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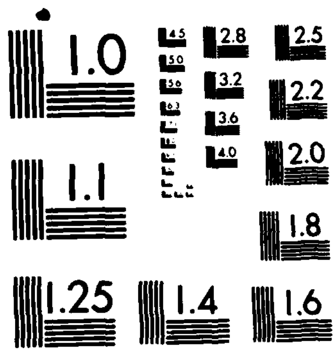

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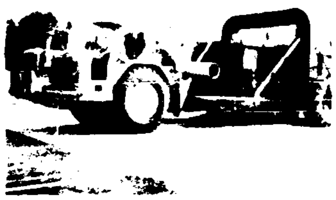

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RELIABILITY OF THE FLEXIBLE PAVEMENT DESIGN MODEL

by

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The design of flexible pavements by the US 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, except that the reliability of the CBR curve is uncertain. This study was undertaken to establish the reliability of the current CBR-based flexible pavement design model, using existing

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20. ABSTRACT (Continued).

data from accelerated traffic tests. The reliability of the design model was found to be 50 percent, excluding the effects of conservative estimates of the design parameters. Keywords:

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Preface

This study was conducted by the Geotechnical Laboratory (GL), US Army Engineer Waterways Experiment Station (WES) during the period February 1984 through September 1985. It was sponsored by the Assistant Secretary of the Army (R&D) under the In-House Laboratory Independent Research (ILIR) Program as Project No. 4A161101A91D, Task Area 02, Work Unit 160.

The study was conducted under the general supervision of Dr. W. F. Marcuson III, Chief, GL; Mr. H. H. Ulery, Jr., Chief, Pavement Systems Division (PSD); Mr. H. L. Green, Chief, Engineering Analysis Group; and Mr. D. M. Ladd, Chief, Criteria Development Unit. The study was conducted by Dr. John C. Potter, PSD, who is the author of the paper. A critical review of the paper was provided by Mr. R. G. Ahlvin, a consultant. This report was edited by Mr. Robert A. Baylot, Jr., Publications and Graphic Arts Division.

The Commanders and Directors of WES during this study were COL Tilford C. Creel, CE, and COL Robert C. Lee, CE; Technical Director was Mr. Fred R. Brown. During the publication of this report, COL Allen F. Grum, USA, was Director of WES; Dr. Robert W. Whalin was Technical Director.

Contents

Conversion Factors, Non-SI to SI (Metric) Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
pounds (force) per square inch	6.894757	kilopascals
square inches	6.4516	square centimetres

RELIABILITY OF THE FLEXIBLE PAVEMENT DESIGN MODEL

Background

1. The design of flexible pavements by the US Army Corps of Engineers is currently based on the California bearing ratio (CBR) equation which was formulated in the 1950's, and extended in the 1970's, based on 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, having 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 encompassing a wide range of strengths. The CBR equation is empirical and the design approach is deterministic. A unique pavement system is designed based on a unique set of variables. 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 reductions in 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 Army pavement design procedures.

Purpose

2. The purpose of this research effort is to briefly review the evolution of the CBR design curve and to evaluate the reliability of this flexible pavement design model. This review will provide a base for implementing probabilistic methods and for analyzing the reliability of the Army's flexible pavement designs.

Scope

3. Old and new flexible pavement test-section data were reviewed in relation to the development of the flexible pavement design model. The reliability of this model was then determined, based on this review.

Review

4. The evolution of the flexible pavement design model can be traced through various references which describe the development of the CBR curve. The basic formulation is described in several US Army Engineer Waterways Experiment Station (WES) Technical Memoranda (WES 1948, 1951a, 1955), Technical Reports (WES 1956, 1959a), a Miscellaneous Paper (WES 1951b), and Instruction Reports (WES 1959b, Pereira 1977) and work by others (Kerkhoven and Dorman 1953). 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 (1971) and Ahlvin et al. (1971). This latter work also included the data generated by accelerated traffic tests with multiple-wheel loads in the late 1960's and early 1970's.

