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RELATIVE DAMAGING POWER OF WHEEL LOADS IN MIXED TRAFFIC

by

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ABSTRACT

A description is given of the features of the AASHO Road Test that are relevant to the issue of the damaging power of wheel loads and to the derivation of the widely used fourth-power relationship between the magnitude of a wheel load and its damaging power. The results of the Road Test are then re-analysed to give more logical damage relationships that take into account the fact that an in-service road deteriorates to failure under the action of a wide spectrum of wheel loads applied in a random sequence. The relations obtained do not fit simple power laws; effective exponents range from about 3 to over 6 with the highest values occurring on weak pavements trafficked by heavy wheel loads, at or above the present legal limit.

1. INTRODUCTION

The systematic observation of road behaviour, in particular the behaviour of the Laboratory's many full-scale road experiments, has demonstrated that structural damage to road pavements is related primarily to the cumulative commercial traffic using them. Although it was also realised that heavy wheel loads must bring about more damage than light ones and that road design should recognise the fact, the mixed traffic conditions on normal roads precluded any detailed conclusions being reached as to the relative damaging power of wheel loads of different magnitudes. Definition of such a relationship was one of the main objectives of the well known road experiment carried out between 1958 and 1960 under the auspices of the American Association of State Highway Officials¹. The AASHO Road Test is important, not only because of its size, but because the results relating to damaging power of wheel loads have been widely adopted in pavement design practice.

In the Road Test any given pavement section was trafficked by wheel loads of a single magnitude. Relations between wheel load, being half the axle load, and damaging power were obtained by comparing the number of repetitions of a load of a given magnitude required to cause structural failure of a pavement with the number of repetitions associated with another wheel load to bring about the same failure condition. The generally accepted interpretation of the results yields the well known fourth-power relation between the magnitude of the wheel load and its structural damaging power.

This Report briefly describes the features of the Road Test that are relevant to the issue of damaging power and to the derivation of the relationship between wheel load and damaging power. The results are then re-analysed to give more logical damage relationships that take into account the fact that an in-service road deteriorates to failure under a wide spectrum of wheel loads rather than by trafficking with wheel loads of only a single magnitude.

2. THE AASHO ROAD TEST

Definition of relationships between the magnitude of a wheel load and its ability to bring about structural damage was one of the primary objectives of the test. The experiment was constructed in Illinois, USA and trafficked over a two-year period. Lorries of different sizes having different wheel loads were used to traffic separate long loops of test road, each loop containing a large number of sections of experimental pavement of both flexible and rigid construction; single and tandem axles were studied separately.

2.1 Assessment of pavement deterioration

Performance of the pavements was expressed in terms of the rate of change of the serviceability of the pavement under trafficking. A panel of road users, including road engineers, travelling over selected roads in vehicles of their choice, assessed the serviceability of these roads on a scale of 0 to 5. The panel were allowed to inspect the road but their final assessment was primarily a measure of the ability of the road to carry them safely and comfortably. On flexible pavements their combined rating was then correlated statistically with three measurements of the state of the road surface: longitudinal slope variance (a measure of riding quality), rut depth and the percentage of the area of the road surface that was either cracked or patched. Multiple regression of the three parameters with the panel ratings defined the Present Serviceability Index (PSI) which was a measure of the current condition of the experimental road pavements during trafficking. The mean value of PSI of the pavements at the beginning of the experiment was 4.2; the value declined as each pavement deteriorated under trafficking until the pavement was taken out of service at a PSI of about 1.5.

In the United Kingdom the deterioration of a flexible pavement is assessed primarily in terms of deformation or rutting in the wheelpath and not in terms of riding quality. However a reasonable correlation exists between PSI and rut depth², which allows the AASHO results to be translated into deterioration expressed in terms of rut depth alone; typical failure in a flexible pavement in the United Kingdom is then judged to be at a PSI of about 2.5.

2.2 Performance model

To analyse the performance data, general mathematical models were selected as being representative of pavement behaviour. The most widely accepted model³ related the decrease in PSI, Δp , from its initial value to other variables, by the equation

$$\frac{\Delta p}{R} = \left(\frac{N}{\rho} \right)^\beta \dots \dots \dots (1)$$

- where
- R is the decrease in PSI at failure
 - N is the number of applications of wheel load at any time
 - ρ is a function of pavement design and wheel load variables representing the expected number of load applications at a serviceability index of 1.5
 - β is a function of pavement design and wheel load variables which govern the shape of the serviceability trend.

When the initial value of PSI is taken as 4.2 and failure is deemed to occur at a value of 2.5, equation 1 can be used to express the instantaneous PSI, p, as

$$p = 4.2 - 2.7 \left(\frac{N}{\rho} \right)^\beta \quad \dots \dots \dots (2)$$

Because β is a function of wheel load, the choice of failure level will influence the relative damaging effect of various wheel loads at failure.

Multiple regression analysis of Road Test observations was used to evaluate terms in the assumed form of functions ρ and β . For seasonally weighted single axle load applications

$$\rho = \frac{10^{5.93} (D + 1)^{9.36}}{(L + 1)^{4.79}} \quad \dots \dots \dots (3)$$

$$\beta = 0.40 + \frac{0.081 (L + 1)^{3.23}}{(D + 1)^{5.19}} \quad \dots \dots \dots (4)$$

where, in each expression

L = load on single axle (lb x 10³)

and D = the structural number

$D = a_1 D_1 + a_2 D_2 + a_3 D_3$ where a_1, a_2 and a_3 are materials coefficients derived from the results of the Road Test and D_1, D_2 and D_3 are the thickness in inches of surfacing, roadbase and sub-base respectively.

