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TRAFFIC EVALUATION FOR

SECONDARY AND LOCAL PAVEMENT DESIGN

by

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A Phase of
Research Project IHR-28
AASHO Road Test

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in cooperation with
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16. Abstract

The relative reliability of the vehicle equivalency factors used in the design of secondary and local roads was studied in an effort to determine more realistic values. The study included the evaluation of classification count data, the determination of the potential effect of seasonal traffic fluctuations, a study of the performance equation and method of traffic conversion, and an analysis of actual pavement performance data. The classification data and pavement performance data indicated that the factors used for Class III roads (400<ADT<1,000) are reasonable for both Class III and Class IV (ADT<400) roads.

Seasonal traffic fluctuations were found to have a potentially significant lowering effect on the average vehicle equivalency factors, however, not to values as low as those currently used for Class IV road design. Nevertheless, retention of the current Class IV factors was recommended on the basis of past experience, possible design philosophy differences, and the questionable applicability of the Present Serviceability Index and performance equation to these pavements.

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SUMMARY

When the Illinois pavement design procedures were developed, no truck weight data and only a limited amount of vehicle classification data were available from secondary and local roads. As a result the vehicle equivalency factors used in traffic analyses for the design of these roads were based on extrapolations of data from primary highways and assumptions that produced pavement design requirements consistent with past experience. The current study was conducted to determine the relative reliability of these vehicle equivalency factors and, if possible, to establish more realistic values.

Truck weight data from local and secondary roads were still unavailable. However, sufficient classification data were available to compute VEFs for both Class III (400<ADT<1,000) and Class IV (ADT<400) roads while using the truck weight data from primary highways. This produced the VEFs that were previously recommended for use in the design of flexible and rigid pavements on Class III roads and rigid pavements on Class IV roads (3). However, the flexible pavement VEFs for Class IV roads from this analysis were considered unrealistic since their use would require heavier pavement designs than are deemed necessary.

To determine other elements that influence the value of the vehicle equivalency factors, the effect of seasonal traffic fluctuations was evaluated and the AASHO Road Test performance equation and the method of converting mixed traffic to an equivalent single loading were studied. Seasonal traffic fluctuations were found to have a potentially significant effect on the average value of the vehicle equivalency factors. However, the effect is not sufficient to explain the difference between the currently used Class IV road VEFs and the ones determined from classification data. The study of the performance equation and method of traffic conversion,

while not producing evidence that could be used to verify or adjust the VEFs, provided considerable insight into the nature of the performance equation and demonstrated that for typical Class IV pavement designs the conversion of mixed traffic can produce questionable results.

Finally, performance data from various county highways were analyzed. This showed that the Class III road VEFs provided a reasonable estimate of the effect of mixed traffic on both Class III and Class IV roads. Nevertheless, for Class IV flexible pavement design, retention of the original VEFs has been recommended on the basis of past experience, possible differences in design philosophy between primary and secondary pavement designers and the questionable applicability of the Present Serviceability Index and the performance equation as extended in the Illinois design procedure to the design of Class IV roads.

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INTRODUCTION

From October 15, 1958 to November 30, 1960, the AASHO Road Test was conducted at a site near Ottawa, Illinois. From the data collected during this test, empirical equations were developed which predict, with a reasonable degree of accuracy, the number of axle load applications a test pavement withstood before its serviceability dropped below any level. These equations were subsequently modified by the Illinois Division of Highways and developed into procedures for designing flexible and rigid pavements (1) (2).

A key factor in these procedures is the evaluation of predicted future traffic in terms of equivalent 18-kip single-axle load applications. This evaluation combines the predicted volumes of passenger cars, single unit trucks and buses, and multiple unit trucks with average 18-kip single-axle load equivalency factors for each vehicle type (hereafter called vehicle equivalency factors or VEF) which were established from historical truck weight and vehicle classification count data.

While sufficient data were available when the procedures were developed to establish the VEFs for trucks and buses on the Interstate and primary highway systems with an acceptable degree of accuracy, no truck weight data and little vehicle classification count data existed for the secondary and local roads. As a result, the VEFs for use in designing these roads were based on extrapolations and estimates that appeared to give reasonable results when used with the design procedures. For the Class III roads ($400<ADT<1,000$) sufficient classification data were available that VEFs were developed using primary highway data adjusted in accordance with the vehicle classification data. However, the amount of classification data available from the lower volume Class IV roads ($ADT<400$) were very limited. For this reason, the VEFs for Class IV roads were established from an analysis of assumed structural designs and traffic conditions which were consistent with previous experience.

The current study was conducted to determine the relative reliability of these factors and, if possible, to establish more realistic values. Truck weight data were still unavailable and, due to the time that would be required to obtain a sufficiently large, representative sample, collection of such data was not considered to be economically feasible. However, a deeper study of the limited available data and the nature of the empirical AASHO Road Test equations was expected to produce more reliable estimates of the true VEFs.

Because most secondary and local roads are flexible pavements, and because VEFs play a more critical role in the design of flexible pavements (3), the study concentrated on the vehicle equivalency factors for flexible pavement design with the intention of applying any significant finding to the factors for rigid pavements as well.

WEIGHT AND CLASSIFICATION DATA ANALYSIS

The analysis of statewide truck weight and classification count data to establish single-unit and multiple-unit VEFs has been discussed in detail elsewhere (3). Basically, this analysis involved two steps. First, the average equivalency factors for individual vehicle types (panels and pickups, two-axle four-tired trucks, two-axle six-tired trucks, etc.) were determined from statewide truck weight data by multiplying the number of axles weighed in each weight range by the axle load equivalency factor for that range, summing these products, and dividing by the total number of vehicles weighed. Secondly, these individual factors were combined in accordance with vehicle distributions obtained in vehicle classification counts to establish single-unit and multiple-unit VEFs. An example of these calculations for single-unit trucks is shown in Table 1.

TABLE 1

EXAMPLE VEF CALCULATIONS - SINGLE-UNIT TRUCKS
Flexible Pavement, p = 2.0

SINGLE-UNIT TRUCKS - 1969 OTHER MAIN RURAL DATA									
Axle Load Range	18 kip Equiv. Factor	Axles Weighed				18 kip Eq. Fact. x Axles			
		Panel & Pickup	2 Axle 4 Tire	2 Axle 6 Tire	3 Axle	Panel & Pickup	2 Axle 4 Tire	2 Axle 6 Tire	3 Axle
SINGLE									
<8 kip	0.0061	648	128	530	67	3.9528	0.7808	3.2330	0.4087
8-12	0.1750			82	41			14.3500	7.1750
12-16	0.6017			55	18			33.0935	10.8306
16-18	1.0000			20	1			20.0000	1.0000
18-20	1.5800			3				4.7400	
Total Weighed		648	128	690	127	3.9528	0.7808	75.4165	19.4143
Total Counted		16,324	908	5,590	982				
TANDEM									
<12 kip	0.0133				42				0.5586
12-18	0.0750				21				1.5750
18-24	0.2417				14				3.3838
24-30	0.6283				31				19.4773
30-32	0.8267				14				11.5738
32-34	1.0733				5				5.3665
Total Weighed					127				41.9350
Total Counted					982				
Trucks Weighed		324	64	345	127				
Trucks Counted		8,162	454	2,795	982				

Panel & Pickup Factor = 3.9528 ÷ 324 = 0.0122

2 Axle 4 Tire Factor = 0.7808 ÷ 64 = 0.0122

2 Axle 6 Tire Factor = 75.4165 ÷ 345 = 0.2186

3 Axle Factor = (19.4143 + 41.9350) ÷ 127 = 0.4831

$$\text{Single-Unit Factor} = \frac{(0.0122 \times 8162) + (0.0122 \times 454) + (0.2186 \times 2795) + (0.4831 \times 982)}{8162 + 454 + 2795 + 982}$$

= 0.096 equivalent 18k S.A.L. applications

This procedure worked well in establishing VEFs for the Interstate and primary highway systems where both truck weight and classification count data were available. However, for secondary and local roads, the approach was not adequate since no truck weight data had been collected from these roads. Consequently, extrapolations and assumptions based on the available data had to be employed. The initial assumption was that the axle weight distributions for individual vehicle types are the same on secondary and local roads as they are on the primary highway system. With this assumption, the results of the first step in the analysis of the primary highway data could be combined with the classification count data from secondary and local roads to determine VEFs. However, adequate classification count data were not available for both classes of these roads.

