Reliability of the Flexible Pavement Design Model

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The design of flexible pavements by the U.S. Army Corps of Engineers is currently based on the California bearing ratio (CBR) curve. The CBR curve is empirical, and the current design approach is deterministic. A probabilistic approach, providing more reliable designs at potentially lower costs, can be developed from the current design procedure if the reliability of the CBR curve is known. This study was undertaken to establish the reliability of the current CBR-based flexible pavement design model using existing data from accelerated traffic tests. The reliability of the design model was found to be about 50 percent, excluding the effects of conservative estimates of the design parameters.

The design of flexible pavements by the U.S. Corps of Engineers is currently based on the California bearing ratio (CBR) equation that was formulated in the 1950s, and extended in the 1970s, on the basis of the results of numerous full-scale accelerated traffic tests. These tests involved full-scale load carts operated on various test section pavements. Both highway vehicles and aircraft landing gears, with a wide variety of contact areas and tire pressures, were represented by the various load cart configurations. The test sections consisted of flexible pavements with many different thicknesses built on subgrades that had a wide range of strengths. The CBR equation is empirical and the design approach is deterministic. A unique pavement system based on a unique set of variables is designed. On the other hand, the design process can be approached probabilistically. This type of approach would allow the design engineer to account for uncertainty in the design variables and to accommodate material variability. The engineer can also ensure a low probability of premature failure, which is to say, a high reliability. Lower costs may be realized by reducing overconservatism in design in the form of excess wearing course, base, or subbase thickness or by reducing unrealistic estimates of pavement service life. The first step in implementing a probabilistic approach is to establish the accuracy or reliability of the basic design model as a predictor of pavement performance. This reliability is expressed in terms of the probability that the design model will correctly predict the performance of a particular pavement, given a particular set of design variables. However, because the CBR equation is based on a curve fit to the data using subjective engineering judgment, the reliability of this fit is uncertain. This constitutes a serious problem in implementing probabilistic methods in current U.S. Army pavement design procedures.

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REVIEW

The evolution of the flexible pavement design model can be traced through various references that describe the development of the CBR curve. The basic formulation is described in several U.S. Army Engineer Waterways Experiment Station (WES) technical memoranda (*1-3*), technical reports (*4, 5*), a miscellaneous paper (*6*), and instruction reports (*7, 8*) and work by others (*9*). The expansion of the CBR equation to include a term for a particular number of tires in a group is documented by Cooksey and Ladd (*10*) and Ahlvin et al. (*11*). This latter work also included the data generated by accelerated traffic tests with multiple-wheel loads in the late 1960s and early 1970s.

In its current form, the CBR equation is

$$t = \alpha\{A[(p/8.1\ CBR) - (1/\pi)]\}^{1/2} \quad \text{for } CBR/p < 0.22 \qquad (1)$$

where

t = pavement thickness (in.),
α = load repetition factor for particular tire group size as a function of traffic volume (discussed later),
A = contact area of one tire (in.^{2}),
p = equivalent single-wheel tire pressure (psi), and
CBR = strength of supporting material.

The curve has a graphic modification that can be described by the quadratic

$$t = \alpha(A\{0.05 - 0.35187\log(CBR/p) + 0.51492\,[\log(CBR/p)]^2\})^{1/2} \quad \text{for } CBR/p \geq 0.22 \qquad (2)$$

The CBR relationship has traditionally been depicted as in Figure 1 (*5*). These plots are characterized by large data scatter. This has been attributed to the effects of variations in the coverages required to produce failure. Here, failure is defined as attaining a maximum rut depth of 1 in., and a coverage is defined as a sufficient number of passes of the design vehicle to cover the entire traffic lane with at least one

