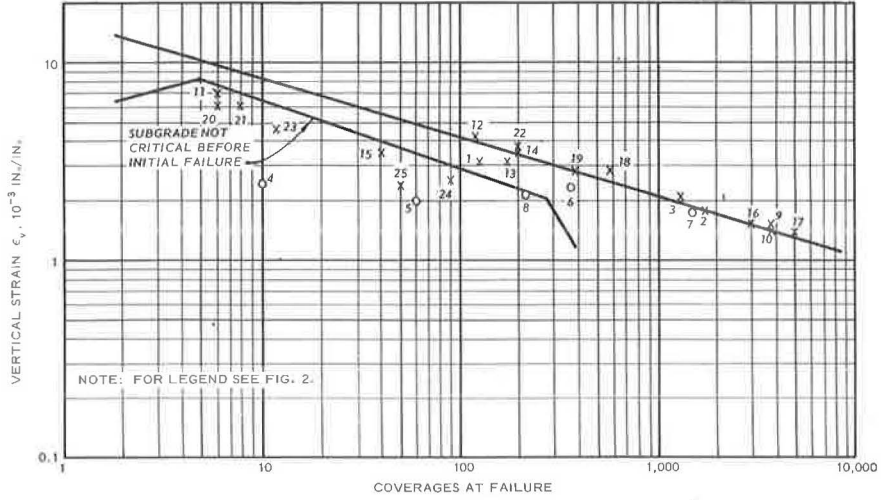


Figure 2. Relationship between radial tensile strain at bottom of asphaltic concrete and performance of pavement under single-wheel loads.

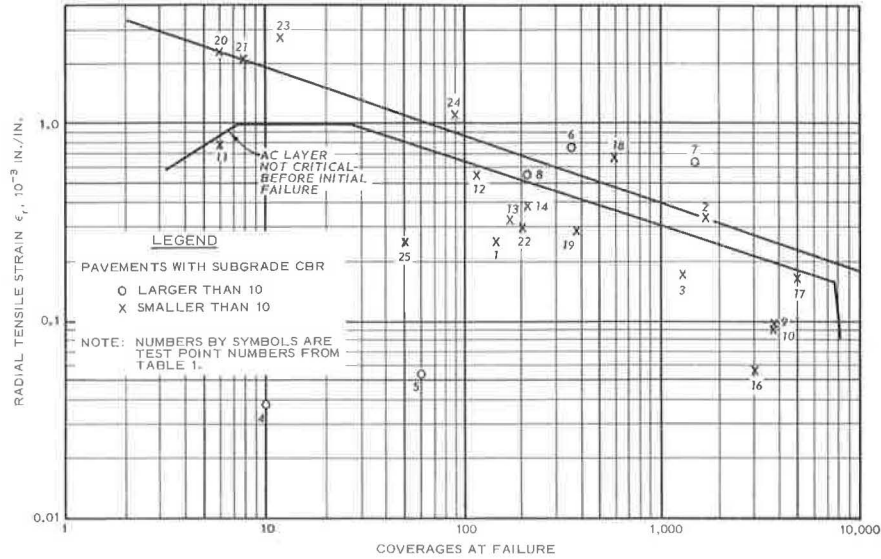
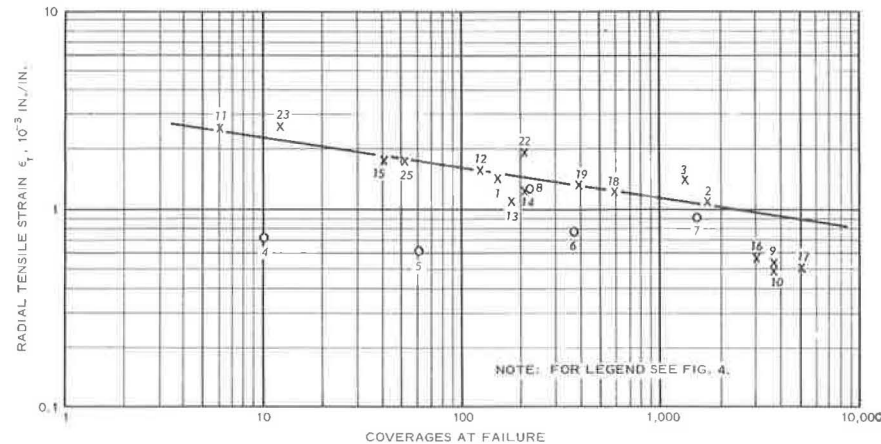


Figure 3. Relationship between maximum radial strain in granular materials and performance of pavement under single-wheel loads.



concrete layer. The straight line in Figure 2 was drawn accordingly and passed through test point 2. Sufficient failure test data were not available at high coverage levels; therefore, the validity of the criteria established at high coverages needs to be further verified by additional field performance data. The radial tensile strains for pavement points below the line were not considered to be critical before initial failure.