Equation 1 enables the number of load applications required to bring about any decrease in PSI to be calculated directly in terms of wheel load and Structural Number. It can therefore be used to compare the predicted number of applications of wheel loads of different magnitudes required to bring about failure in a given pavement. The well known fourth-power law relating the damaging effect of a wheel load to the fourth-power of its magnitude was derived in this way. Figure 1 shows the deviations from this law for four flexible pavements of widely different thickness and construction (and therefore Structural Number); only on the weakest pavement was there any significant deviation from the fourth-power, heavy wheel loads proving to be, as one would expect, more damaging. It is customary to relate the damaging effect of wheel loads of different magnitudes to a standard reference wheel load, normally 40 kN: this has been done in Figure 1.

For design purposes the damaging power of the different elements of the expected spectrum of wheel loads are converted into equivalent 40 kN wheel loads (80 kN axles) and summed to give a total number of these axles. A convenient indication of the damaging power of a given spectrum is that number of 80 kN axles that are equivalent to 100 axles of the traffic spectrum considered.

2.3 Acceptability of the model

The 'law' developed is a mathematical one rather than one based directly on the physical behaviour of pavement materials and soils; it has also been shown that the choice of the form of the mathematical model greatly influences the results obtained². An unsatisfactory aspect of the model described above is shown in Figure 2. Equation 1 predicts more rapid early deterioration of pavements under light wheel

loads than under heavy ones; similarly, thicker pavements are initially more vulnerable than thin ones under a wheel load of given size.

The AASHO test site had a severe continental climate and the majority of pavement deterioration took place during and soon after the spring thaw. At this time the test pavements were in a very weak condition; the strength of the subgrade and sub-base was markedly lowered because of the presence of excess moisture as the result of previous frost action and the stiffness of the bituminous pavement materials was also reduced by rising spring temperatures. In an attempt to allow for this weighting factors related to measured pavement deflections were applied to the numbers of load applications actually carried out. Deterioration is to some extent seasonal in all climates but it is difficult to judge the suitability of the weighting used to conditions other than those occurring in the Test.

The consensus view is that the model generates results about damaging power which, when applied to design, produce solutions that are in broad accord with observed behaviour; precise validation is difficult because the observed behaviour of roads under normal traffic cannot provide detailed information on the link between pavement damage and axle weight. Conclusions relating to the prediction of pavement life are likely to require modification for application to pavements in other climatic conditions. Whatever the limitation of the AASHO Road Test and the damage laws derived from it, the test did serve to focus attention on the great structural damaging power of heavy wheel loads and the subsequent adoption of design procedures taking this into account was a considerable step forward in relation to previous practice. General acceptance of the results also reflected the fact that more authoritative information about the damaging power of wheel loads would be difficult to obtain until methods of structural design have been more fully developed; these would automatically yield damage laws based on the actual physical behaviour of road materials.

3. AN ALTERNATIVE INCREMENTAL ANALYSIS

One of the more important differences between the AASHO Road Test and in-service road conditions is the nature of the trafficking. Whereas roads in service experience a range of wheel loads applied in virtually random order, pavements in the Road Test received repeated applications of wheel loads of one magnitude only. Given the present acceptance of the basic AASHO model of pavement performance for evaluating relative damaging power of wheel loads, it is logical to use that model to examine the way in which pavements deteriorate under mixed traffic conditions. This can be done if it is assumed that the rate of deterioration of a pavement at any given condition, represented by a particular PSI value, is independent of the previous history of loading of the pavement, ie the way in which the decrease in PSI to that condition takes place. This assumption, although not strictly correct in many pavement situations, is a necessary simplification and is, of course, also inherent in the conventional usage of the fourth-power law to characterise mixed traffic.

In principle the proposed incremental method of analysis calculates the decrement in PSI of a pavement caused by the application of each single wheel load in the traffic spectrum when applied in random order from the beginning of trafficking until failure occurs; it thus allows for the non-linearity of the PSI trends shown in Figure 2 whereas the conventional analysis makes no allowance for non-linearity. The decrease in PSI caused by all wheel loads of any given magnitude in the traffic spectrum applied until failure, and thence the damage caused by each individual wheel load, is then calculated. This damage is then compared with values similarly derived for other magnitudes of wheel load to give damage equivalences for mixed traffic conditions.

In practice the analytical procedure is carried out by a computer. For a given traffic spectrum, the frequency with which each of 10 wheel load categories occurs is known and is expressed as a percentage of the total. These percentages are then summed consecutively so that each wheel load category is represented by a range of numbers within the scale zero to 100. A random number within this scale is then generated thus selecting a wheel load to commence the analysis. Even with the aid of computing techniques it is impractical in most cases to calculate the deterioration caused by an individual wheel load and the analysis is therefore carried out using groups of wheel loads of a single magnitude. Group size influences the accuracy of the computations and computer time involved. Large group sizes applied to weak pavements give inaccurate results but strong pavements deteriorate slowly under trafficking and therefore require wheel-load groups in excess of 100 in order to keep computer time within practical limits. The values of β and ρ appropriate to the pavement and wheel load being known, Equation 2 is solved to give the loss in serviceability or decrement of PSI resulting from the application of this group of wheel loads.

A second random number is generated thus selecting possibly the same, but normally a different, wheel load. The values of β and ρ defining the performance curve appropriate to the new wheel load are then used to determine that number of applications of the second wheel load which bring about the same decrement of PSI as was produced by the first group of wheel loads. The co-ordinates of PSI and number of load applications on the performance curve having been established, the further decrement of PSI associated with the second group of applied wheel loads is calculated. The procedure is repeated until the serviceability level has fallen to a failure condition, taken as a PSI value of 2.4, just below that associated with failure in the United Kingdom.

As damage accumulates the program stores the numbers of wheel loads applied in each category and the increments of damage attributable to them. At intervals of 0.1 in PSI, results indicating the serviceability trend at that point are produced, together with the damage contributed by each wheel-load category.