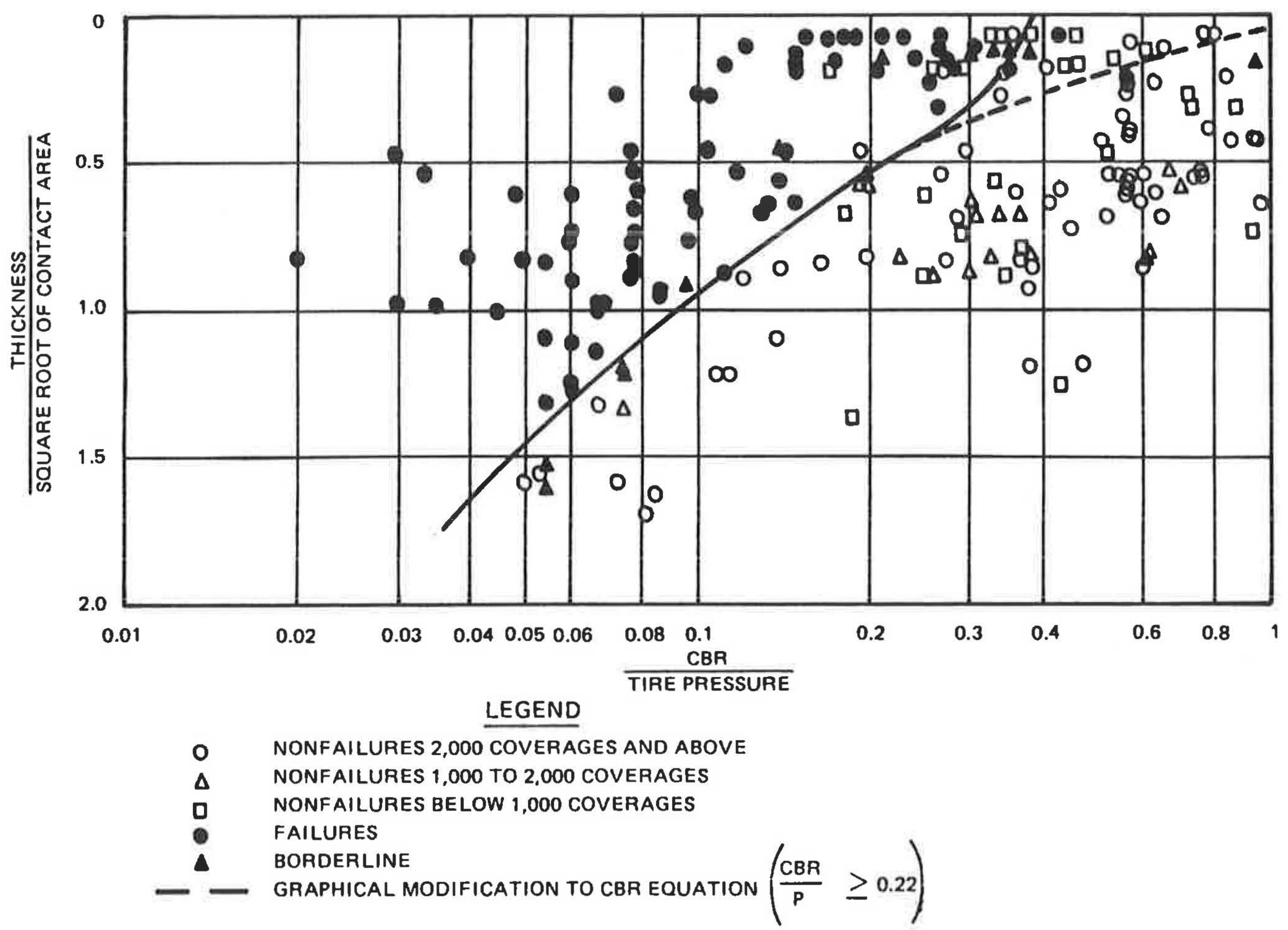

FIGURE 1 Curve from CBR formula compared with behavior data.

wheel load. Because the curve in Figure 1 passes essentially below and to the right of the failure data, it could be argued that the CBR relation is a conservative bound on actual behavior. However, plots such as Figure 1 are misleading in this respect. As noted in Technical Report 3-495 (*5*), the failure points that fall above and to the left of the curve are for coverage levels below 5,000. A review of the tabulated data in Technical Report 3-495 reveals that none of the "failures" shown in Figure 1 are for coverage levels above 5,000. Thus the appropriate conclusion is that the curve in Figure 1 represents the bound for failures (that is, the limit for satisfactory performance) occurring at coverage levels less than 5,000. From the position of the curve with respect to the coverage data, it would appear reasonable that the curve might also be close to the best fit for failure at 5,000 coverages. Variations in traffic volume were considered by adjusting the design thickness by an *f*-factor equal to 0.15 + 0.23 log *C*, where *C* is the total number of coverages of the design vehicle gear (*6*).