FEPAVE computes the pavement's elastic stresses and strains, which give an indication of the response of the pavement structure to load before initial failure occurs but do not provide information on the failure modes of the pavements. Figures 1 and 2, for example, do not show that pavements represented by points 20 and 21 were failed in the asphaltic concrete layers and that neither was failed in the subgrade soils. Rather, during the initial stage of the traffic, asphaltic concrete layers under the 75-kip (336 kN) load, 278-psi (1920 kPa) contact pressure were more critical than the subgrade. As load repetition increased, small cracks could have developed in the asphaltic concrete layers and therefore could have reduced its stiffness. As a result, the vertical strains at the subgrade surface would have increased and caused the subgrade soil to fail, which in turn would have increased the cracks in the asphaltic concrete. Eventually, the pavements would fail both in the asphaltic concrete and in the subgrade soil, as was actually observed in the test sections. One can see that the computed values provide information on the most critical factors contributing to the pavement failures rather than indicate the failure modes of the pavement under load.

For pavements with thin asphaltic concrete surface layers, i.e., 2 or 3 in. (5.1 or 7.6 cm) thick, it is believed that such thin layers would not contribute to pavement failure as much as other pavement components. Therefore, design considerations should not be solely based on the thin layer of asphaltic concrete even though the computations indicate that this thin layer is critical under the load (Fig. 2, point 18).

The results of analysis of pavements with base courses of unbound granular materials are shown in Figures 3 and 4; full-depth asphaltic concrete pavements were excluded. Figure 3 shows the results of maximum radial tensile strains. The computer output indicated that locations of the maximum strains were generally near the load axis but varied in depth. The straight line was drawn based on the following reasoning. Because the failure coverages of the pavements (shown by points 2, 18, 19, 1, and 12) could be predicted by the correlation of subgrade vertical strains (Fig. 1), it was reasonable to assume that the failure coverages of these pavements should be at least equal to or greater than those predicted by the criteria of shear failure in the unbound granular materials. The straight line was therefore drawn through these pavement data points. Note that the straight line also went through data points 15 and 11. The pavement shown by point 15 had no asphaltic concrete surface but consisted of a 5-in.-thick (12.7 cm) crushed-stone base placed on 6-CBR subgrade soil. Figure 1 shows that the failure of this pavement did not initiate from the subgrade soil and that, because this pavement had no other component layer but the granular material, the failure of this pavement had to initiate from the unbound granular material under the 10,000-lb (4530 kg) load and 110-psi (760 kPa) contact pressure. Pavement 11 had thin structural layers [3-in.-thick (7.6 cm) asphaltic concrete over a 6-in. base and 6-in. (15.2 cm) subbase] and was subjected to a relatively heavy load [50,000 lb (22 600 kg)] at a high tire-contact pressure [175 psi (1210 kPa)]. Figures 1 and 2 show that the initiation of failure of test pavement 11 was not from the asphaltic concrete layer but was probably from the subgrade soil. Figure 3 shows that failure of this pavement was initiated from the granular materials because of the excessive shear stress.

Pavement data points 7, 9, 10, 13, and 14 (Fig. 3) plotted below the straight line indicate that failure of these pavements was not initiated from the unbound granular materials. In fact, these pavement points all plotted along the vertical strain criterion in Figure 1.

Because of the possibility of shear failure of unbound bases induced by tensile stresses, Brown and Pell (12) suggested that the design criteria for unbound materials should be a horizontal tensile stress that should not exceed 0.5 times the vertical stress plus the horizontal overburden pressure. For all the pavements analyzed, it was found from the computer output that shear failure of unbound bases did not occur. Figure 4 shows the plot of the ratios of radial tensile stress to vertical stress in the unbound

Figure 4. Relationship between vertical strain at subgrade surface and performance of pavement under single-wheel loads.