If the damage expressed as the decrement of PSI attributable to N_i wheel loads of magnitude W_i is Δp_i , this being part of a larger decrement of PSI associated with all the applied wheel loads in the traffic spectrum, then the damaging power of each of the N_i wheel loads of magnitude W_i is $\frac{\Delta p_i}{N_i}$. The relative damaging power of the wheel load in the traffic spectrum considered is then obtained by comparing $\frac{\Delta p_i}{N_i}$ with the same parameter for a standard reference wheel load in the spectrum (40 kN). The computer program enables these calculations to be made at intervals of 0.1 in PSI throughout the service life of the pavement.

4. RESULTS

The analysis covered most of the range of traffic spectra and pavement strengths likely to be encountered in the United Kingdom. Table 1 gives details of four spectra measured by the Laboratory⁴ and considered to be typical of very heavy, heavy, medium and light traffic; their damaging power as derived from conventional analysis is also quoted.

TABLE 1

Traffic spectra used in the analysis

	Traffic spectrum	Wheel-load category (kN)										Total
		4	13	22	31	40	49	58	67	76	85	
Percentage of total number of axles in each wheel-load category	very heavy	7.4	28.8	24.6	15.7	9.2	7.4	4.9	1.6	0.3	0.1	
	heavy	16.5	25.9	22.0	10.3	11.8	9.7	3.0	0.5	0.2	0.1	
	medium	13.3	36.3	27.0	10.4	7.8	3.9	1.1	0.2	0.0	0.0	
	light	16.7	59.5	14.5	6.3	2.5	0.3	0.2	0.0	0.0	0.0	
Standard axles per 100 axles	very heavy	0.0	0.3	2.2	5.5	9.2	17.0	21.5	12.5	4.1	1.5	74
	heavy	0.0	0.3	2.0	3.6	11.8	22.3	13.2	3.8	2.4	1.9	61
	medium	0.0	0.4	2.4	3.7	7.8	8.9	4.8	1.4	0.5	0.7	31
	light	0.0	0.6	1.3	2.2	2.5	0.6	0.8	0.1	0.0	0.0	8

Table 2 gives details of the four pavement constructions considered. All combinations of traffic spectra and pavement constructions were analysed except the unlikely combinations of lighter traffic spectra on strong pavements and of the very heavy traffic spectrum on the weaker pavements.

TABLE 2

Pavement constructions used in the analysis

Pavement type	Type and thickness of			Structural Number D*
	surfacing	roadbase	sub-base	
A very strong	100 mm asphaltic concrete	165 mm asphaltic concrete	300 mm sandy gravel	5.97
B strong	100 mm asphaltic concrete	120 mm asphaltic concrete	300 mm sandy gravel	5.18
C medium	100 mm asphaltic concrete	200 mm crushed stone	150 mm sandy gravel	3.54
D weak	50 mm asphaltic concrete	150 mm crushed stone	150 mm sandy gravel	2.38

* Structural Number calculated from Equation 5 using values for the coefficients derived in the AASHO Road Test¹.

Figures 3, 4, 5 and 6 present the damaging power of wheel loads, derived by the incremental method described above, in relation to the damaging power of a reference wheel load of 40 kN. All curves relate to failure conditions at a PSI of 2.5. On these double logarithmic plots a simple power relationship between wheel load and damaging power would be a straight line. Lines representing power relations of varying exponents are also shown for purposes of comparison.

No single power law emerges and in most cases a single exponent does not characterise the response of a particular pavement to a given traffic spectrum. The secant slope of the curve between any given wheel load and the standard reference wheel load indicates the effective exponent of the damage relationship for that wheel load in the given spectrum. Table 3 summarises this effective or secant exponent for each of 5 wheel loads occurring in the four different traffic spectra when applied to the four selected pavements. Results are tabulated for the standard reference value of 40 kN, a load of half its size (20 kN), the present United Kingdom legal maximum of approximately 50 kN, the highest legal maximum in the EEC of 65 kN and finally a value of 80 kN, an exceptionally heavy wheel load.

TABLE 3

Effective exponents of damaging power derived from the alternative analysis

Structural Number	Wheel-load spectrum	Wheel load (kN)				
		20	40	50	65	80
5.97	very heavy	3.9	3.9	3.2	3.0	4.1
	heavy	3.9	3.9	3.2	3.0	4.1
5.18	very heavy	3.8	3.8	3.3	3.5	3.9
	heavy	3.8	3.8	3.3	3.5	3.9
	medium	3.8	3.8	3.3	3.5	3.9
3.54	heavy	2.4	2.6	3.8	4.2	4.5
	medium	2.8	3.1	4.6	4.7	4.9
	light	3.3	4.0	4.6	5.6	—
2.38	heavy	3.0	5.4	6.0	5.8	6.6
	medium	3.3	5.7	6.0	5.8	6.6
	light	4.2	6.0	6.0	5.8	—

The incremental analysis recognises the non-linearity of the serviceability trends and as a result predicts wider deviation from a simple fourth-power law than would the conventional analysis. An overall impression of the difference between the two methods of analysis is given in Table 4 which compares equivalence factors derived from the conventional analysis, with factors obtained by the incremental analysis. The results are for the same combinations of wheel-load spectra and pavement structure already examined. On all pavements the incremental approach suggests that lighter loads will cause more damage than would be indicated by a conventional analysis. On pavements of weak and medium strength, heavy wheel loads, above the present UK legal limit, will cause more damage than would be suggested by the conventional analysis.

The exponents given in Table 3 and the equivalence factors from which they are derived, in Table 4, are directly drawn from Figures 3–6. Although these exponents (and equivalence factors) permit the damaging effect of an axle, relative to that of an 80 kN standard axle to be calculated they may not be used to deduce the damaging effect relative to that of any other axle weight, eg the frequently used 10 tonne axle. To calculate the damaging power, D_x , of a particular axle weight, x , in terms of that of an 80 kN standard axle, D_o , the appropriate exponent, n , from Table 3 is used in the formula $\frac{D_x}{D_o} = \left(\frac{x}{80}\right)^n$.