For flexible pavement design purposes, secondary and local roads in Illinois fall primarily in two roadway classifications -- Class III roads, those with estimated ADT volumes between 400 and 1,000, and Class IV roads, those with estimated ADT volumes below 400. When the design procedures were being developed, sufficient classification data for determining VEFs by the method described above were available only for the Class III roads. This produced Class III vehicle equivalency factors of 0.098 for single units and 0.794 for multiple units. However, without adequate data, VEFs for the Class IV roads had to be assumed.

Initially, the Class III factors were thought to be reasonable assumptions for Class IV road design. However, these values were found to produce designs that appeared to be unrealistically heavy. Subsequently, VEFs for Class IV roads were developed by working backward through the design equations from known structural designs that service experience had proved to be adequate. From this procedure, VEFs of 0.027 for single units and 0.216 for multiple units were adopted for design.

Following the adoption of the design procedures, the collection and analysis of axle weight data continued annually until sufficient data were available to warrant revising the original VEFs. At that time additional classification data were also available, including sufficient Class IV road data to allow computation of both Class III and Class IV vehicle equivalency factors. These computations are shown in Tables 2 and 3. As before, the Class IV factors were found to produce flexible pavement designs that appeared to be unrealistic. Consequently, for lack of a more realistic process, retention of the original Class IV vehicle equivalency factors was recommended pending the results of the present study (3).

SEASONAL WEIGHTING FACTORS

As the current study was being initiated, it was recognized that additional variables not considered in the earlier analyses might exist which would significantly influence the actual value of the VEFs. In particular, seasonal traffic fluctuations, which could have a profound impact on pavement performance, had never been evaluated.

During the AASHO Road Test, 80 percent of the flexible pavement failures occurred during the spring of the year. As a result, pavement performance was found to correlate better with the numbers of load applications if they were modified by a seasonal weighting function (4). This function was incorporated into the Illinois flexible pavement design procedures by using the performance equation developed from the seasonally weighted load applications. However, because the truck weight and classification count data were available only on a total year basis, the seasonal weighting function could not be incorporated into the data analyses and no procedure was available for including this factor in the pavement design analyses. For primary highways this is not considered to be a serious omission. However, with springtime load restrictions posted on some local roads and the increased truck traffic in rural areas during the harvest season, the effect could be significant for local and secondary roads.

TABLE 2

VEF ADJUSTMENTS FOR CLASS III AND CLASS IV ROADS

SINGLE UNITS
FLEXIBLE PAVEMENT

Year	Road Class	Percent of Single Units 2-Axle	Percent of Single Units 3-Axle	Equivalency Factor 2-Axle	Equivalency Factor 3-Axle	Single-Unit Equivalency Factor 1/
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1963	III	96.3	3.7	0.080	0.281	0.087
	IV	95.4	4.6	0.080	0.281	0.089
1964	III	94.1	5.9	0.070	0.363	0.087
	IV	95.6	4.4	0.070	0.363	0.083
1965	III	96.3	3.7	0.088	0.299	0.096
	IV	95.4	4.6	0.088	0.299	0.098
1966	III	94.3	5.7	0.066	0.474	0.089
	IV	94.7	5.3	0.066	0.474	0.088
1967	III	96.2	3.8	0.066	0.432	0.080
	IV	86.3	13.7	0.066	0.432	0.116
1968	III	93.0	7.0	0.066	0.502	0.097
	IV	91.6	8.4	0.066	0.502	0.103
1969	III	95.9	4.1	0.063	0.488	0.080
	IV	86.9	13.1	0.063	0.488	0.119
Average (1963-1969)	III					0.088
	IV					0.099

1/ $$\text{S.U.E.F.} = \frac{(\text{Col. 3} \times \text{Col. 5}) + (\text{Col. 4} \times \text{Col. 6})}{100}$$

TABLE 3

VEF ADJUSTMENT FOR CLASS III AND CLASS IV ROADS

MULTIPLE UNITS
FLEXIBLE PAVEMENT

Year	Road Class	Percent of Multiple Units 3-Axle	4-Axle	5-Axle	Equivalency Factor 3-Axle	4-Axle	5-Axle	Multiple-Unit Equivalency Factor 1/
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1963	III	19.5	48.7	31.8	0.685	0.880	0.836	0.828
	IV	28.1	52.1	19.8	0.685	0.880	0.836	0.817
1964	III	13.9	42.8	43.3	0.674	0.864	0.854	0.833
	IV	20.4	32.4	47.2	0.674	0.864	0.854	0.821
1965	III	12.8	36.9	50.3	0.749	0.873	0.893	0.867
	IV	11.7	32.0	56.3	0.749	0.873	0.893	0.870
1966	III	9.1	35.3	55.6	0.635	0.743	0.847	0.791
	IV	27.4	13.7	58.9	0.635	0.743	0.847	0.775
1967	III	7.6	21.5	70.9	0.664	0.775	0.935	0.880
	IV	0.0	7.6	92.4	0.664	0.775	0.935	0.923
1968	III	8.9	20.5	70.6	0.669	0.724	0.917	0.855
	IV	8.8	15.8	75.4	0.669	0.724	0.917	0.865
1969	III	9.1	22.3	68.6	0.605	0.707	0.913	0.839
	IV	4.3	7.1	88.7	0.605	0.707	0.913	0.886
Average (1963-1969)	III							0.842
	IV							0.851

1/ $\text{M.U.E.F.} = \frac{(\text{Col. 3} \times \text{Col. 6}) + (\text{Col. 4} \times \text{Col. 7}) + (\text{Col. 5} \times \text{Col. 8})}{100}$

As a beginning point in determining the potential effect of this factor, the current policies for posting spring load limits were studied. However, with each County in Illinois establishing its own policy, often on a road-by-road basis, no general or predominant policy was found. Consequently, it was decided to study the ultimate possible effect of a 90-day load limit. For the ultimate effect the load limit was assumed to be that no trucks would be allowed during the three spring months.

Average monthly weighting factors were developed by sorting the individual factors used at the Road Test according to the month in which they were used and computing the average value. These averages were subsequently normalized to an average value of one. This was necessary so that an individual vehicle making a single trip each day would not be evaluated as more than 365 total weighted vehicle load applications per year. These calculations are shown in Table 4.

To determine the effect of the assumed "ultimate load limit," the vehicle equivalency factors for Class IV roads computed on Tables 2 and 3 were adjusted by the ratio of 5.71 (the sum of the June through February monthly weighting factors) to 12. This resulted in a single-unit factor of 0.047 and a multiple-unit factor of 0.405. While these values were still almost double the currently used factors for Class IV roads, they nevertheless demonstrated that seasonal traffic variations and spring load limits can have a significant impact on the average annual VEFs.

PERFORMANCE EQUATION AND TRAFFIC CONVERSION

While additional factors which could have a significant effect on the value of the VEFs for secondary and local roads (especially Class IV roads) were not immediately obvious, a plausible approach for identifying such factors was to conduct a detailed study of both the AASHO Road Test Performance equation and the method used to convert mixed traffic to an equivalent single loading.