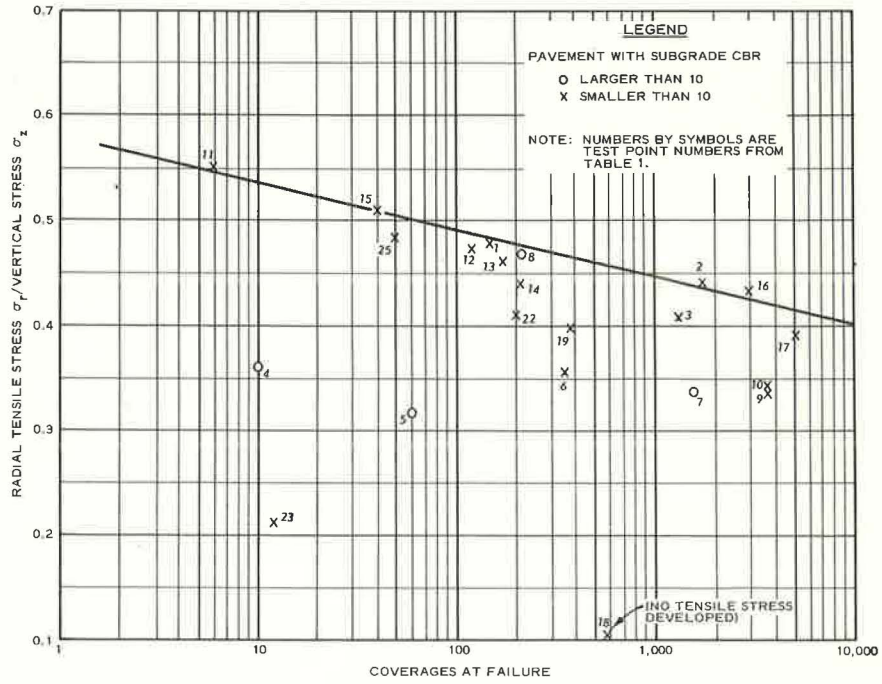


Figure 5. Relationship between vertical strain at subgrade surface and performance of pavement under multiple-wheel loads.

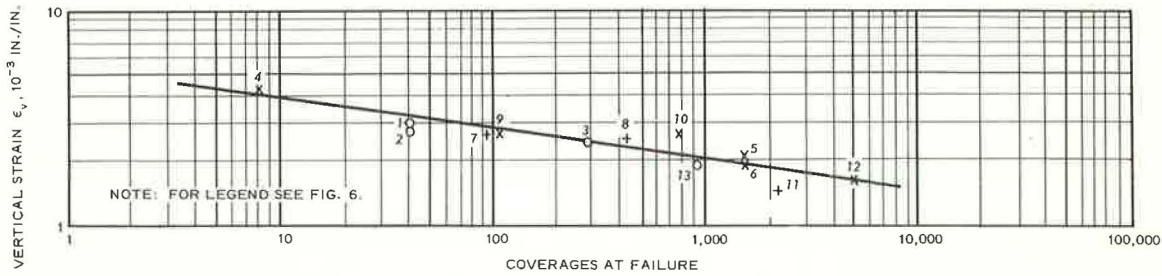
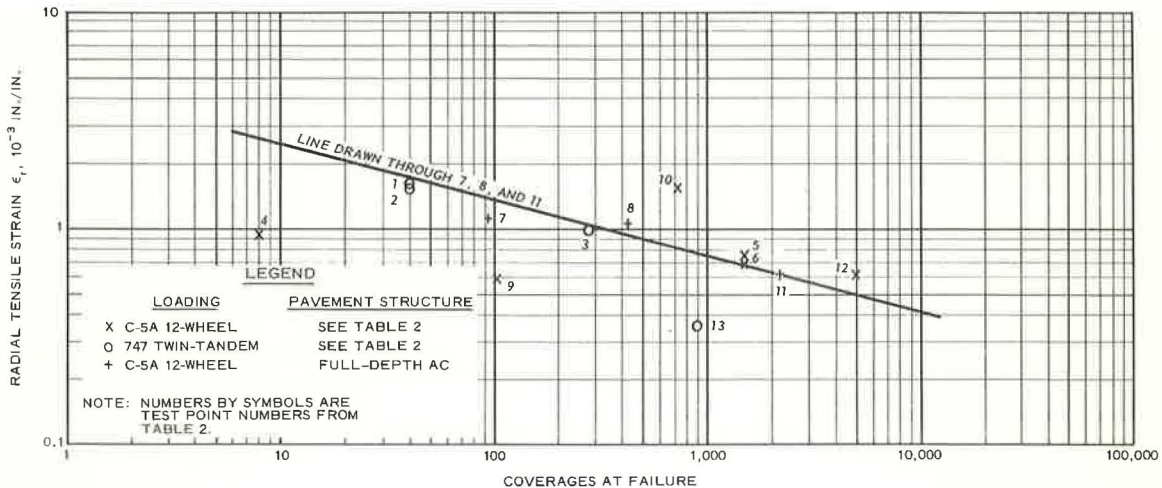