TABLE 4

Comparison of equivalence factors obtained by conventional and incremental analysis
(equivalence factor for 40 kN wheel-load = 1)

Structural Number	Wheel-load spectrum	Wheel load (kN)											
		20		50		65		80					
		Conventional	Incremental	Conventional	Incremental	Conventional	Incremental	Conventional	Incremental				
5.97	very heavy heavy	0.055	0.066	2.54	2.3	6.88	5.7	15.5	13.5				
			0.065		2.4		5.8			13.7			
5.18	very heavy heavy medium	0.050	0.082	2.42	1.9	6.32	5.0	14.2	12.5				
			0.078		2.0		5.4			13.1			
3.54	heavy medium light	0.066	0.070		2.2		6.0		15.9				
			0.184		2.2		8.1			37			
2.38	heavy medium light	0.072	0.141	2.45	2.4	6.40	9.0	15.5	42				
			0.098		2.6		10.1			—			
			0.137		4.1		19.6			87			
			0.104	2.56	4.4	7.60	22.5	20.0	68				
			0.065		3.8		13.9		—				

5. IMPLICATIONS FOR PAVEMENT DESIGN

It is important to examine the possible consequences for pavement design of using incremental rather than conventional damage factors. Table 5 compares the standard axles per 100 commercial axles (or axle damage factors) indicated by the incremental approach for the four wheel-load spectra trafficking the four pavements with the axle damage factors used in Road Note 29⁵; the latter are independent of the strength of the pavement to which the spectrum is applied. Close agreement is obtained except for the medium and heavy spectra of wheel loads applied to the weaker pavements, when incremental analysis gives a much greater damage factor. If the incremental analysis had been carried out for the heaviest spectrum applied to the weak pavement, similar or possibly greater differences would have emerged.

TABLE 5

Comparison of the axle damage factors of different wheel-load spectra derived by conventional and incremental analysis

Wheel load spectrum	Conventional standard axles /100 axles	Incremental analysis standard axles/100 axles			
		very strong pavement	strong pavement	medium pavement	weak pavement
very heavy	74	67.6	66.2	—	—
heavy	61	56.7	54.5	74.3	124.4
medium	30	—	30	40.3	58.5
light	8	—	—	10.6	10.1

The thickness recommendations in Road Note 29 were prepared from an analysis of the behaviour of full-scale road pavements covering a wide range of pavement thickness under traffic best characterised by an axle damage factor of between 20 and 50 standard axles/100 axles. Weak pavements subjected to this traffic would appear to have an additional inbuilt margin of safety when their performance is analysed using the incremental approach. In practice they have not, because both the evolution of the design standards and the design of an individual pavement are equally affected by the difference between the incremental and the conventional approach. However the interpretation of the results of one or two full scale road experiments containing thin pavements will require re-appraisal to avoid possible under design when the results are used as part of any extrapolation process to produce designs for heavy traffic conditions.

Whilst the use of incremental analysis should not change design thicknesses or design lives for typical pavements under most mixed traffic conditions, the design of roads to carry some types of specialised traffic may require re-assessment. Pavements of moderate thickness that are intended to carry a relatively limited number of heavy wheel loads above the present legal limit may need a more conservative design approach; likewise the consequences of overloading on weak pavements carrying lighter traffic would be greater than indicated by the fourth-power law.

6. CONCLUSIONS

1. The fourth-power relation between the magnitude of a wheel load and its ability to damage a flexible pavement was derived from testing experimental pavements at the AASHO Road Test to failure under repeated applications of wheel loads of one magnitude only. The re-analysis of the results retains the basic form of the power relationships assumed in the original analysis between wheel load, layer type and thickness, and the rate of deterioration of the pavement, but takes into account the fact that in practice a pavement normally deteriorates under the action of a wide spectrum of wheel loads. Damage equivalences are calculated that are different from those derived by the conventional analysis.
2. Analysis of the effect of different traffic spectra on four pavements covering a wide range of pavement strength does not yield a single power law; neither, in most cases, does a single exponent characterise the response of a particular pavement to a spectrum of traffic. Effective exponents in relation to a standard reference wheel load of 40 kN are between 2.4 and 6.6, the higher values being for wheel loads in excess of the present UK legal limit operating on relatively weak pavements. The lowest values of exponent are an apparently unrealistic consequence of the basic formulation of the original AASHO damage equations.
3. Comparison of wheel-load equivalence factors obtained by incremental analysis with those from the conventional derivation (quoted in Road Note 29) indicates reasonable agreement except for the medium and heavy spectra of wheel loads applied to the weakest pavements when the incremental analysis indicates a greater damaging effect.
4. Pavements that are intended to carry a limited number of heavy wheel loads may need to be of greater thickness and the consequences of overloading on weak pavements designed for lighter traffic would be more serious than indicated by the fourth-power law.

