RESEARCH REPORT **RR 91/155**

DEPARTMENT OF TRANSPORT

Equivalent damage of loads on pavements

MAY 1995

TITEL/TITLE:	Equivalent damage of loads on pavements		
VERSLAG NR: REPORT NO: RR 91/155	**ISBN:**	**DATUM: DATE:** May 1995	**VERSLAGSTATUS: REPORT STATUS:** Final

NOAK NR/RDAC NO: 91/155

GEDOEN DEUR:
CARRIED OUT BY:
Division of Roads and Transport Technology
CSIR
P O Box 395
PRETORIA
0001

OPDRAGGEWER:
COMMISSIONED BY:
Director General : Transport
Private Bag X193
PRETORIA
0001

OUTEUR(S):
AUTHOR(S):
J A Prozzi
P J Sanders
G D du Toit
M de Beer

NAVRAE:
ENQUIRIES:
Department of Transport
Directorate : Transport Economic Analysis
Private Bag X193
PRETORIA
0001

SINOPSIS:

Hierdie verslag beskryf 'n nuwe metode vir die bepaling van Ekwivalente Skade Faktore (ESF) van veral swaarvoertuie met meerassige- en wielkonfigurasies op plaveisels. Die metode is gebaseer op die begrip van "ekwivalente plaveiselresponse impliseer ekwivalente plaveiselskade". 'n Ondersoek na bestaande metodes het aangedui dat beide oorsese en lokale metodes nie ten volle geskik is vir die bepaling van ekwivalente skade aan plaveisels as gevolg van voertuie met meerassige- en wielkonfigurasies nie. Hierdie studie beklemtoon verder die noodsaaklikheid van die toepasbaarheid van die Suid-Afrikaanse Meganistiese Ontwerp Metode vir die bepaling van ekwivalente faktore. Die nuwe metode wat ontwikkel is, is gebaseer op verskeie plaveisel-skadewette (oordragsfunksies) en kan moontlik ook gebruik word om toelaatbare asmassas vir voertuie met meerassige- en wielkonfigurasies anders as die standaard te bepaal.

SYNOPSIS:

This report describes a new methodology for the determination of Equivalent Damage Factors (EDFs) of vehicles with multiple axle and wheel configurations on pavements. The basic premise of this new procedure is that "equivalent pavement response implies equivalent pavement damage". An analysis of currently used methods in South Africa and an overview of existing procedures, both locally and internationally, made evident the lack of a suitable technique capable of assessing the determination of equivalent factors for multiple axle and wheel configuration. This study covers this necessity by extending the applicability of the South African Mechanistic Design Method to the determination of equivalency factors. The method is based on locally obtained pavement damage laws (transfer functions) and might also help in the determination of permissible load for wheel and axle configurations different than the standard.

TREFWOORDE: Las-ekwivalensie faktore, meerassige- en wielkonfigurasies, plaveiselresponse, ekwivalente respons, ekwivalente skade, E80

KEYWORD: Load equivalency factors, multiple wheel and axle configurations, pavement response, equivalent response, equivalent damage, E80

VERSLAGKOSTE:
REPORT COST:

DISCLAIMER

The views and opinions expressed in this report are those of the author(s) and do not represent Department of Transport Policy.

ACKNOWLEDGEMENTS

The assistance provided by the following persons is gratefully acknowledged:

Mr. Colin Fisher, Division of Roads and Transport Technology, CSIR.

REVIEWED BY:

Dr. G. J. Jordaan, African Consulting Engineers INC, P. O. Box 36439, Pretoria.
Dr. P. J. Strauss, B K S INC Consulting Engineers, P. O. Box 3173, Pretoria.
Dr. J. P. Lourens, B K S INC Consulting Engineers, P. O. Box 3173, Pretoria.

LIST OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Page

SUMMARY

The derivation of equivalent damage factors has been approached by various organisations using different techniques over the past thirty or more years. Typically a load-based approach has been used which principally originated from the AASHO road test findings in the late 1950s and early 1960s. The latest approaches base the determination of load equivalency factors on pavement response (mainly measured surface deflections). After an overview of existing methods was done, their applicability to local conditions was analyzed based on a field test. In view of the identified limitations of these procedures an alternative method was formulated to overcome these shortcomings. In **Section 1** a short introduction to the present study is given.

A general overview of existing methods is presented in **Section 2.** For convenience the different techniques and approaches are grouped under the three main headings, i.e. **"Empirical"**, **"Theoretical"** and **"Mechanistic"**. Under the **"Empirical"** heading design methods developed from the AASHO Road Test data, the California Division of Highways and the U.S. Corps of Engineers are discussed as well as the "4^{th} Power Law" which was principally derived from the AASHO Road Test.