Ahlvin et al. (*11*) published an alternate CBR equation resulting from their best fit of a cubic equation to Equations 1 and 2. This equation is

$$t = \alpha(A^{1/2})\{-0.0481 - 1.1562 \log(CBR/p) - 0.6414[\log(CBR/p)]^2 - 0.4730[\log(CBR/p)]^3\} \quad (3)$$

The associated load repetition factor (α) curves, shown in Figure 2, were developed from the data in Table 1. This relationship allows consideration of variations in pass level, gear configuration, and vehicle wander. Here, pass level is defined as the number of movements (passes) of the design vehicle gear past a given point on the pavement. Such considerations are not possible with the basic relationship shown in Figure 1. The data were analyzed separately and weighted on the basis of differences in individual test objectives, failure criteria, methods of determining strength, frequency of field observations and measurements, construction techniques and materials, and methods of applying traffic. This reduced the effects of data scatter and is discussed in some detail by Ahlvin et al. (*11*).

As shown in Figure 3, the CBR curve (Equations 1 and 2) is essentially the same as the regression equation (Equation 3). This implies that the reliability of the two functions is equivalent. The U.S. Corps of Engineers uses these two relationships interchangeably (*8*).

In Technical Report 3-495 (*5*), the effect of multiple-wheel gears was recognized and the multiple-wheel data were reduced to equivalent single-wheel loads (ESWLs) for plotting on Figure 1. The technique for using elastic layer theory to compute the ESWL is described in detail by Ahlvin et al. (*11*). The equivalent single-wheel tire pressure (p) is obtained by dividing the ESWL by the contact area (A) of one tire. Later, Ahlvin et al. (*11*) developed the load repetition (α) factor