7. ACKNOWLEDGEMENTS

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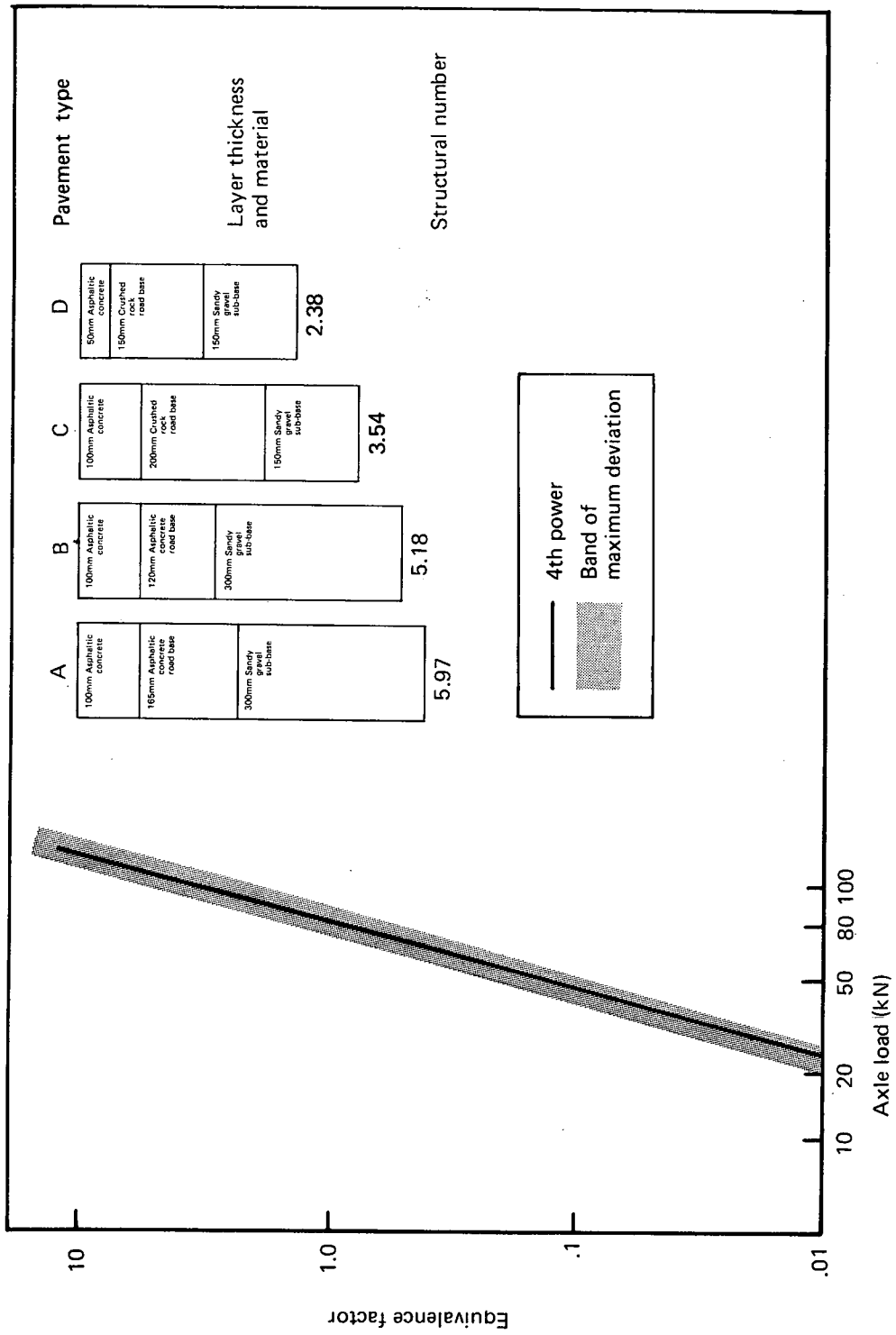


Fig. 1 VARIATION OF EQUIVALENCE FACTOR WITH AXLE LOAD FOR 4 PAVEMENTS OF DIFFERENT STRENGTHS. BASED ON AASHO ROAD TEST EQUATION