The **"Theoretical"** approach discusses various methods used by different organisations to calculate Load Equivalency Factors (LEFs) using combinations of theory, laboratory and field test results. Methods using response parameters such as tensile asphalt strain and compressive subgrade strain are briefly discussed as well as an approach that considers the rate of change of applied energy to pavement layers.

The **"Mechanistic"** approach has the primary distinction that response and distress parameters are measured in-situ and are not simply calculated. The response and distress parameters are then used in modelling pavement performance. Details are given of studies by Christison (1986), Rilett and Hutchinson (1988) and Scala and Potter (1981) which (*inter alia*) suggest ways of incorporating multiple axles into LEF calculations. The **"Empirical-Mechanistic"** method used in South Africa (which takes deformation as opposed to serviceability for definition of pavement damage) is also discussed. The variation in damage coefficients for different pavements is here noted. The work of Shackelton (1990) shows that a damage coefficient of d $\approx$ 2-3 in $(P/80)^d$ seems to hold true for pavements with granular bases and subbases. A

provisional suggestion is made to further investigate the possibilities of using Christison's 1986 deflection-based method.

In **Section 3** the applicability of deflection-based methods for LEF determination is discussed. Factors influencing pavement deflections, response and performance are also discussed in some detail. These include pavement type, material type and load history, moisture condition, temperature, tyre pressure and type, contact area and load intensity, vehicle speed and load application rate, wheel and axle configuration and the measurement technique used. The need for a method that incorporates variable axle spacings, wheel loads and failure modes pertinent to specific pavement types is noted. A comparison of prediction methods using locally obtained deflection data is given and discussed. In particular, Christison's 1986 deflection based-method is compared with predictions using the "4^{th} Power Law", the method proposed by the California Division of Highways and the HVS-based method. Except for the analysis of single axles with dual wheels, no clear trend is obvious, indicating the importance of incorporating wheel and axle configurations into calculations.

In **Section 4** an alternative method for calculation of equivalent number of 80 kN axles (E80s) recommended for use in South African conditions is given. It is a response-based method using calculated stresses and strains as response parameters. These parameters were chosen in preference to deflections as they differentiate between pavement failure modes, which are dependent on (*inter alia*) pavement type, load configuration and environmental factors. The essential feature of the new approach is that an alternative to the traditional load equivalence factor (LEF) is calculated by combining partial factors that take into account the effects of axle group loading (GEF), the load magnitude (ALF) and tyre pressure (TPF). Calculations are based on linear elastic theory and functions relating response parameters to pavement lives obtained from laboratory test results, empirical *in-situ* data and literature. Thus, the method is applicable to static and slow moving wheel loads.

The term **"Equivalent Damage Factor"** (EDF) is used in place of LEF to describe the combination of the partial factors. Failure modes considered are those of shear for granular bases, rutting in granular and cement stabilised materials and fatigue failure in asphalt and cement stabilised materials, (failure conditions being adjusted for different pavement categories). Curves relating damage of axles with single-wheels under various loads to standard axle configurations have also been drawn up. This differs from the "Equivalent Single Wheel Load" approach traditionally used to compare and design for various axle groups, in that two single wheels are used in place of one single wheel.

The method has been derived from the empirical-mechanistic design method used currently in South Africa. Therefore for use in conditions other than those typically encountered in South Africa, further commissioning of the proposed method is required.

Guidelines for using the method are given in **Section 5** as well as a worked example using a typical heavy vehicle with single, tandem and tridem axle configurations on four of the pavement types investigated.

Finally, conclusions and recommendations are presented in **Sections 6** and **7**.

1. INTRODUCTION

Pavements are designed with the primary aim of withstanding a given amount and composition of traffic loading before a specified level of distress occurs. The time span in which such a terminal level of distress occurs, is referred to as "structural life". The performance of a pavement therefore refers to the ability of a pavement to meet these design objectives. This might seem a very logical and simple approach, but bearing in mind that traffic as encountered on highways and all trafficked routes comprises of a whole spectrum of axle and load configurations which in turn is subjected to a host of environmental influences, this "simple approach" becomes rather complicated. Deacon (1963) stated that the nature of highway traffic loading and the climatic environment is such that any point within a pavement is subjected to a diverse and almost infinite spectrum of stresses and strains. Analysis of this complex loading is facilitated by expressing the damaging effect of all loads in terms of the equivalent number of applications of a standard or base load; this conversion is normally done by means of *Load Equivalency Factors* (LEFs).