5. In its current form, the CBR equation is

$$t = \alpha\sqrt{A\left(\frac{p}{8.1\ \mathrm{CBR}} - \frac{1}{\pi}\right)} \quad \text{for } \frac{\mathrm{CBR}}{p} < 0.22 \tag{1}$$

where

- t = pavement thickness, in.
- α = load repetition factor for particular tire group size as a function of traffic volume (discussed in paragraph 9)

A = contact area of one tire, sq in.

p = equivalent single-wheel tire pressure, psi

CBR = strength of supporting material

The curve has a graphical modification which can be described by the quadratic

$$t = \alpha\sqrt{A\left\{0.05 - 0.35187 \log\left(\frac{CBR}{p}\right) + 0.51492\left[\log\left(\frac{CBR}{p}\right)\right]^2\right\}} \qquad (2)$$

$$\text{for } \frac{CBR}{p} \geq 0.22$$

6. The CBR relationship has traditionally been depicted as in Figure 1 (WES 1959a). These plots are characterized by large data scatter. This has been attributed to the effects of variations in the coverages required to produce failure, from the basic level of 5,000, assumed for design. 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 the actual behavior. However, plots such as Figure 1 are misleading in this respect. As noted in Technical Report 3-495 (WES 1959a), the failure points falling 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.

7. Ahlvin et al. (1971) published an alternate CBR equation resulting from their best fit of a cubic equation to Equations 1 and 2. This equation is:

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

This version of the CBR relation is shown in Figure 2. The associated load repetition factor (α) curves, shown in Figure 3, were developed from the data shown in Table 1. This relationship allows consideration of variations in pass level,* gear configuration and vehicle wander. Such considerations are not possible with the basic relationship shown in Figure 1. Previously, 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 (WES 1951b). The data were analyzed separately and weighted based on 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. (1971).

8. As shown in Figure 4, 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. In fact, the US Army Corps of Engineers uses these two relationships interchangeably (Pereira 1977). The new data included by Ahlvin et al. (1971) and by Cooksey and Ladd (1971) did not change the reliability of the basic CBR relationship. Rather, the new data were used to adjust the load repetition factor curves to obtain the best fit to the actual performance data. This is why the original CBR equations (Equations 1 and 2) and the regression equation (Equation 3) can be essentially the same curves, in spite of the fact that additional data were used in developing Equation 3 and the associated load repetition factor curves.

9. In Technical Report 3-495 (WES 1959a), the effect of multiple-wheel gears was recognized and the multiple-wheel data were reduced to equivalent single-wheel loads (ESWL) for plotting on Figure 1. Later, Ahlvin et al. (1971) developed the load repetition (α) factor 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 (1971) developed the α-factor

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* The pass level is defined as the number of movements (passes) of the design vehicle gear past a given point on the pavement.

** A coverage is defined as a sufficient number of passes of the design vehicle gear to cover the entire traffic lane with at least one wheel load.

curves shown in Figure 5 in terms of coverages. This is the form currently used by the Corps of Engineers, with either 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. (1971), these relationships consider essentially all available data.

10. The α-factor curves developed by Cooksey and Ladd (1971) are based on the data shown in Table 1, plus the additional data shown 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 using accepted construction materials. Only subgrade failures were considered since the thickness design procedure is based upon 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 rather than material requirements. Therefore, only data from failures on materials meeting quality standards were used.

Analysis

11. If Equation 3 (Ahlvin et al. 1971) and the α-factor curves (Cooksey and Ladd 1971) currently used for flexible pavement design by the Corps of Engineers are best-fit representations of the performance data, then the curves pass through the expected value of the dependent variable (that is, $t/\sqrt{A}$) for any values of the independent variables. In this case, the current criteria represent the 50^{th} percentile of the data, and have a reliability of 50 percent. This conclusion is easily verified by plotting the data from Tables 1 and 2, using α-factors from Figure 5, on a graph of Equations 1 and 2 or Equation 3. As shown in Figure 6, these data are evenly distributed along either side of a plot of Equation 3. The conclusion that the current Corps of Engineers flexible pavement design model has a reliability of about 50 percent is therefore confirmed.

Conclusions

12. The reliability of the flexible pavement design model is about 50 percent. The 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 effects of conservative estimates ("design" values) for the parameters for material strength (CBR), traffic load (p), and traffic intensity (α). The effects of uncertainty or conservatism in these estimates are being addressed by the RDTE Program Project No. 4A161102AT22, Task AO, Work Unit 09, "Methodology for Considering Material Variability in Pavement Design," and will be documented in a technical report to be entitled "Probabilistic Analysis of the CBR Design Method for Flexible Airfield Pavements."