TABLE 4. Monthly Seasonal Weighting Factor Computations

Month	AASHO Road Test Seasonal Weighting Factors [1]									Average	Normalized Monthly Factors [2]
	1958			1959			1960				
January				0.00	0.00		0.16	0.28		0.11	0.09
February				0.11	1.28		0.44	0.22		0.51	0.42
March				4.55	4.84		0.16	1.60		2.79	2.30
April				4.27	2.35		4.27	3.48		3.59	2.95
May				1.78	1.28		1.28	0.75		1.27	1.04
June				1.44	1.00		1.00	1.78	1.44	1.33	1.09
July				1.28	1.00	1.14	1.78	1.28		1.30	1.07
August				1.14	0.87		1.14	1.00		1.04	0.86
September				1.00	0.54		0.87	1.00		0.85	0.70
October				0.75	0.54		0.75	0.64		0.67	0.55
November	1.28	1.00		0.44	0.54		0.54	0.28	0.44	0.65	0.53
December	0.87	0.11	0.07	0.75	0.87	0.22				0.48	0.40
									Total	14.59	12.00

1/ Source: Appendix B, Reference 4

2/ Normalized Monthly Factors = Monthly Average x $\frac{12.00}{14.59}$

conduct a detailed study of both the AASHO Road Test Performance equation and the method used to convert mixed traffic to an equivalent single loading.

The Performance Equation

The AASHO Road Test was conducted on six pavement loops having various thickness designs of flexible and rigid pavements. Five of these loops were subjected to concentrated traffic loadings of known magnitude and axle configuration. Within each lane of each loop, the loading magnitude and the axle configuration were held constant with the loadings differing from lane to lane and loop to loop. Single-axle loads were applied to the inner lanes and tandem-axle loads to the outer lanes. The sixth loop was not subjected to traffic but was used as a control loop for various physical measurements.

At the conclusion of the Road Test, statistical regression techniques were used to develop empirical equations which predict the performance of the test pavements when subjected to repeated applications of a single weight of axle and one-axle configuration. For flexible pavement, the performance equation is:

$$\log W = \log \rho + \frac{\log G}{\beta}$$

where

$$G = \left(\frac{c-p}{c-1.5}\right)$$

$$\rho = \frac{10^{5.93}\,(D+1)^{9.36}\,L_2^{\;4.33}}{(L_1 + L_2)^{4.79}}$$

$$\beta = 0.4 + \frac{0.081\,(L_1 + L_2)^{3.23}}{(D+1)^{5.19}\,L_2^{\;3.23}}$$

W = the weighted number of axle load applications the pavement carried before its serviceability dropped to a level p

p = Present Serviceability Index (PSI), a measure of pavement surface condition developed for use at the Road Test

c = the initial PSI of the pavement (at the Road Test the average value of 4.2 was used for the flexible pavement sections)

D = a thickness of index developed at the Road Test which combined the thickness of the surface, base, and subbase into a single numerical value based on their relative influence on pavement performance

L_1 = axle load in kips

L_2 = 1 for single axle loads and 2 for tandem axle loads

This equation predicts the number of axle load applications (W) of a given weight (L_1) which a Road Test pavement of a particular design (D) sustained before its PSI was reduced to a given level (p). The ρ term gives the number of axle load applications to a PSI of 1.5. The G term is the serviceability loss ratio which together with β modifies ρ to give the number of axle loadings to any level of serviceability above 1.5. The β term, which modifies the serviceability loss ratio, controls the shape of the W versus PSI curve. At the Road Test, the poorer performing flexible pavements exhibited an increasing rate of serviceability loss with time, while the better performing sections had a decreasing rate of loss. Consequently, some curves were concave up and others concave down. In the equation, the β term controls this concavity. If β is less than one, the curve is concave up. When β equals one, the performance equation is a straight line. β, greater than one, makes the curve concave down. Since β depends on the pavement design and the loading, these factors control the value of the β term. For a given design, increasing the magnitude of the loading will increase the β value. If the loading is held constant, increasing the structural design will lower the β value.

In the attempt to apply the equation to design, the first problem encountered was the multiplicity of axle loadings applied to normal highway pavements. As can be seen, the performance equation is limited to a single weight of axle loading. Thus, a method of accounting for differing loadings on the same pavement had to be developed.

Equivalent Load Method

The commonly employed method of taking these differing axle loads into account is called the Equivalent Load Method. This method, with the 18-kip single-axle load taken as a base value, is used in the Illinois design procedure. The method involves developing axle load equivalency factors which, when applied to applications of a mixture of loadings, produces a number of equivalent 18-kip single-axle load applications. This number can then be used in the performance equation to predict pavement life.

The axle load equivalency factors were developed as the ratios of predicted number of load applications with the number of 18-kip single-axle load applications as a base value. Thus the equivalency factor for an axle load i (E_i) was defined as:

$$E_i = \frac{W18}{W_i} = \frac{\rho_{18} G^{1/\beta_{18}}}{\rho_i G^{1/\beta_i}} = G^{(1\beta_{18}-1/\beta)} \frac{\rho_{18}}{\rho_i}$$

$$= G^{(1/\beta_{18}-1/\beta_i)} \left\{ \left[\frac{10^{5.93} (D+1)^{9.36} \, 1^{4.33}}{(18+1)^{4.79}} \right] \Big/ \left[\frac{10^{5.93} (D+1)^{9.36} L_2^{4.33}}{(L_i + L_2)^{4.79}} \right] \right\}$$

$$= G^{(1/\beta_{18}-1/\beta_i)} \left\{ \frac{(L_i + L_2)^{4.79}}{(18+1)^{4.79} L_2^{4.33}} \right.$$

Examination of this equation reveals that when G equals 1.0 (PSI=1.5) the equivalency factor is a function of the magnitude of the axle loadings alone. However, for other values of G, the equivalency factor is also a function of β which varies with the pavement design.

As noted earlier, the β term controls the shape of the performance curve which can vary from concave upwards to concave downwards. With this great desparity in possible shapes, it would appear that considerable variation could exist in equivalency factors at various PSI levels, suggesting that the equivalency factor changes over the life of the pavement. A possible effect of this variation can be visualized as shown in Figure 1 where at level A $W_x/W_y > 1$, at B $W_x/W_y = 1$, and at level C $W_x/W_y < 1$. While this figure is a somewhat exaggerated presentation, similar questionable effects can be noted by developing equivalency factors at high PSI levels.

This suggests that, to eliminate inconsistencies from pavement designs and analyses based on the AASHO equation, the G and β values should be controlled in a way that will minimize the effects of the shape of the curves. This can be done two different ways. First, by making analyses only at low PSI levels, the G value is close to unity and the effect of the curvature is held to a minimum. Secondly, if β is not allowed to exceed one in any design or analysis, the curves for the different axle weights will all have the same general shape and the equivalencies will remain relatively constant and consistent over the entire performance curve.

One might infer from this that (1) pavement traffic analyses should be limited. to pavements whose PSIs are near the terminal level, and (2) the analysis and design of pavements should be restricted to thickness and loading combinations that provide a β term that does not exceed one. Since for any pavement traffic is an uncontrolled variable, only the pavement structure can be adjusted in order to comply with the second inference. The effect, therefore, would be to limit pavement evaluation and design based on the AASHO Performance Equation to only those pavements with the higher thickness indexes (D). Assuming that 18-kip single-axle load as the maximum loading anticipated, this would limit AASHO based analyses and designs to pavements with thickness indexes greater than 3.25.