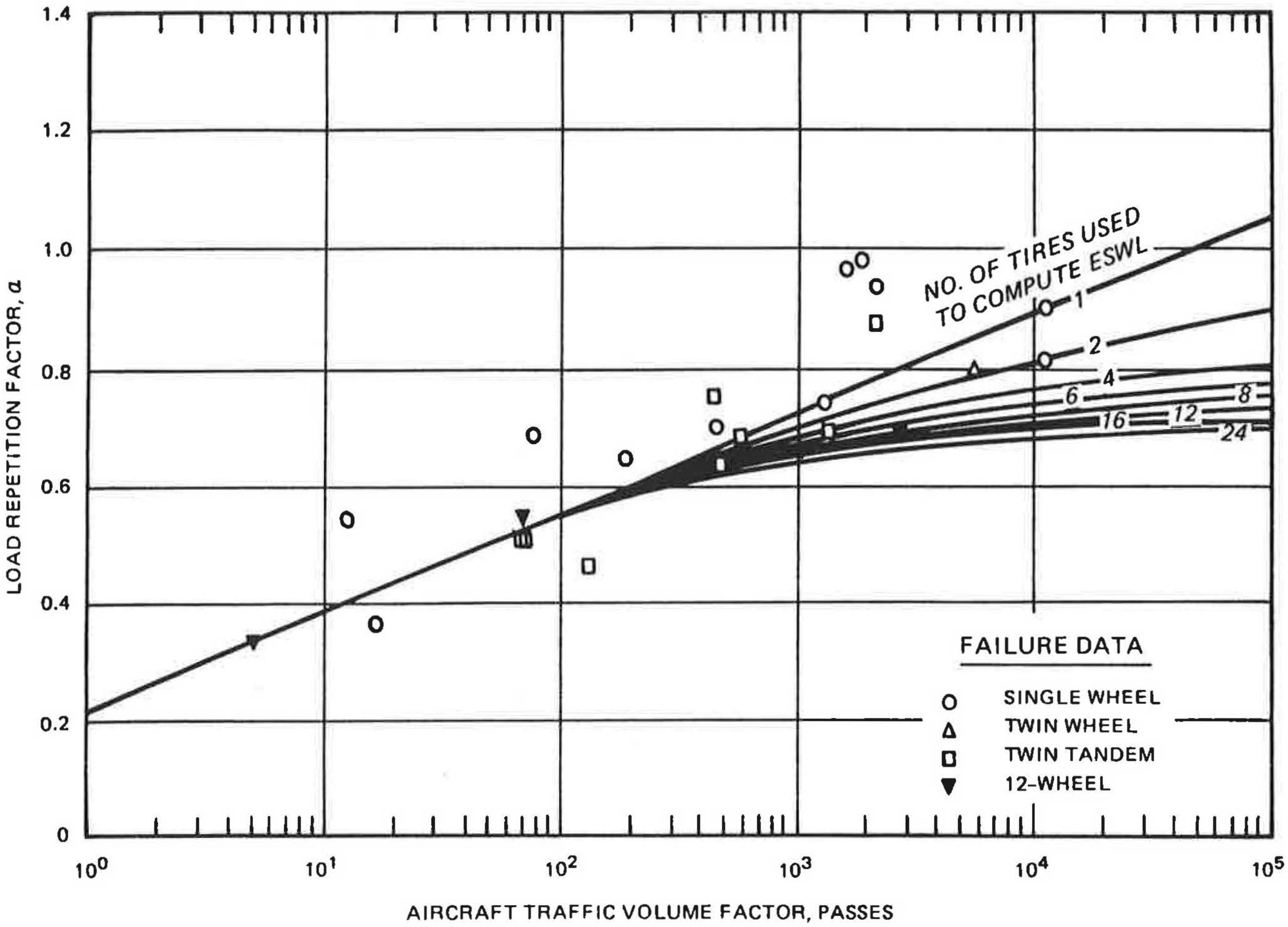

FIGURE 2 **Composite plot of load repetitions factor versus passes, after Ahlvin et al. (*11*).**

TABLE 1 SELECTED CBR FAILURE DATA

Reference	Gear Type	p(psi)	A(in.^{2})	t(in.)	CBR	C	α	t_{CBR} (in.)	t/t_{CBR}
12	Single	133.2	1,501	39.0	6.0	150	0.645	38.9	1.0025
		133.2	1,501	44.0	9.0	1,700	0.855	40.7	1.0811
		133.2	1,501	18.0	16.0	10	0.405	13.2	1.3616
		133.2	1,501	20.5	18.0	60	0.565	16.9	1.2135
		133.2	1,501	23.5	15.5	360	0.720	24.0	0.9774
		133.2	1,501	30.0	17.5	1,500	0.845	25.8	1.1622
		133.2	1,501	49.0	8.0	1,300	0.830	42.4	1.1559
13	Single	60.0	250	10.0	8.0	3,760	0.915	11.3	0.8867
		60.0	250	10.0	9.0	3,760	0.915	10.3	0.9729
11	Single	175.4	285	15.0	3.7	6	0.350	13.9	1.0790
		175.4	285	24.0	4.4	200	0.670	24.3	0.9889
		105.3	285	15.0	3.7	120	0.625	18.9	0.7955
14	B-29	126.7	330	10.0	20.0	2,000	0.805	10.0	1.0043
	B-36	241.6	262	14.0	16.0	1,000	0.710	14.3	0.9798
15	B-36	318.7	150	16.0	12.0	312	0.645	13.6	1.1772
		318.7	150	16.0	5.0	90	0.565	19.0	0.8415
		318.7	150	16.0	15.0	1,500	0.730	13.6	1.1789
11	B-747	430.3	290	33.0	3.8	40	0.510	32.1	1.0279
		430.3	290	33.0	4.0	40	0.510	31.3	1.0553
		496.6	290	41.0	4.0	280	0.640	42.2	0.9711
11	C-5A	218.5	285	15.0	3.7	8	0.345	15.4	0.9753
		272.8	285	24.0	4.4	104	0.550	25.2	0.9542
		330.9	285	33.0	3.8	1,500	0.670	36.5	0.9032
		330.9	285	33.0	4.0	1,500	0.670	35.6	0.9274