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Table 1

Selected CBR Failure Data (After Ahlvin et al. 1971)

Source	Type Assembly	Wheel Spacing in.	Assembly Load kips	Tire Contact Area, sq in.	Thickness above Subgrade, in.	CBR	Coverages at Failure
O. J. Porter and Co. (1948)	Single	--	200	1,501	39.0	6.0	150
			200	1,501	44.0	9.0	1,700
			200	1,501	18.0	16.0	10
			200	1,501	20.5	18.0	60
			200	1,501	23.5	15.5	360
			200	1,501	30.0	17.5	1,500
			200	1,501	49.0	8.0	1,300
WES (1947)	Single	--	15	250	10.0	8.0	3,760
			15	250	10.0	9.0	3,760
Ahlvin et al. (1971)	Single	--	50	285	15.0	3.7	6
			50	285	24.0	4.4	200
			30	285	15.0	3.7	120
WES (1952)	Twin	37	70	330	10.0	20.0	2,000
	Twin tandem	31 × 63	150	262	14.0	16.0	1,000
WES (1950)	Twin tandem	31-1/2 × 60	120	150	16.0	12.0	312
			120	150	16.0	5.0	90
			120	150	16.0	15.0	1,500
Ahlvin et al. (1971)	Twin tandem	44 × 58	240	290	33.0	3.8	40
			240	290	33.0	4.0	40
			240	290	41.0	4.0	280
	12-wheel	C-5A gear	360	285	15.0	3.7	8
			360	285	24.0	4.4	104
			360	285	33.0	3.8	1,500
			360	285	33.0	4.0	1,500

Table 2

Selected CBR Failure Data (After Cooksey and Ladd 1971)

Source	Type Assembly	Wheel Spacing in.	Assembly Load kips	Tire Contact Area, sq in.	Thickness above Subgrade, in.	CBR	Coverages at Failure
WES (1950)	Single	--	30	150	12.0	14.0	216
			30	150	12.0	7.0	178
			30	150	12.0	6.0	203
WES (1962)	Single	--	10	91	5.0	6.0	40