Mixed Traffic Theory

An alternate method of accounting for the differing loadings applied to a pavement was developed by Scrivner and Duzan (5). This method, called the Mixed Traffic Theory, employs an integration technique and assumes that the serviceability loss due to a given axle application is a function not only of the axle weight but

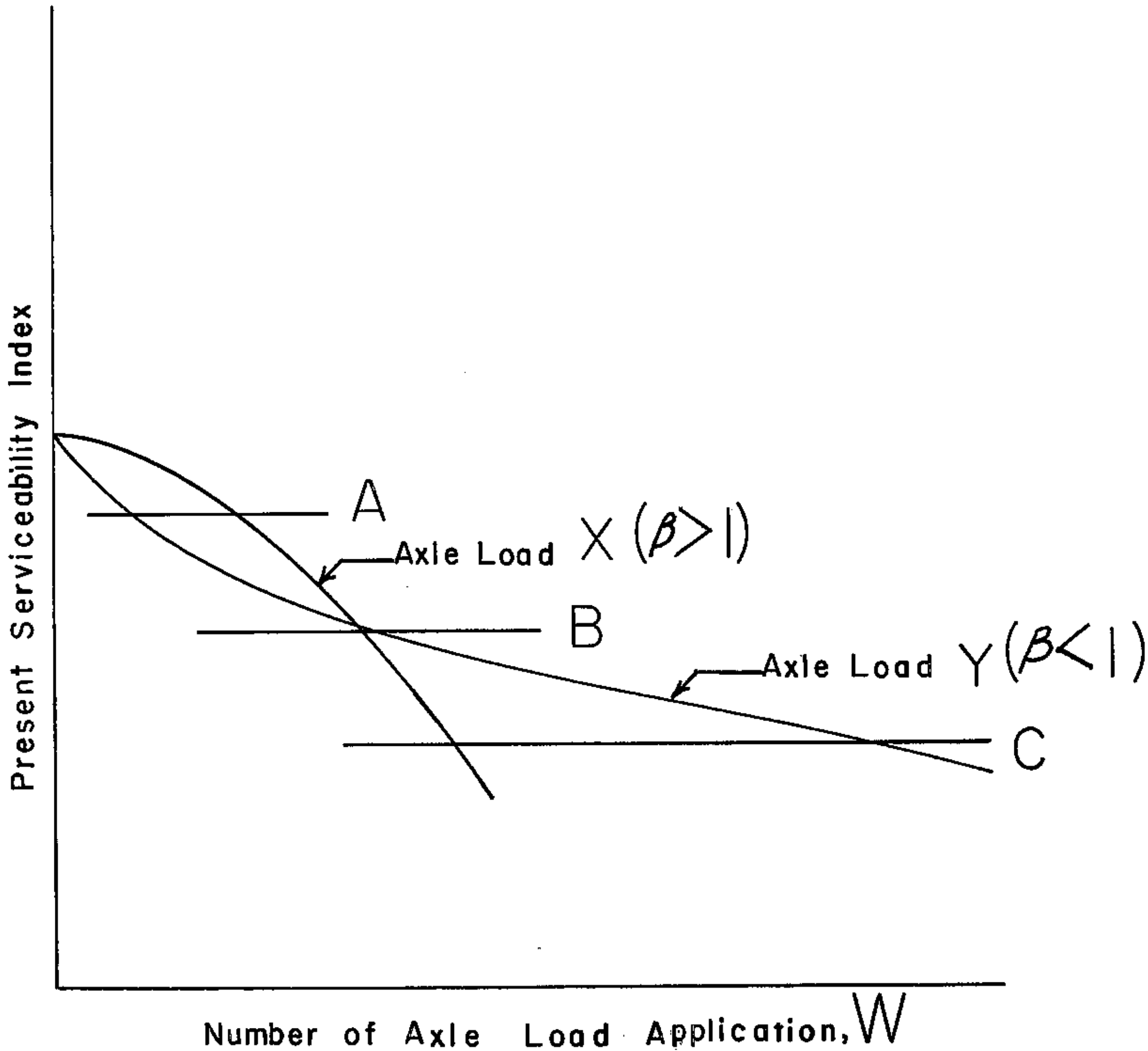

FIGURE I. Hypothetical Performance Curves also of the PSI at the time of application. In this way, the method accounts for the shape of the serviceability versus applications curve and, therefore, is mathematically more precise.

The mathematical expression for this method is:

$$N = \int_0^{G_f} \frac{dG}{\sum_{i=1}^{n} \frac{C_i \beta_i G^{(1-1/\beta_i)}}{\rho_i}}$$

in which

N = total number of axles applied over the life of the pavement

C_i = proportion of the total number of axles that are in weight category i.

G_f = final or terminal value of G

β_i, ρ_i = β and ρ values for the given design for weight category i

Comparison of Conversion Methods

To make a direct comparison of the two methods, the Equivalent Load Method was reduced to a similar mathematical expression. With the definition of variables as used previously, the equivalency factor in the Equivalent Load Method can be expressed as:

$$E_i = \frac{W_{18}}{W_i} = \frac{W_{18}}{\rho_i \, G_f^{1/\beta_i}}$$

Total traffic for the pavement life, expressed in 18-kip single-axle load applications is:

$$W_{18} = \sum_{i=1}^{n} C_i N E_i = N \sum_{i=1}^{n} C_i \, E_i$$

Thus

$$N = \frac{W_{18}}{\sum_{i=1}^{n} C_i E_i} = \frac{W_{18}}{\sum_{i=1}^{n} C_i \left(\frac{W_{18}}{\rho_i G_f^{\,1/\beta_i}}\right)} = \frac{1}{\sum_{i=1}^{n} \frac{C_i\, G_f^{(-1/\beta_i)}}{\rho_i}}$$

Visual comparison of this equation with the Mixed Traffic Theory equation indicates a very strong similarity. In fact, by taking the derivatives dG/dN of both equations, it can be shown that if β were a constant, the two methods would yield identical results (5). Consequently, as could have been expected from the previous discussion, the fact that the Equivalent Load Method ignores the shape of the curve introduces a discrepancy between the methods of traffic conversion.

In the presentation by Scrivner and Duzan, several pavement life predictions were made using both methods. These showed that for medium to heavily designed pavements (D ranging from 4 to 10), the two methods gave nearly identical results. However, for the thinner pavements (D=2), such as are constructed on Class IV roads, the Equivalent Load Method's estimate of pavement traffic life was about double the estimate obtained using the Mixed Traffic Theory.

Because of the more rigorous development, the Mixed Traffic Theory on the surface appears to be the more correct method, suggesting that it should be applied to pavement design, especially for the thinner pavements. However, reflection on the reality of the situation and on the empirical nature of the AASHO Road Test performance equation brings serious doubts as to the validity of the rigor applied in the Mixed Traffic Theory. For example, for a pavement with D equal to 2.0, the initial serviceability loss predicted by the performance equation due to 5,000 applications of an 18-kip single-axle load is equal to the predicted initial loss due to only 125 applications of a 6-kip single-axle load (an effect similar to the hypothetical one shown on Figure 1). This results because of differences in the shape of the performance curves (β is much greater than one for the 18-kip load and less than one for the 6-kip load) and the much greater initial slope of the curve for the lighter axle. Undoubtedly, the lighter loading does not actually cause the greater initial damage and this result is obtained only because of the empirical nature of the performance equation and the curve fitting methods employed in its development.

Nevertheless, by accounting for the shape of the performance curves, this anomaly is a very integral part of any traffic analysis using the Mixed Traffic Theory and, for the thinner pavements at least, would appear to produce erroneous pavement life estimates. With this theory, the predicted PSI is reduced rapidly in the pavement's early life by the many light vehicles. Later, the PSI declines more rapidly due to the heavy loads. This combination would seem to exaggerate the effect of mixed traffic. In this respect, the currently used method of traffic conversion, the Equivalent Load Method, despite its less precise development, would appear to provide the more realistic estimate of pavement life.