Figure 6. Relationship between radial tensile strain at bottom of asphaltic concrete and performance of pavement under multiple-wheel loads.



granular materials versus failure coverages of test pavements. The ratios were the maximum values selected within the unbound bases. In most cases, this ratio and the maximum tensile strain do not occur at the same element. The straight line, which was very compatible with the line in Figure 3, was drawn through the upper boundary of the plotted points. Similarly, the points located below the line were the pavements in which the shear failures of unbound granular layers were not critical. The results shown in Figures 3 and 4 are both for unbound granular materials, but they use different parameters. The relations were good except for test points 16, 19, and 22.

The information shown in Figures 1 to 4 could be used to optimize the design of a pavement. For instance, the pavement shown as test point 2 had an optimum design, because the subgrade, the unbound base, and the asphaltic concrete were all equally critical under the 200-kip (890 kN) wheel load.

There were uncertainties in the computed results for test pavements 4 to 8. These pavements had subgrade soils with high CBRs ranging from 14 to 18. The test points are represented by open circles in Figures 1 to 4 and were not considered in drawing the straight lines in the figures. The computed values for pavements 4 (CBR = 16) and 5 (CBR = 18) were much too low in Figures 1 to 4; this indicates that the moduli computed by the relation $E = 1,500 \text{ CBR}$ were too high. However, the computed results for pavements 6 (CBR = 15.5), 7 (CBR = 17.5), and 8 (CBR = 14) seem to fit the line fairly well and indicate the relation $E = 1,500 \text{ CBR}$ was not bad. The situation is puzzling. Nevertheless, the moduli determined from the relation $E = 1,500$ do not seem to be adequate for high-CBR subgrade soils.

Multiple-Wheel Test Data

Table 2 gives test data for 13 selected test pavements under multiple-wheel, heavy gear loads. The pavements were trafficked by full prototype loadings of a 12-wheel assembly (one main gear of a C-5A) and a twin-tandem assembly (one twin-tandem component of a Boeing 747).

FEPAVE is not capable of handling multiple-wheel loads. In fact, such a nonlinear program is not available currently. Thus, the superposition principle was used to obtain solutions for multiple-wheel loads. This was done by first constructing the stress basins under the single-wheel load and then superposing the ordinates for different wheels. A computer program was prepared to compute the stresses and strains at any location under the multiple-wheel loads. The limitations on the use of the principle of superposition and the validity of the superposition principle are discussed elsewhere (13, 14). Stresses and strains were computed along the load axis of one wheel of the assembly, maxima were not searched for. For the C-5A gear assembly, the wheel was chosen to be either one of the inner wheels in the second row of the 12-wheel assembly. When computations are made for thick pavements, the values computed are smaller than the real maxima. This could be one of the factors contributing to the scattering of results in Figures 5 and 6.

Figure 5 shows the relation between vertical strains on the subgrade surface and failure coverages; a straight line could be drawn through pavement points at all coverage levels. The correlations indicate that pavement performance subject to multiple-wheel loads could be predicted by computed values of vertical strain on a subgrade surface. However, the correlation (Fig. 5) should not be interpreted because all of these pavements were failed in the subgrade soil.

Figure 6 shows the relation between radial tensile strain at the bottom of asphaltic concrete layers and performance of the test pavements under multiple-wheel loads. For pavements with 3-in.-thick (7.6 cm) asphaltic concrete surface layers, the radial tensile strain (at the bottom of the concrete) under a single-wheel load was the same as the strain under one wheel of the multiple-wheel load. This is because the asphaltic concrete was thin and the wheels were spaced so far apart. For the full-depth asphaltic concrete pavements, at the bottom of the thick pavement, the radial tensile strain under one wheel had added strains that were contributed by the other wheels. Therefore, it was believed that radial tensile strain in pavements with thin asphaltic concrete layers should not be considered as the controlling factor in predicting pavement performance.