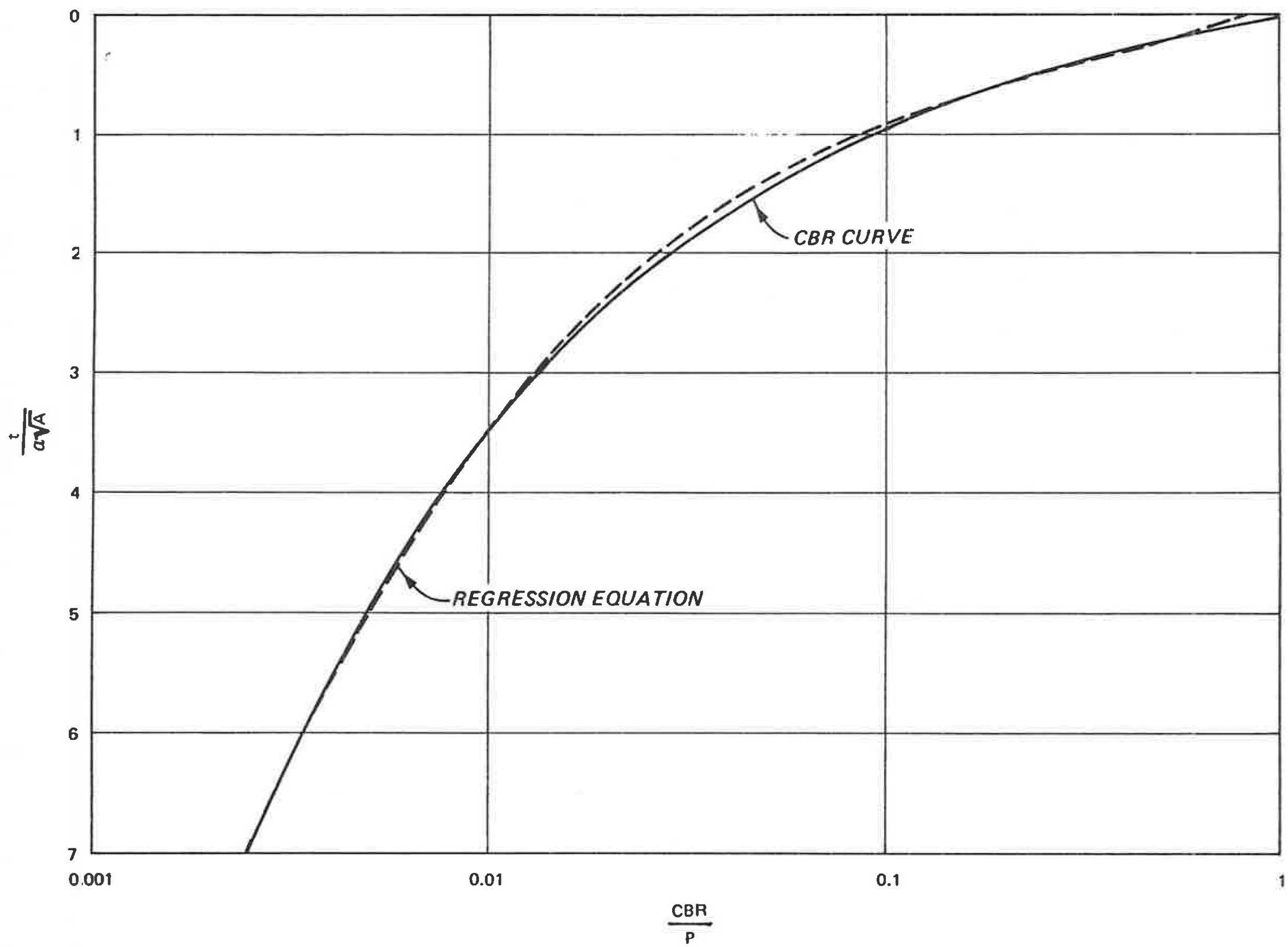

FIGURE 3 Comparison of the CBR curve and the regression equation (*11*).

shown in Equations 1, 2, and 3 to better account for the effects of multiple-wheel loading and to account for variations in traffic volume. Cooksey and Ladd (*10*) developed the α-factor curves shown in Figure 4 in terms of coverages. This is the form currently used by the Corps of Engineers, with Equations 1 and 2 or Equation 3. Because little full-scale accelerated traffic testing has been done since the multiple-wheel, heavy-gear load tests reported by Ahlvin et al. (*11*), these relationships consider essentially all available data.