In this connection, the importance of understanding the nature and development of the performance equation becomes increasingly apparent. The AASHO Performance Equation is not a precise mathematical expression based on "irrefutable" laws of nature, but is merely an empirical equation developed through statistical curve fitting methods. The precision and accuracy of the equation, therefore, is limited by the conditions and data from which it was developed and by its ability to fit that data. To apply the equation without considering these limitations can lead to inconsistency and serious error.

Nevertheless, the discrepancies due to the shape of the performance curves should not be ignored since they may contain implications regarding the application of the Road Test equation to the satisfactory design of flexible pavements. From this viewpoint, a significant observation from this phase of the study is that the discrepancy between the two traffic conversion methods coincides with the finding that "designs failing early in the Road Test tended to have an increasing rate of serviceability loss ($\beta<1$), while more adequate designs as a rule had a decreasing loss rate ($\beta<1$)" (4). This would seem to suggest that, for a flexible pavement to be adequately designed, β should be less than or equal to 1 for all anticipated axle loads. For a maximum single-axle load of 16 kips (the legal limit on flexible pavements in Illinois) this would require a minimum D of 3.0 (e.g. 3 inches of bituminous concrete surface, 8 inches of crushed stone base course, and 6 inches of gravel subbase) which is somewhat greater than is normally employed on Class IV roads.

APPLICABILITY TO SECONDARY AND LOCAL ROAD DESIGN

In addition to studying the mathematics of the performance equation and traffic conversion methods, the applicability of the design procedure to secondary and local roads was reviewed.

The design procedure was developed as an extension of the AASHO Road Test performance equation. This extension was accomplished by analyzing the performance of actual pavements in Illinois. Through a conversion factor, called the Time Traffic Exposure Factor, the performance equation was modified to more closely fit Illinois experience. However, all the data used in developing this modification were obtained from primary highways. In this respect, the modification may not be consistent with the performance experience of secondary and local roads in Illinois.

Sufficient data were not available to totally evaluate this point. Nevertheless, the analysis of secondary and local pavement data which is presented later seems to indicate that the modification is not unreasonable.

At the same time, however, applying the Present Serviceability Index, which forms the basis of the performance equation, to secondary and local roads, is open to question. This index was developed for the Road Test to provide a quantative measure of a pavement's ability to serve traffic. Short stretches of pavement were rated by a panel of highway users on a scale of 1 to 5 (5 being an excellent rating) according to their ability to serve high-speed, high-volume mixed traffic. These ratings were subsequently used together with measurements of various pavement surface properties (roughness, cracking, patching, and rutting) to develop the mathematical expression called the Present Serviceability Index (PSI). This expression was later used to rate the Road Test pavements and, as such, provided the basis of the AASHO performance equation.

As developed, the Present Serviceability Index was intended for use with primary highways and, in this respect, may not be directly applicable to secondary and local roads. Indeed, if the same concept were developed for these roads, it is likely that an entirely different set of relationships and acceptability criteria would result. While such a development was beyond the scope of the present study, the effect of a minor change in acceptability criteria could be investigated.

In applying the performance equation to pavement design, Illinois recognized that the minimum acceptable level of serviceability differes with different types of highways. Based on a study of the PSI of various primary highways just prior to being resurfaced, the Illinois design procedures were developed on the basis of a final PSI of 2.5 for Interstate and other four-lane divided highways (Class I roads) and 2.0 for all other highways. From this it seemed logical to assume that an even lower PSI might be acceptable for local and secondary roads. Consequently, the effect of redefining the final PSI of these roads to be 1.5 was studied.

The performance equation was used with AASHO Road Test thickness indexes representative of typical Class IV road designs to compute the predicted number of 18-kip single-axle loads that similar pavements at the Road Test would have carried to a PSI of 2.0. These numbers were then reentered into the equation and thickness indexes were determined for pavements that would have carried the same number of loads, but to a PSI of 1.5. These computations showed that for typical Class IV roads a redefinition of the final PSI to 1.5 would result in a reduced thickness requirement equivalent to less than 1/2 inch of crushed stone base course. This was not considered to be sufficient to warrant making any change in the design procedure.

ANALYSIS OF SECONDARY AND LOCAL PAVEMENT DATA

As a final attempt to evaluate the realiability of the currently used vehicle equivalency factors, the County Highway Departments were requested to submit information concerning the pavement designs, traffic conditions, and maintenance histories of their secondary and local highway sections. It was hped that data would be received from a sufficient number of sections that the performance, design, and traffic histories could be used in a statisticaly regression analysis to obtain estimates of the VEFs. To be usable in this analysis, the data had to be from pavement sections with completely known structural designs that had required little or no maintenance since first constructed and that were nearing the end of their initial lives. It was recognized that, due to the nature and evolution of the local and secondary road system, the number of sections meeting these criteria would be limited. Nevertheless, the response to the request was encouraging as the Counties submitted data from 154 pavement sections that they felt most closely fit these criteria. However, after careful screening, only 21 sections were found that could be used and 10 of these had PSIs greater than 3.25, indicating that they were not as close to the end of their service lives as desired. Most of the rejected sections were unusable because either insufficient traffic and design data were available or extensive maintenance had been required which precluded determining structural numbers and PSIs that could be used in pavement performance analyses. The remaining sections that could not be used were rejected because their PSIs were too high to permit evaluation of their performance.

Regression Analysis

Despite this disappointing amount of usable data, the intended regression analyses were performed but with reduced hopes for obtaining meaningful results. The regression equation took the general form:

$$W = a_1 + a_2\ PC + a_3\ SU + a_4\ MU$$

in which

W = number of equivalent 18-kip single-axle loads applied to the pavement

PC, SU, MU = one direction total number of passenger cars, single units and multiple units respectively that have used the pavement since constructed

a_1, a_2, a_3, a_4 = regression coefficients

This equation was selected because of its similarity to the Illinois traffic evaluation equation which, in its simplest form, can be represented by:

$$TF = (a_p\ PC + a_s\ SU + a_m\ MU)\ 10^{-6}$$

in which

TF = Traffic Factor, number of equivalent 18-kip single-axle load applications expressed in millions

PC, SU, MU = Total number of passenger cars, single units and multiple units, respectively, expected to use the principal traffic lane over the design life of the pavement

a_p, a_s, a_m = VEFs for passenger cars, single units and multiple units respectively

From this it would appear that the regression coefficients a_2, a_3, and a_4 would approximate the vehicle equivalency factors a_p, a_s, and a_m.

For the regression analysis, W values were determined in the following manner using the AASHO Road Test performance equation. First, measures of pavement roughness, cracking, patching, and rutting were obtained from each pavement section included in the analysis and were used to determine the current PSI of the section. Second, the thickness of the various pavement components--surfacing, base, and subbase--were combined in accordance with the structural coefficients used for the material in design to determine the Illinois Structural Number of the pavement. This number was subsequently converted to a thickness index equivalent to that of an AASHO Road Test flexible pavement section by dividing the Structural number by the Time Traffic Exposure Factor. This factor, equal to 1.1, was established when the Illinois flexible pavement design procedure was developed to modify the Performance Equation to correspond with the performance of normal flexible highway pavements in Illinois (1). Finally, this Thickness Index and PSI were used with the performance equation to determine a performance W.

Since only the current PSIs were available, this final step required that an initial PSI be assumed for all sections. Without a better value being available, the average initial PSI of the Road Test flexible pavements, 4.2, was assumed.