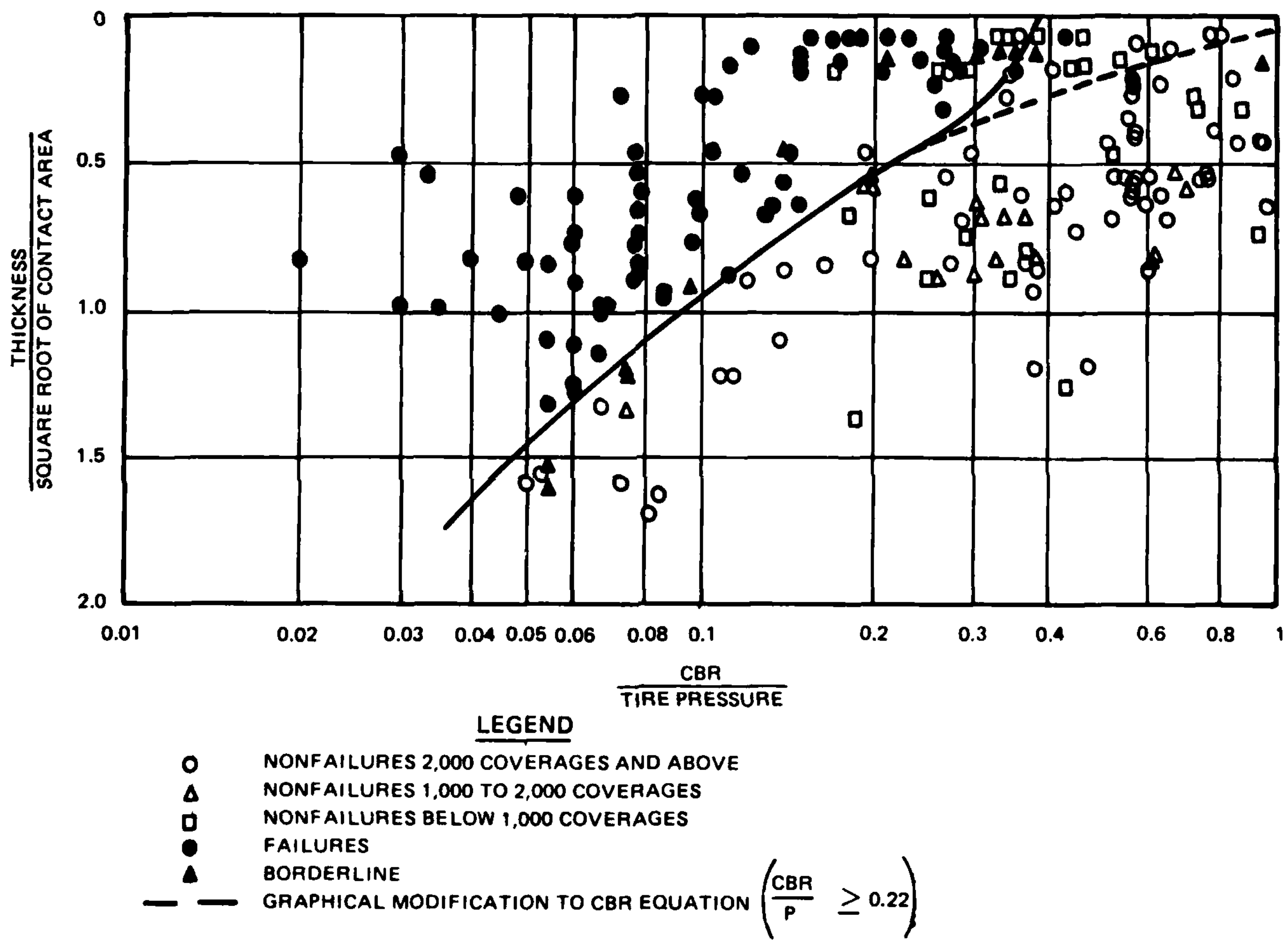

Figure 1. Curve from CBR formula compared to behavior data (after WES 1959a)