The line in Figure 6 was drawn through the full-depth pavements, test points 7, 8, and 11; pavements 1 to 6, 12, and 13, which were flexible pavements with 3-in.-thick (7.6 cm) asphaltic concrete surfaces, were not considered. Computed radial tensile strains in pavements 1, 2, 4, and 13 were small as compared with those of the full-depth asphaltic concrete pavements because of the nature of the thin asphaltic concrete layer. For pavements 3, 5, 6, and 12, however, they fell rather close to the straight line, but the closeness was considered to be only coincidental. In fact, the computed radial strains at the bottom of the 3-in.-thick (7.6 cm) asphaltic concrete layer in pavements 1 to 6 were nearly the same. The reason for test point 10 being located so far above the line is not known. One possible explanation is that the temperature input into the computer program was incorrect for the 9-in.-thick (22.8 cm) asphaltic concrete layer.

The results shown in Figures 5 and 6 are limited to subgrade soils with a strength of 4 CBR. The question arises whether the criteria established would be applicable to subgrade soils of different strengths. Because of the criteria for single-wheel loads (Figs. 1 to 4), in which the results for pavements on low-CBR subgrades were mixed well with those for pavements with higher CBR subgrades, one can justifiably conclude that the correlations in Figures 5 and 6 for multiple-wheel loads can be used for pavements other than those on 4-CBR subgrade soils.

Criterion for shear failure in unbound granular materials was not established for pavements subjected to multiple-wheel loads because of the difficulty in searching for maximum radial tensile strain (Fig. 3) and the maximum stress ratio (Fig. 4) in the unbound layers when the principle of superposition is used.

Special Remarks

Although the nonlinear finite-element method possesses many deficiencies in computing stresses and strains in pavement structures under traffic loads, it still yields reasonably good results. Apparently, this method is a good and powerful tool to find correlations between theory and observed performance. This is manifested by the good correlations shown in Figures 1 to 6.

Tensile strains (fatigue cracking) in an asphaltic concrete layer are more critical in cold weather. Most test pavements analyzed in this study were trafficked in relatively warm weather. Some full-depth asphaltic concrete pavements should be tested in cold weather and information obtained would undoubtedly enhance the proposed criteria.

It should be emphasized that the purpose of this study was not to predict the stresses and strains in the pavement structures; the purpose was to use the computed values to predict pavement performance. Attempts were made to compute the stresses and strains in pavement structures as close as possible to those actually measured. Because the computed values are computer-generated reproducible numbers the performance of a new pavement subject to a new aircraft can be predicted from the established correlations through computations. When better stress-strain relations of pavement materials are formulated and when more information on traffic tests are collected, the established correlations can be refined and the design procedures can be improved and updated. Therefore, the nonlinear finite-element method is not only the best method to fulfill the goal of this study, but it holds promise for the future.

Because of space limitations, many important features of the proposed design method cannot be presented in detail (13).

APPLICATION OF ANALYSIS

The results shown in Figures 1 to 6 represent correlations between theory and performance. The computed values are reproducible quantities generated by the computer and are based on the nonlinear theory of elasticity; the repetitions at failure are observed performance from full-scale accelerated traffic test sections. Test data covered a broad spectrum of test sections, wheel loads and configurations, subgrade soil strengths, and coverage levels (Table 2). Confidence is justified in using the limiting criteria for the design of pavements for new aircraft. We believe that cor-

relations developed can be used for designing and evaluating pavements subjected to mixed traffic and overlays of existing flexible airfield pavements.

Design of New Pavements

Although the CBR equation has been used satisfactorily by CE in the design of flexible airfield pavements for many years, there are certain pavement characteristics that are not adequately accounted for by the CBR equation because of the lack of supporting information. Designs for pavements in this category tend to be conservative. Pavements designed by the CBR equation have adequate thickness over the subgrade and over each layer of the structure to prevent shear failure. There is, however, no benefit given for use of materials stronger than the required minimum. For instance, when subbase material is replaced by material qualified as base course, the total thickness of the pavement structure is not reduced and the designed coverage level is not increased. The lack of consideration of these aspects in the CBR equation is not because of lack of knowledge but is mainly because of the lack of sufficient experimental data to support these considerations, and the CBR equation was based on test results of conventional flexible pavements and is, therefore, not adequate for designing pavements with full-depth asphaltic concrete or pavements with unconventional materials. However, with the proposed method in this study, i.e., computing the critical stresses and strains for the particular pavement section, some of the drawbacks inherent in the CBR equation can be overcome.