To complete the data required for the regression analysis, the total numbers of each type of vehicle that had used one lane of each pavement was determined by averaging the present and initial vehicle type ADT and multiplying by the number of days since the section was first opened to traffic.

Two regression analyses were made using the data from all 21 sections. The first used the general equation shown before:

$$W = a_1 + a_2 \text{ PC} + a_3 \text{ SU} = a_4 \text{ MU}$$

The second employed a modification of this equation in which the equivalent 18-kip single-axle load applications attributable to passenger cars were removed from the variable W. With the vehicle equivalency factor for passenger cars being 0.0004, this regression equation was:

$$W' = a_1 + a_2\ SU + a_3\ MU$$

in which

$$W' = W - 0.0004\ PC$$

The two analyses were performed for two reasons. First, since the average weight of passenger cars should be about the same on all roads, the VEF for passenger cars should be constant. By removing them from the analysis, the number of variables was reduced, increasing the chances of obtaining meaningful results. Secondly, it was reasoned that ideally the two analyses should yield approximately the same for the truck VEFs and that the first method should yield a passenger car VEF to 0.0004. If these ideal results were obtained, they would provide evidence, in addition to the standard statistical tests, that the analyses provided meaningful information.

In addition, similar analyses were made using only the 11 sections having PSIs less than 3.25. This was done in an attempt to reduce any error due to the assumed initial PSI value and to possibly avoid any discrepancies that might be introduced by the shape of the performance curves.

Table 6 displays the results of the analyses. None of these are statistically significant and the expected similarities between the coefficients obtained using the different regression equations were not obtained. However, the most alarming aspect of the results is the negative coefficients obtained. Obviously the regression analyses failed to provide any usable information.

Structural Number Analysis

As a result, the data were approached from a somewhat different angle. This approach compared the pavements' actual Structural Numbers to the Structural Numbers

TABLE 6: Results of Regression Analysis of County Highway Data

All Sections:

$$W = 26.920 + 0.078\ PC - 0.21175\ SU - 0.6198\ MU$$

Multiple Correlation Coefficient -R = 0.3811

$R(a = .05) = 0.601$

$$W' = 30.7829 + 0.0519\ SU - 0.3552\ MU$$

Multiple Correlation Coefficient -R = 0.2142

$R(a = .05) = 0.532$

Sections with PSI < 3.25:

$$W = 10.536 + 0.0856\ PC - 0.2091\ SU - 0.6162\ MU$$

Multiple Correlation Coefficient -R = 0.6585

$R(a = .05) = 0.807$

$$W' = 18.605 + 0.06205\ SU - 0.2849\ MU$$

Multiple Correlation Coefficient -R = 0.3428

$R(a = .05) = 0.601$

that the design procedure would require for similar traffic volumes. To make this comparison, it was first necessary to extend the traffic volumes carried to date to some future time when the sections' PSIs would be 2.0.

To do this, it was again necessary to assume an initial PSI value and, as before, 4.2 was used. For extrapolation purposes, an assumption concerning the general shape of the serviceability loss curve also had to be made. As discussed previously, the Road Test showed that this shape can vary from concave downward to concave upward. Within the range of pavement designs and loadings involved in these data, both general shapes would be expected. It was, therefore, decided to assume a straight line relationship. With this assumption, the total traffic carried by each section was extrapolated from the current PSI value to a PSI of 2.0 using the equation:

$$T = t \times \frac{4.2 - 2.0}{4.2 - PSI}$$

where

T = total number of passenger cars, single units, and multiple units the pavement is expected to carry before its PSI equals 2.0

t = total number of passenger cars, single units, and multiple units the pavement has carried to the present time

PSI = current PSI

Subsequently, the extrapolated data were used with the Class III and Class IV vehicle equivalency factors to compute Traffic Factors. These factors and the soil support CBR values supplied by the Counties were then applied to the Illinois design nomograph to select design Structural Numbers for each section. These values are compared with the actual Structural Numbers on Figures 2 and 3. Also shown on these figures are confidence limits based on the AASHO Road Test data analysis which showed that the actual performance of 90 percent of the test pavements fell within a band equal to $D \pm .11\ (D+1)$ where D is the test pavement thickness index. These limits are shown as dashed lines. For the purpose of this study, the VEFs were assumed to have provided a reasonable estimate of the effects of mixed traffic for the sections falling within these limits.

On Figure 2, which displays the results of the analysis using Class IV vehicle equivalency factors, 10 sections are above the limits, 7 within the limits, and 4 below. This suggests that, in general, the Class IV factors underestimate the effect of mixed traffic and result in underdesigned pavement sections. This appears particularly true for the Class III sections which show 6 above, 4 within, and 2 below the limits. The average Structural Number error of estimate using the Class IV factors is -0.25, which is equivalent to an underdesign of approximately 2 inches of crushed stone base course.

Figure 3 displays the results of the analysis using the Class III vehicle equivalency factors. On this figure, 2 Class III and 3 Class IV sections plot above, 6 Class III and 3 Class IV sections plot within, and 4 Class III and 3 Class IV sections plot below the limits. With the exception of two sections that fall well below the limits, these factors appear to predict the effects of mixed traffic reasonably well. The average Structural Number error of estimate for all sections is + 0.19 which is equivalent to an overdesign of approximately 1 1/2 inches of crushed stone base course. However, if the two sections falling farthest below the limits are ignored, the average error is only 0.01, or approximately 0.1 inches of crushed stone.

To test whether the assumption used to extend the traffic data to a PSI value of 2.0 introduced any detectable error in the analysis, the error in predicting the Structural Number of each section (using the Class III VEFs) was plotted versus the current PSI on Figure 4. The correlation coefficient for this plot is -0.09, indicating that the assumption was reasonable and did not introduce a consistent error to the analysis.

CONCLUSION

In the initial work that extended the AASHO Road Test performance equation for use in Illinois pavement design, three kinds of data were required....performance