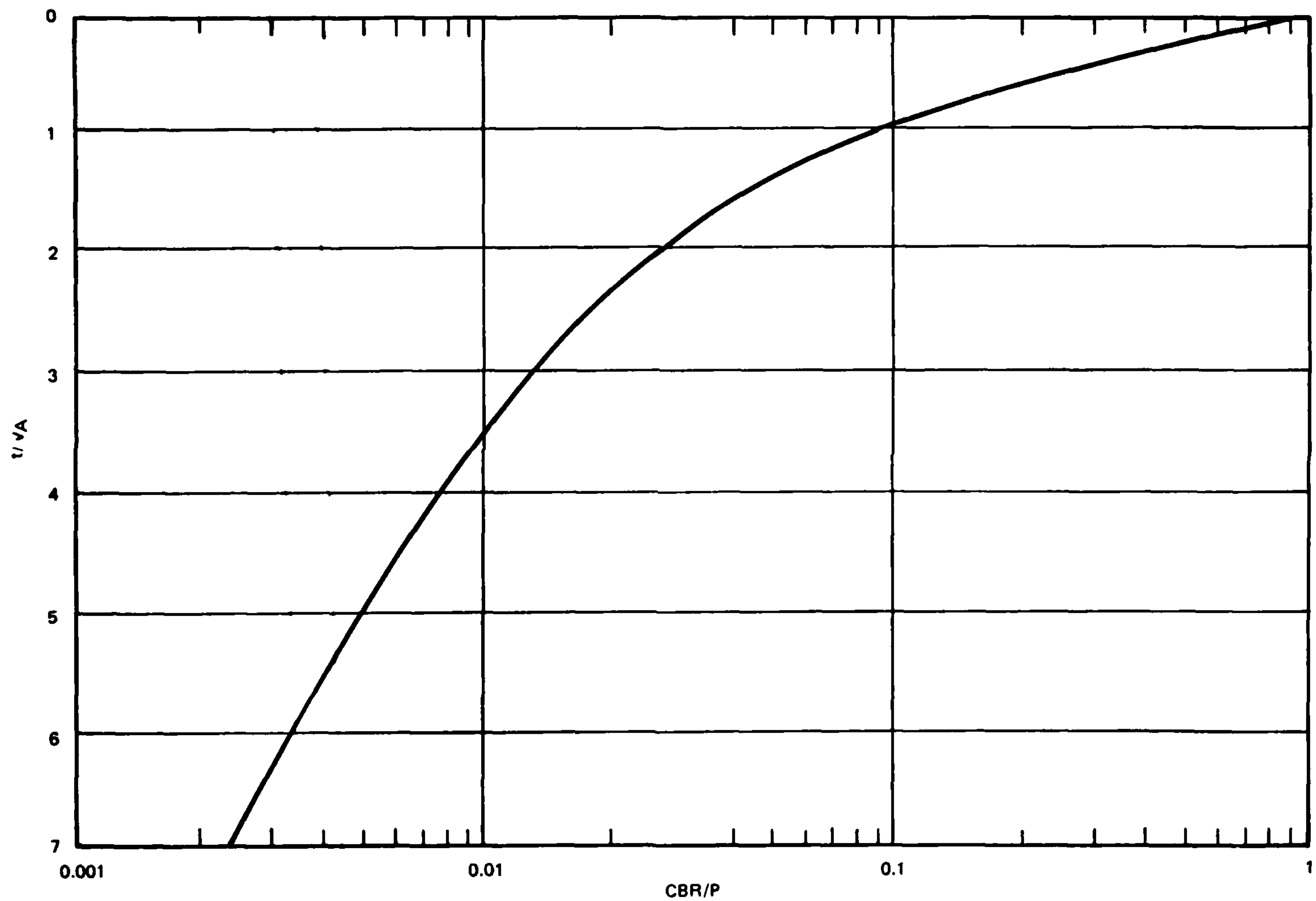

Figure 2. $t/\sqrt{A}$ versus CBR/p

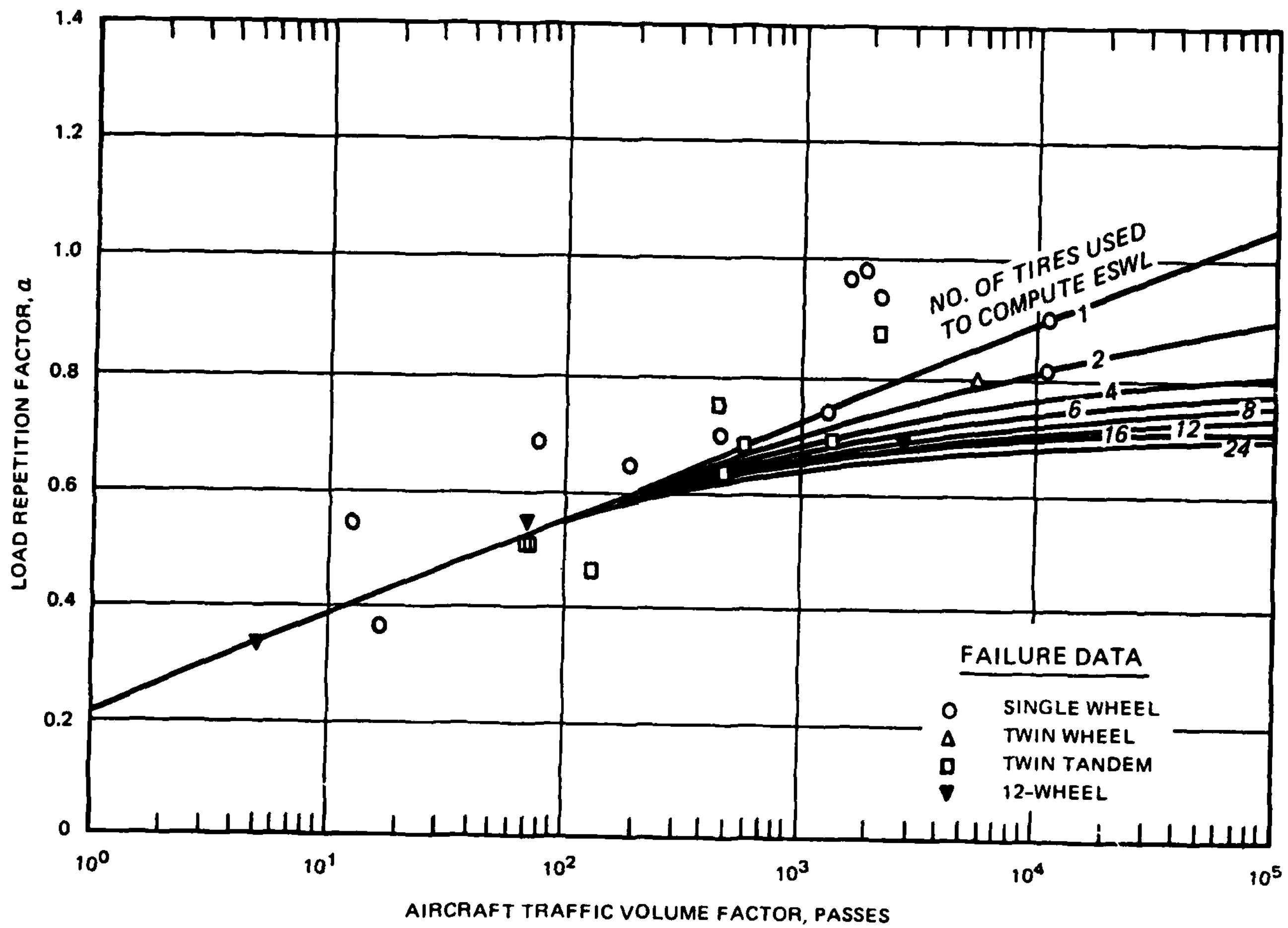

Figure 3. Composite plot of load repetition factors versus passes (after Ahlvin et al. 1971)