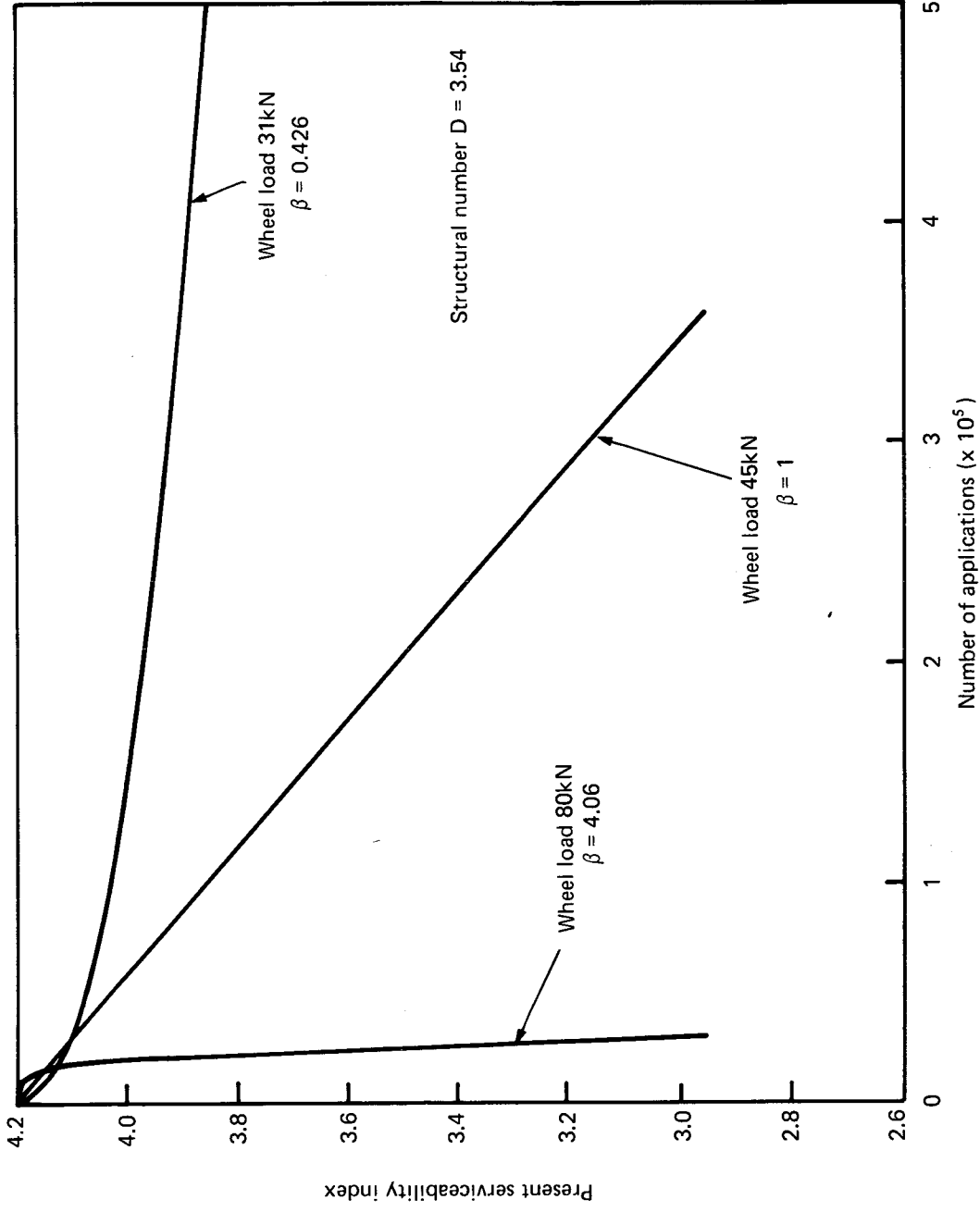


Fig. 2 COMPARISON OF SERVICEABILITY TRENDS FOR THREE WHEEL LOADS AS DERIVED FROM AASHO ROAD TEST EQUATION

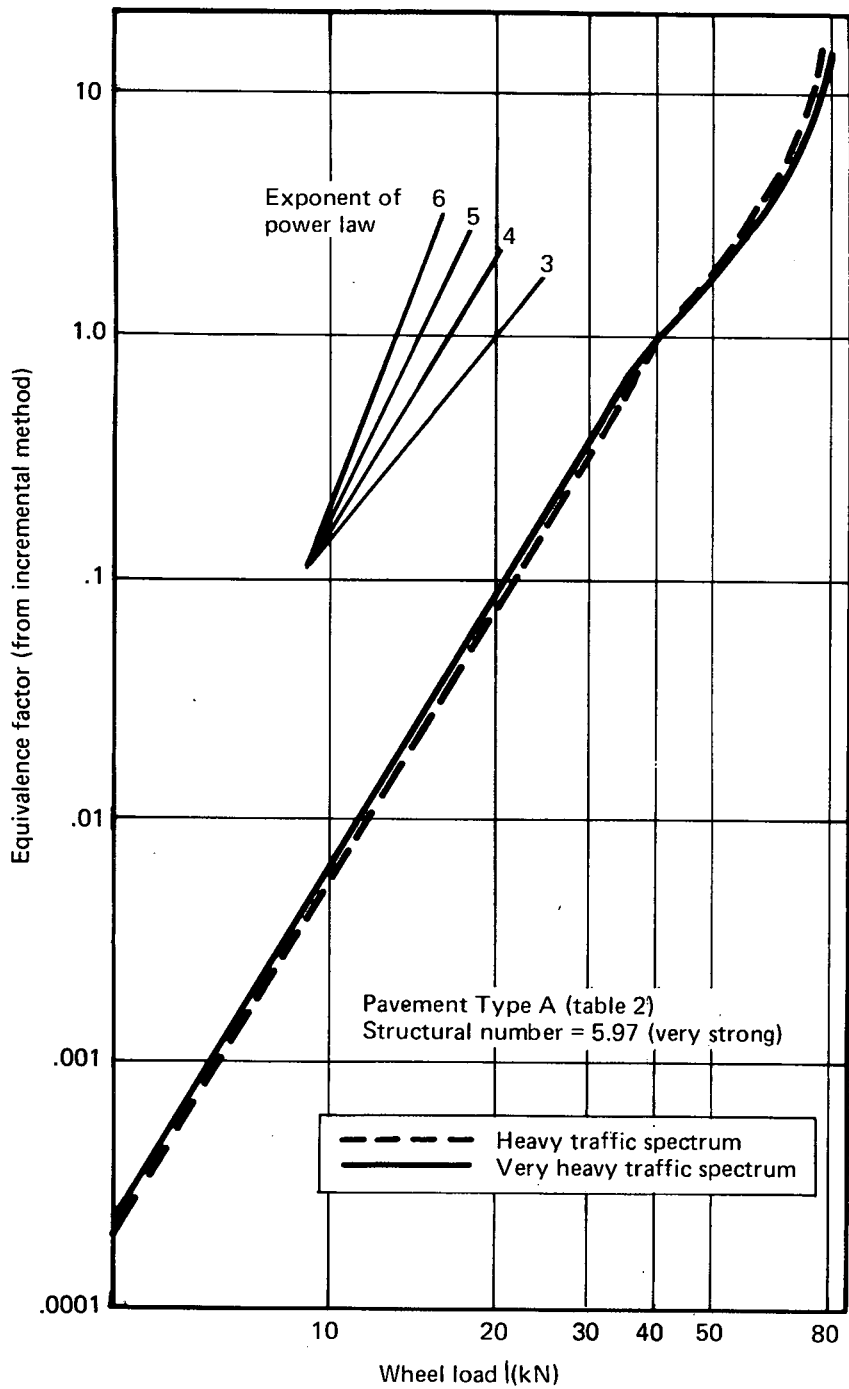


Fig. 3 VARIATION OF INCREMENTAL EQUIVALENCE FACTOR WITH WHEEL LOAD UNDER MIXED TRAFFIC CONDITIONS ON A VERY STRONG PAVEMENT