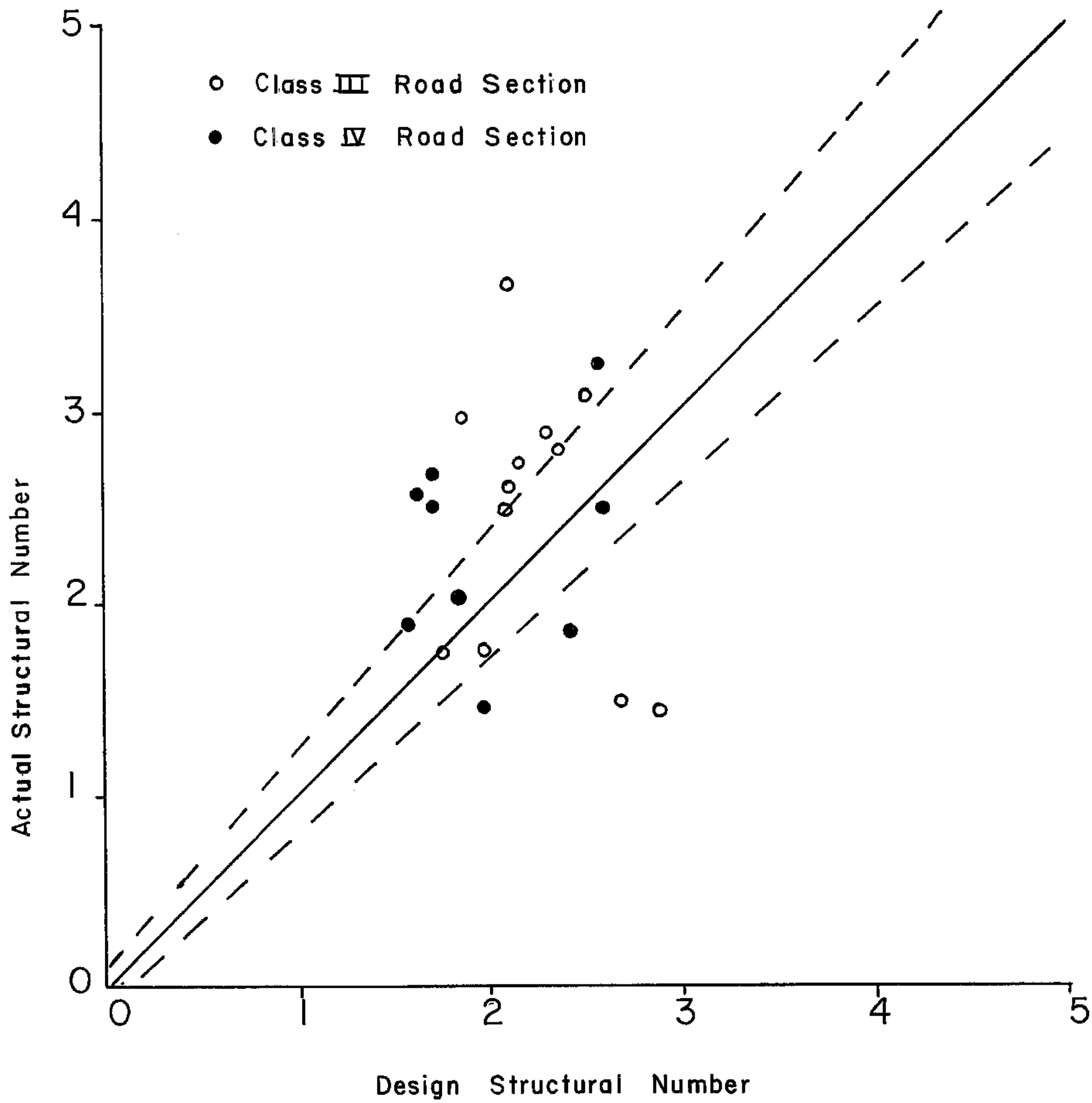

Figure 2. Structural Number Comparison Using Class IV VEFS

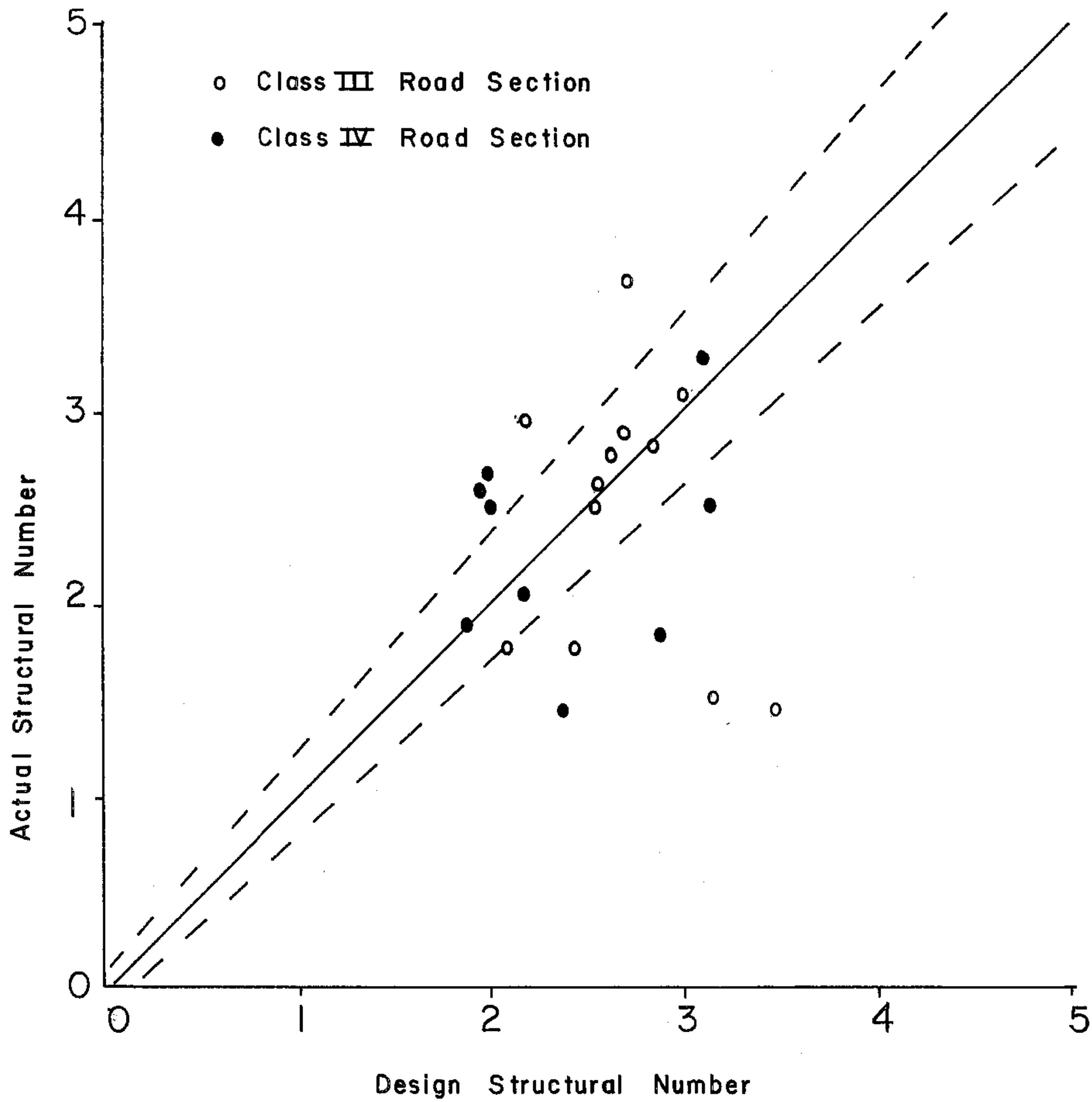

Figure 3. Structural Number Comparison Using Class III VEFS

data from in-service pavements, vehicle classification data, and truck weight data. Of these, only vehicle classification data in limited amount were available from secondary and local roads, necessitating that extrapolations and estimates be employed to extend the procedure for use in designing these roads.

For the present study, secondary and local road truck weight data were still unavailable and only an extremely limited amount of pavement performance data could be obtained. Nevertheless, the reliability of the vehicle equivalency factors currently being used in the design of secondary and local flexible pavements has been investigated by several different methods....an analysis of vehicle classification data, the evaluation of the effect of spring load limits and seasonal traffic fluctuations, a study of the performance equation and method of traffic conversion, and analyses of actual pavement performance. While none of these produced totally concrete results, the classification data and the pavement performance data both indicated that the previously developed factors for Class III roads (400<ADT<1,000) are reasonable and that similar values should be used in designing Class IV roads (ADT<400). Spring load limits and seasonal traffic fluctuations were found to have a potential for lowering the average vehicle equivalency factor values significantly, however, not to the level of the current Class IV VEFs.