The α-factor curves developed by Cooksey and Ladd (*10*) are based on the data in Table 1 plus the additional data given in Table 2. Of all available data, only the data in Tables 1 and 2 resulted from subgrade failures, consisted of only one loading condition or intensity, and represented pavements made of accepted construction materials. Only subgrade failures were considered because the thickness design procedure is based on protecting the subgrade from failure. Only those failures produced by one loading condition were considered to eliminate uncertainty introduced by assumptions about the effects of mixed traffic. The design of pavements for different coverage levels is done by changing thickness requirements instead of material requirements. Therefore only data from failures on material meeting quality standards were used.

ANALYSIS

A design thickness (t_{CBR}) can be computed from the CBR equation for each data point in Tables 1 and 2. For example, the first line in Table 1 is from a test section on a 6 CBR subgrade, subjected to 150 coverages of a 133.2-psi, 1,501-in.^{2} single-wheel load. From Figure 4, the α-factor is 0.645. The design thickness calculated using Equation 1 is 38.9 in. This design thickness can then be compared with the actual test section thickness (t) as shown in Figure 5 for all of the data. Note that all points lie close to the line of equality.

The ratio of the actual test section thickness to the design thickness can be used as a measure of correlation. In the previous example, the thickness ratio is 1.0025, meaning that the actual thickness was 0.25 percent greater than that predicted by the CBR equation. The value of this ratio, in general, can be viewed as a random variable, with the ratios tabulated in Tables 1 and 2 being samples from the population. The 28 thickness ratios in Tables 1 and 2 have a mean value (average) (μ) of 1.0053. The standard deviation (σ) is 0.1497.

The reliability of the CBR equation is the probability (P) that the actual test section thickness (t) is less than the design thickness (t_{CBR}). That is,

$$\text{Reliability} = \text{Probability}\ (t \leq t_{CBR}) = P\ [(t/t_{CBR}) \leq 1] \quad (4)$$

Assuming a normal distribution for the ratio of the thickness,

$$\text{Reliability} = \Phi(1 - \mu/\sigma) = \Phi(-0.035) = 48.6 \text{ percent} \quad (5)$$