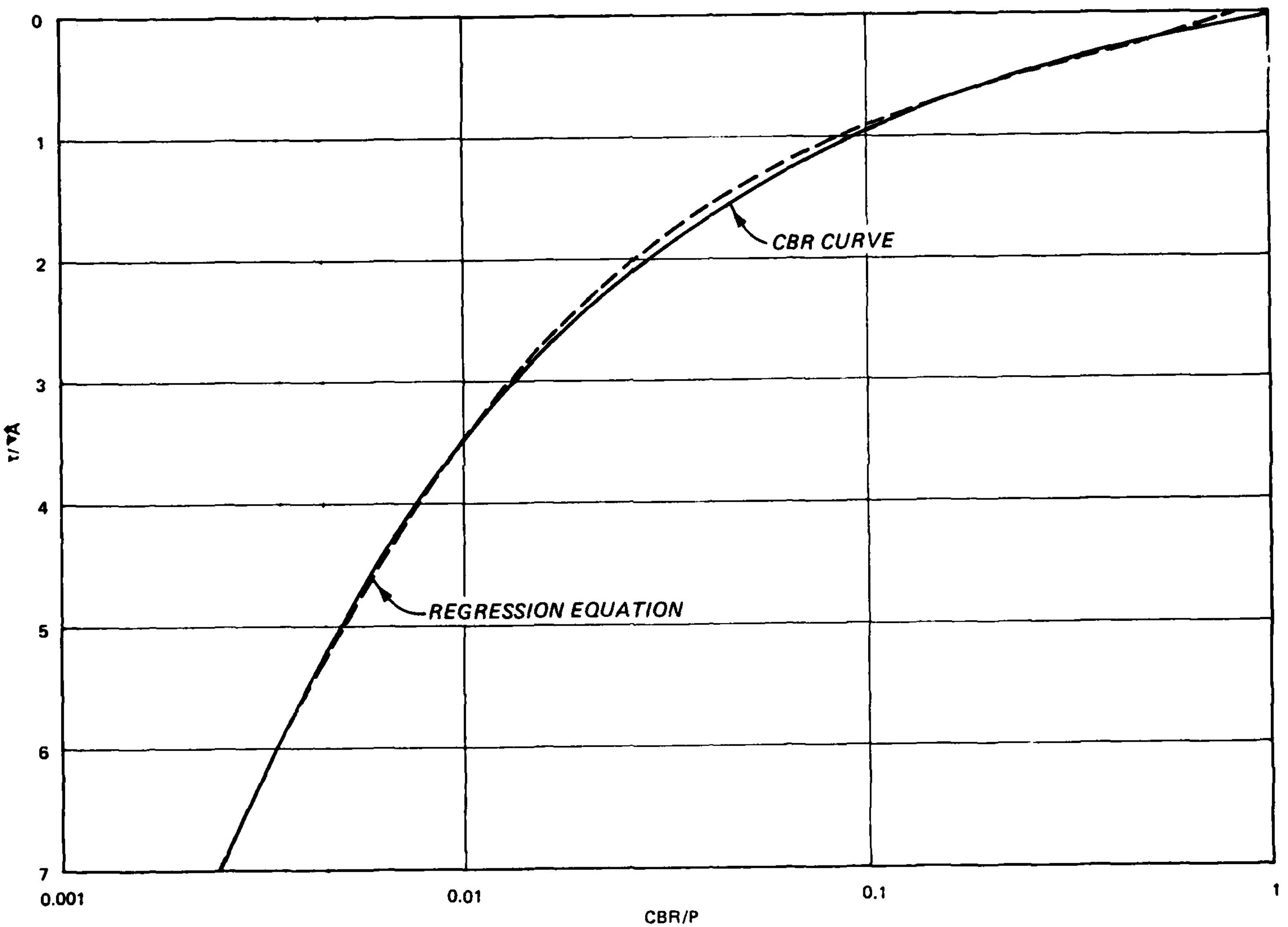

Figure 4. Comparison of the CBR curve and the regression equation by Ahlvin et al. (1971)