Despite the consistent indication that higher values should be used in the design of Class IV pavements, the direct and immediate application of this finding must be seriously questioned since the general opinion of the engineers involved in the design, construction, and maintenance of these roads is that higher VEFs would require pavement designs much thicker than past experience has indicated to be necessary. While within the scope of the present study it was not possible to explore the reason for the apparent inconsistency between this engineering experience and the indications of the study, it is suspected that the discrepancy is attributable to differences

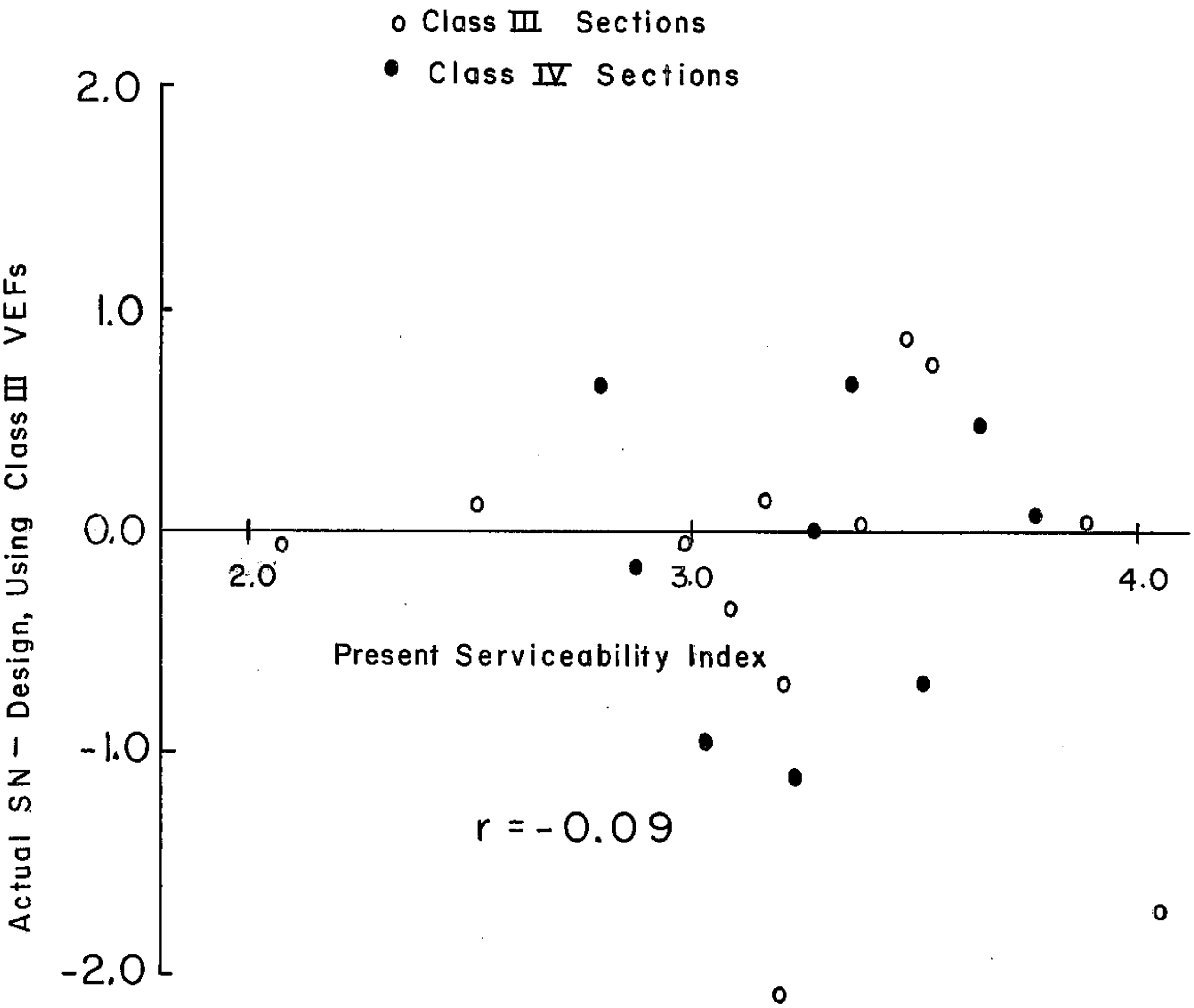

Figure 4. Stuctural Number Prediction Error Versus Current PSI.

in pavement design philosophy and to the application of the AASHO performance equation and Present Serviceability Index to low-volume roads.

As conceived and as developed from the AASHO Road Test data, the Illinois design procedure is intended to provide pavement designs that will perform adequately with little or no maintenance for a 15- to 20-year design period. While this philosophy corresponds with the intentions and experience of engineers involved in the design of primary highways, it is likely that the designers of Class IV roads are willing to accept greater and earlier maintenance. In fact, the limited funding available to most local agencies for road construction and the very real necessity of meeting the transportation demands of the local community may, in a large part, dictate such a philosophical difference. This is not to argue the right or wrong of either philosophy, but simply an attempt to pinpoint a possible reason for the discrepancy.

In addition to this, and as discussed earlier, the application of the Present Serviceability Index and the AASHO performance equation to secondary and local road designs is open to question. The Present Serviceability Index and the performance equation were developed specifically for the primary highway system. As stated in the Road Test report, "Obviously, serviceability must be defined relative to the intended use of the road" (4). While a quantitative appraisal of the effect of redefining serviceability and performance for low-volume roads was beyond the scope of this study, intuitively such a redefinition undoubtedly would reduce the pavement design thickness required for acceptable performance.

At the same time, the extension of the performance equation for use in Illinois pavement design may itself not be directly applicable to the design of Class IV roads. This is not to suggest that the use of the procedure for these roads should be discontinued. On the contrary, since 1964 this procedure has been used for the structural design of both Class III and Class IV roads with reasonable success, indicating that the assumed Class IV vehicle equivalency factors have done an adequate job of accounting for the difficulties mentioned above. In this respect, it is recommended that use of the procedure be continued with no change in the VEFs. However, those involved in the design of these roads should be thoroughly informed of the findings of this study to allow them to develop designs consistent with the performance they desire.

In this respect, future research directed toward determining acceptability requirements for secondary and local road performance is recommended. This work should include the determination of serviceability criteria at the levels and frequency of maintenance deemed tolerable during a normal design life.

Other research to establish the reasonableness of the assumption utilized in analyzing vehicle classification count data is also in order. This would entail collecting sufficient truck weight data to allow testing the hypothesis that the axle weight distributions for individual vehicle types are the same on secondary and local roads as they are on the primary highway system. At the present time, however, the collection of such data is considered too costly to be warranted. Nevertheless, with the development of more sophisticated and reliable weigh-in-motion devices, this work will be feasible in the future.

IMPLEMENTATION

The findings of this study are not recommended to be implemented in the sense that changes will be made in the current design procedures. However, it is recommended that this report receive widespread distribution among the local agencies in Illinois that are concerned with the design of low-volume roads. In this way, they will be made cognizant of the fact that major pavement maintenance can be expected within the design period on Class IV roads designed using the current vehicle equivalency factors. This knowledge will sharpen their engineering judgment and will enable them to develop designs having a much greater probability of requiring no maintenance during the design period if they so desire.

REFERENCES

1. Chastain, W. E., Sr., and Schwartz, D. R., "AASHO Road Test Equations Applied to the Design of Bituminous Pavements in Illinois," Highway Research Record 90, 1965.

2. Chastain, W. E., Sr., Beanblossom, J. A., and Chastain, W. E., Jr., "AASHO Road Test Equations Applied to the Design of Portland Cement Concrete Pavements in Illinois," Highway Research Record 90, 1965.

3. Chastain, W. E., Jr., and Elliott, R. P., "Traffic Evaluation for Illinois Pavement Design," Research and Development Report 36, Bureau of Research and Development, Illinois Department of Transportation, April 1972.

4. "The AASHO Road Test, Report 5, Pavement Research," Special Report 61E Highway Research Board, 1962.

5. Scrivner, F. H., and Duzan, H. C., "Application of AASHO Road Test Equations to Mixed Traffic," Special Report 73, Highway Research Board, 1962.