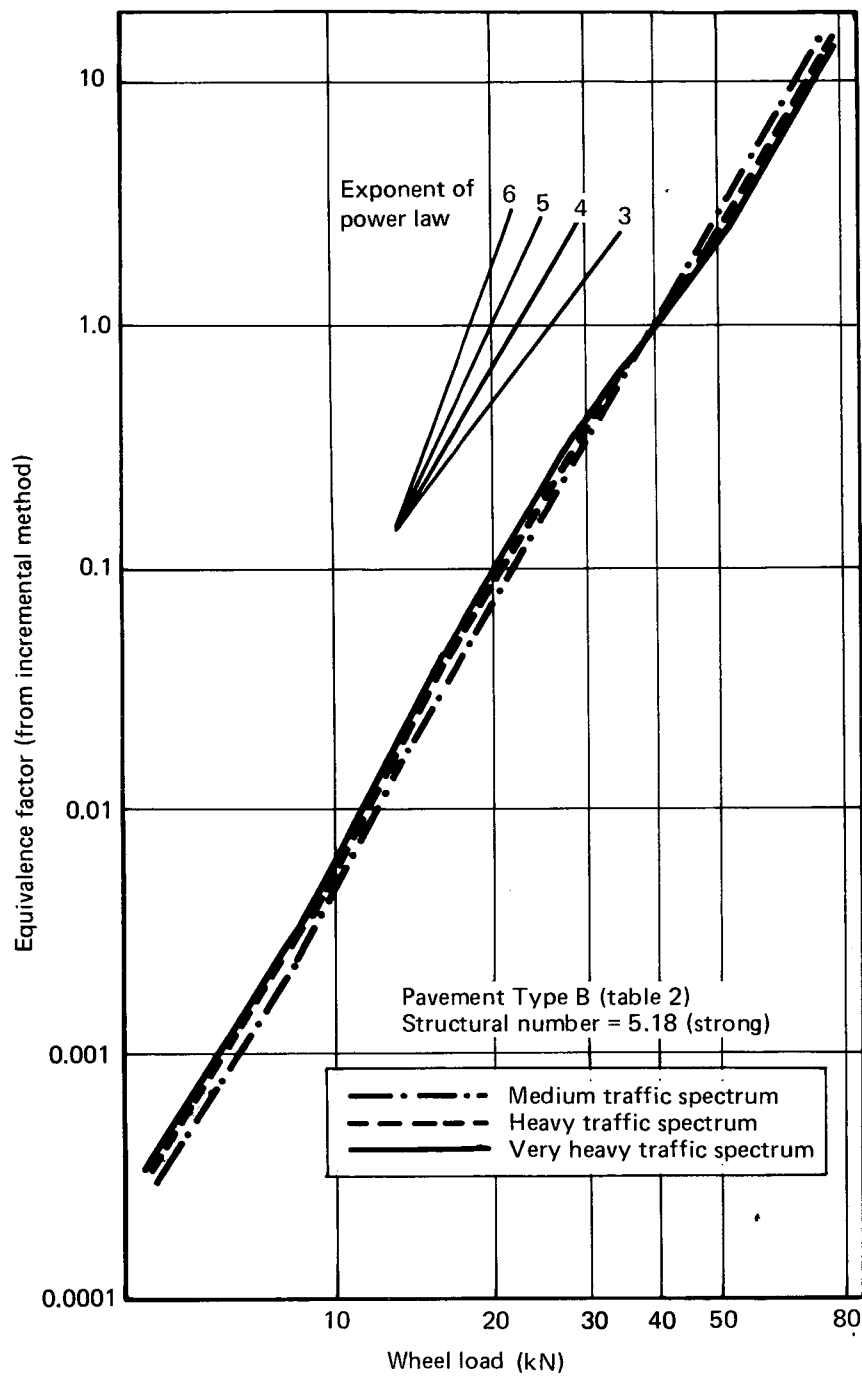


Fig. 4 VARIATION OF INCREMENTAL EQUIVALENCE FACTOR WITH WHEEL LOAD UNDER MIXED TRAFFIC CONDITIONS ON A STRONG PAVEMENT

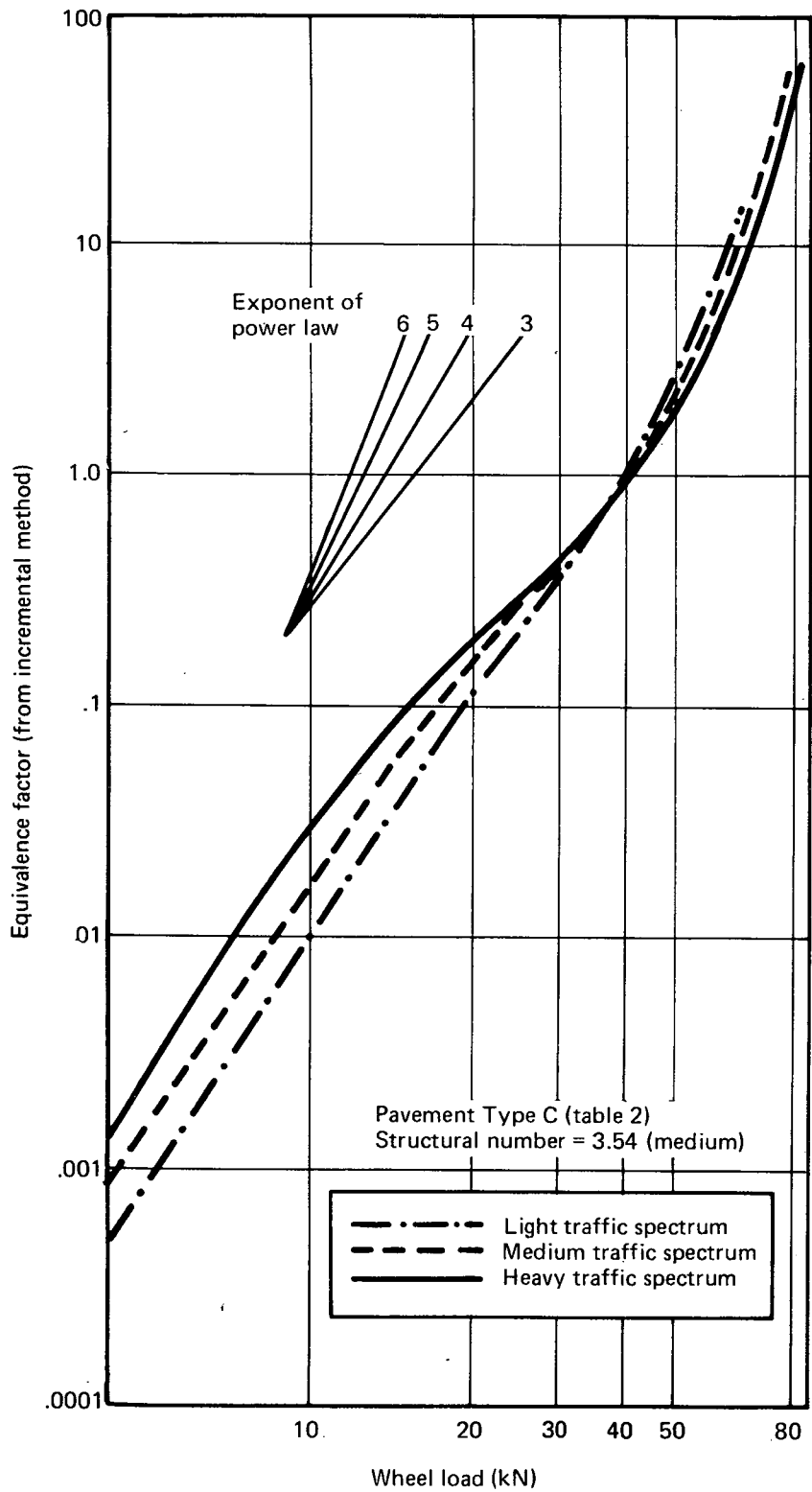


Fig. 5 VARIATION OF INCREMENTAL EQUIVALENCE FACTOR WITH WHEEL LOAD UNDER