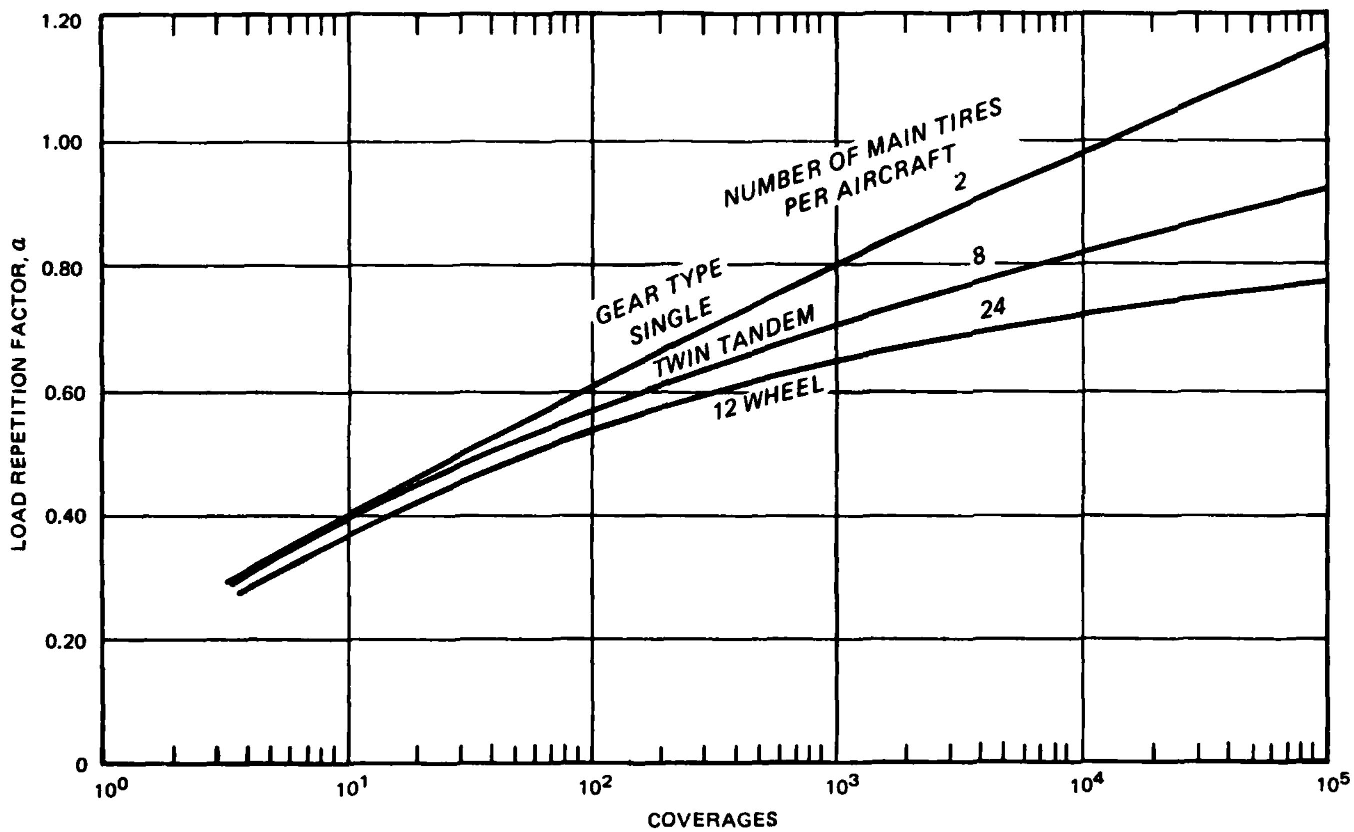

Figure 5. Flexible pavement thickness adjustment curves for various landing gears (after Cooksey and Ladd 1971)

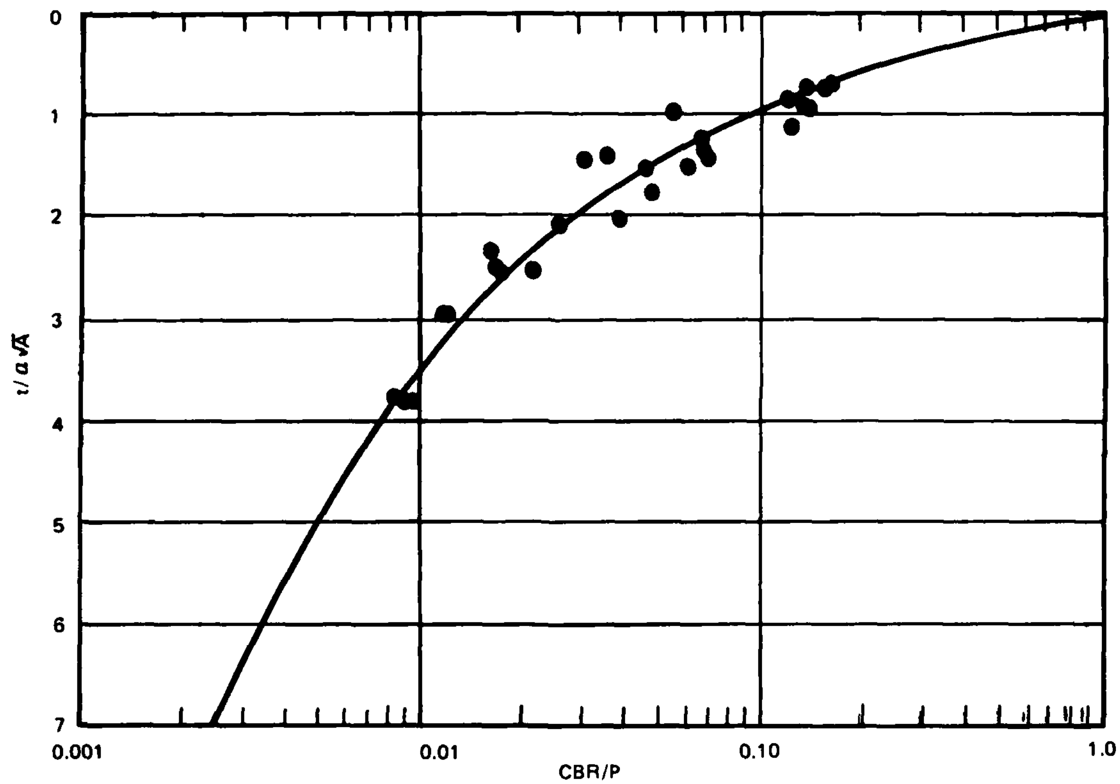

Figure 6. Complete $t/\alpha\sqrt{A}$ versus CBR/p