The design method has the capability of analyzing more sophisticated future pavements made of new pavement materials. The characterization of the new materials as determined in the laboratory is required for this. From the criteria in Figures 1 to 6 and FEPAVE, the thickness of each component layer of the pavement can be optimized so that the failure of the pavement initiates in surface, base, and subgrade layers at the same coverage level.

For multiple-wheel gear loads, the results in Figures 5 and 6 have their special merit because the influence of gear configuration on a particular pavement can be evaluated. For instance, the performance of pavements under the loading of the C-5A assembly has been evaluated experimentally in recently completed full-scale traffic tests (7). The performance of pavements under the loading of other multiple-wheel assemblies with different gear configurations can be evaluated readily by the method in this paper, and there is no need to conduct another full-scale traffic test that would be costly and time-consuming. It is also known (8) that the advantage of increasing the number of wheels in a gear assembly, such as the 12-wheel, main-gear C-5A assemblies, is that the load can be spread out into a larger area and, consequently, the deflections and shearing strains in the pavement are more reduced than those under an equivalent single-wheel load. The question often arises whether it is more economical to increase the number of wheels in a gear assembly or to increase the thickness of the airfield pavement to accommodate the demands imposed by ever-increasing heavy aircraft loads. The method in this paper provides a powerful tool for determining an optimum design for such a situation.

Evaluation of Pavements for Mixed Traffic and Overlays

The correlations in Figures 1 to 6 can be used to evaluate the performance of pavements subjected to mixed traffic and overlays of existing flexible pavements; the procedures for such an evaluation follow.

Mixed Traffic—When a certain percentage of the service life of an airfield pavement has been used by a given aircraft load, the remaining life of this pavement for another type of aircraft loading can be evaluated by using the proposed criteria as follows:

1. Required. For a pavement that was designed for 5,000 coverages for a 30-kip (133 kN) single-wheel load that has been trafficked for 2,000 coverages, determine the remaining service life of this pavement under a 50-kip (222 kN) single-wheel load.

2. Solution. A computation is first made for this pavement under a 50-kip (222 kN)

load and the anticipated coverages at failure are determined—for illustration, assume that the result is 1,000 coverages. Because 40 percent of the design service life of the pavement has already been used by the 30-kip (13 N) load, the pavement would sustain 600 more coverages of the 50-kip (22 N) load.

Overlays—For the pavement described in the previous example, 3 in. (7.6 cm) of asphaltic concrete may be required for resurfacing the old pavement to satisfy durability requirements when the load is increased from 30 (133 kN) to 50 kips (222 kN). The remaining service life of the pavement with the overlay can be evaluated accordingly. Computation is first made for the new pavement, i.e., the original pavement plus the overlay, and the anticipated coverages at failure are then determined. Assuming that 1,500 coverages are indicated instead of the original 1,000, the pavement with overlay would then be expected to sustain 900 additional coverages under the 30 (133 kN) to 50 kips (222 kN) load.

The method of evaluating existing pavements described is still tentative. Its validity needs further verification by field performance comparisons and from controlled tests.

CONCLUSIONS

Good correlations have been established between the observed performance of numerous accelerated traffic test pavements, under both single- and multiple-wheel gear loads, and the stresses and strains computed for these pavements by the nonlinear finite-element method. The limiting criteria are vertical strain on the subgrade surfaces, shear failure in unbound bases, and radial tensile strain in the asphaltic concrete layers.

Good correlations were obtained for pavements with subgrade soils of low and medium CBRs when the relation $E = 1,500 \text{ CBR}$ was used in the analysis, but such correlations were not obtained for pavements with subgrade soils with high CBRs.

The criteria developed are applicable to designing new pavements for new aircrafts and to evaluating pavement performance under mixed traffic and with overlays. With the limiting criteria, the thickness of each component layer of the pavement can be optimized, i.e., failure would initiate in the surface, base, and subgrade layers at the same coverage level.

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