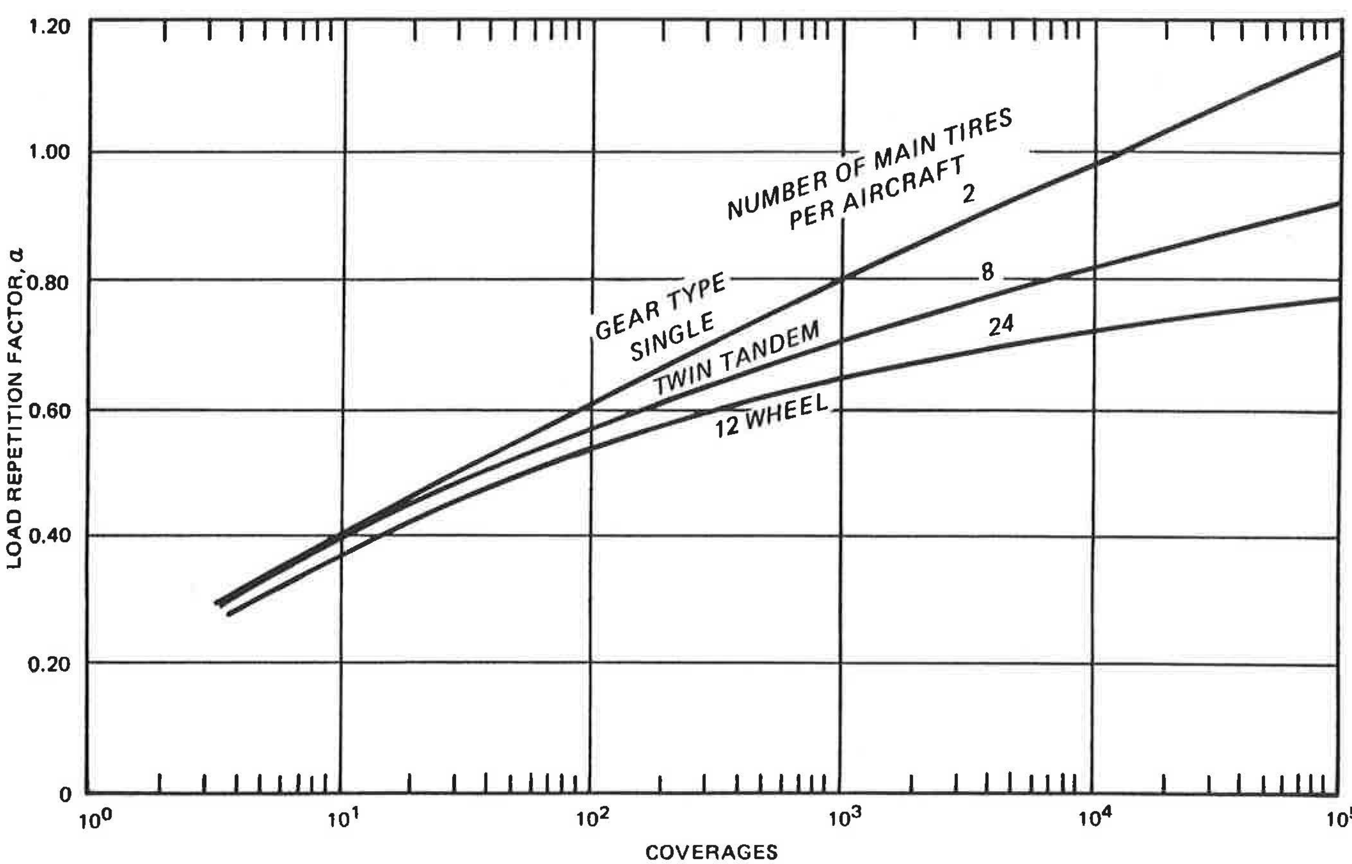

FIGURE 4 Flexible pavement thickness adjustment curves for various landing gears, after Cooksey and Ladd (*10*).

TABLE 2 ADDITIONAL CBR FAILURE DATA

Reference	Gear Type	p(psi)	A(in.^{2})	t(in.)	CBR	C	α	t_{CBR} (in.)	t/t_{CBR}
15	Single	200.0	150	12.0	14.0	216	0.680	10.0	1.1985
		200.0	150	12.0	7.0	178	0.665	14.6	0.8225
		200.0	150	12.0	6.0	203	0.675	16.1	0.7449
16	Single	109.9	91	5.0	6.0	40	0.530	7.0	0.7095