MIXED LOADING CONDITIONS ON A PAVEMENT OF MEDIUM STRENGTH

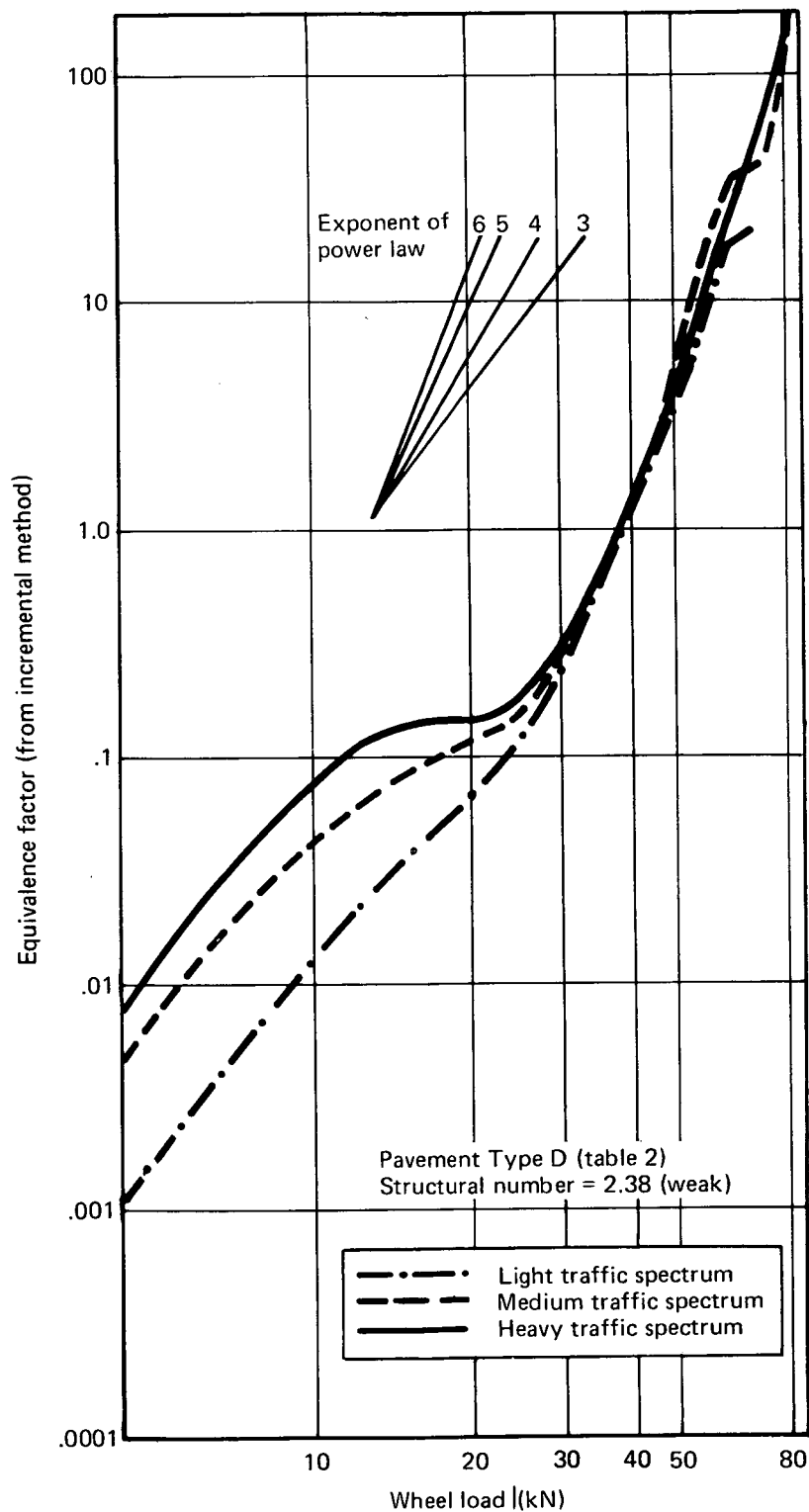


Fig. 6 VARIATION OF INCREMENTAL EQUIVALENCE FACTOR WITH WHEEL LOAD UNDER MIXED TRAFFIC CONDITIONS ON A WEAK PAVEMENT

ABSTRACT

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