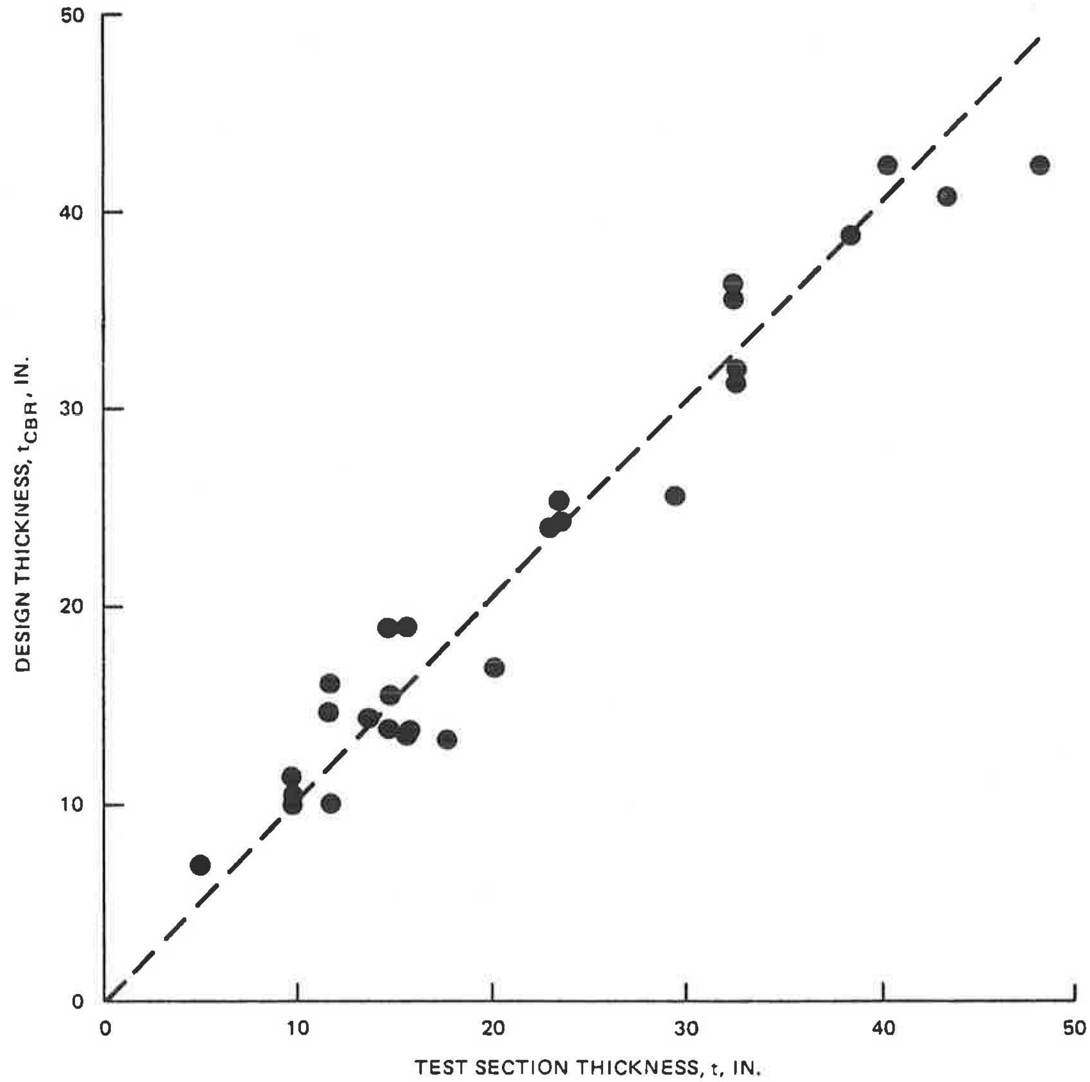

FIGURE 5 Test section thickness versus design thickness.

Because neither of the thicknesses nor their ratio can be negative, the ratio could be log normally distributed. In this case,

$$\text{Reliability} = \Phi(0 - \ln\{\mu/[1 + (\sigma^2/\mu^2)]^{1/2}\}/\{\ln[1 + (\sigma^2/\mu^2)]\}^{1/2}) = \Phi(0.038) = 51.5 \text{ percent} \qquad (6)$$

Finally, a beta distribution could be fitted to the data, using two other data statistics. The skewness of the data (β_1) (third moment about the mean) is 0.1416 and the kurtosis (β_2) (fourth moment about the mean) is 2.76. The beta distribution fitting these statistics has an alpha value of 7.14 and a beta value of 10.38. The minimum and maximum values of the variate are 0.432 and 1.806, respectively. For this distribution the reliability is

$$\text{Reliability} = P[(t/t_{CBR}) \leq 1] = 49.3 \text{ percent} \qquad (7)$$

CONCLUSIONS

The reliability of the U.S. Corps of Engineers flexible pavement design model is about 50 percent. This flexible pavement design model will therefore provide very nearly the best estimate or expected value of the pavement thickness required for the given design parameters, including the required service life. On the average, about one-half of all pavements designed using this model will fail before the design service life is reached and one-half will continue to perform beyond their design service life.

This reliability statement does not include the difference between the performance of the accelerated traffic test sections and the long-term performance of actual in-service pavements. Also not included are the effects of conservative estimates ("design" values) for the parameters for material strength (CBR), traffic load (p), and traffic intensity (α). Because the soaked CBR or the 15th percentile of the field CBR-values is often used for conservative design, for example, the reliability of the resulting pavement is much greater than 50 percent. This is consistent with the findings of long-term studies of in-service pavements: more than 50 percent are performing beyond their design life. Quantifying the effects on reliability of choosing conservative design values or of uncertainty in the true values of the design parameters is the subject of follow-on work at the Waterways Experiment Station (in publication) by Y. T. Chou.

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