Recent suggestions to change the maximum axle load limits on South African roads have sparked a renewed interest into the methods to quantify traffic load associated damage on pavements. Methods to quantify the damaging effects of mixed traffic on pavement structures, are of great importance to both pavement engineers and road authorities. For pavement engineers the interest stems from managing the network, while road authorities wish to recover the cost of damage. Therefore methods used to calculate LEFs should be evaluated in terms of their ability to deal with all practical load and axle arrangements of mixed traffic. This will then permit the development of guidelines on permissible axle loads for different axle arrangements.

The method currently used in South Africa to calculate LEFs is based on the "power law" relationship (Walker et al, 1977; Maree, 1982; CSRA, 1989) which was derived from the AASHO Road Test and experience gained on pavement performance by the California Division of Highways in the 1960s. Extensive research with the Heavy Vehicle Simulator (HVS) on existing pavement sections in South Africa over the past years has led to a more fundamental understanding of pavement performance and permitted the development of load equivalency factors for single-axle dual-wheel loads. A major limitation of the HVS-based method is that it does not facilitate the calculation of damage associated LEFs for multiple axle arrangements (HVS only simulates single axle loading conditions). Thus, an alternative method and/or means to extend the existing HVS-based method is needed when multiple axle are to be considered.

The aim of this study is to identify existing methods which predict pavement damage caused by the passage of various axle and load configurations and (if appropriate) improve and develop them. Most of the existing methods use damage parameters such as serviceability, stresses, strains, permanent deformation or elastic deflections and each method has its own inherent limitations. The potential applicability of these methods to South African conditions is to be reviewed in this research project in an attempt to identify a method which can be used to extend the HVS results towards multiple wheel and axle load configurations. The focus is to be placed on pavement response based methods, such as those based on measured surface deflections. Field work is to be carried out to compare methods and to assess their applicability to South African conditions.

Once the advantages and disadvantages of the different methods are identified, an alternative method and/or guidelines for the determination of load equivalency factors are to be formulated. The new approach should be capable of overcoming the limitations of those existing. The basic principle pertinent to this approach should be that equivalent pavement response (however defined) implies equivalent pavement damage. Whatever the new methodology for the determination of load equivalency factors is, it will be based on pavement response parameters and will enable the prediction of LEFs based on the traffic load configuration.

Finally, this methodology is to be presented in an easily usable form so as to be used by designers as well as legislators to derive equivalent and/or permissible loads on the current South African road network.

2. EXISTING METHODS TO DERIVE EQUIVALENCY FACTORS

Various methods exist which attempt to quantify the structural damaging effect of different wheel and axle load arrangements on pavements. As these methods encompass different approaches and use different parameters to relate mixed traffic situations to pavement damage, some understanding regarding the methodology and assumptions pertinent to these methods is necessary in order to evaluate them. Several parameters have been developed to quantify pavement performance (functional and structural), therefore their ability to represent pavement performance as related to the actual field performance of the pavement as a system, needs to be assessed.

The following discussion describes some of the methods adopted by different researchers, research institutions and road authorities to derive load equivalency factors (LEFs). The methods adopted can, for explanatory purposes, be grouped into three categories: empirical, theoretical and mechanistic approaches. Special reference is made to the Heavy Vehicle Simulator (HVS) based method used in South Africa. Empirical methods were developed from observed performance during testing of experimental pavement sections and applying "best fit" mathematical relationships to the measured data. The mechanistic approach is similar to the theoretical approach, with the primary difference being that the distress parameters (pavement response parameters) are measured in situ and not calculated. As some of these methods are in fact a combination of different approaches, they will be categorised according to the predominant approach used. These categories are by no means exhaustive. Reference to the limitations and applicability of these methods are also made and given in the respective discussions.

2.1 EMPIRICAL APPROACH

Probably one of the most comprehensive sources of data on the damaging effect of wheel loads on pavements, stems from the findings of the (then) American Association of State Highway Officials (AASHO) Road Test conducted from October 1958 to November 1960 near Ottawa in the state of Illinois (HRB, 1962a). Some discussion on the approach adopted during the execution of the AASHO Road Test is warranted, as the findings and the data obtained during this full scale experiment were used as reference for many load-damage postulations developed thereafter.

2.1.1 The AASHO Road Test - Serviceability

The test sections used for the analysis in the AASHO Road Test comprised of a total of 468 flexible pavements sections and 368 rigid pavement sections built into six test loops. At the termination of the test a total of 1 114 000 repetitions of various loads had been applied to the sections. The applied load ranged from 900 kg (single axle) to 22 000 kg (tandem axle). Considerable overlap was provided by means of control sections which were built across the test loops to facilitate an assessment of the effect of axle loading on the same pavement design. One of the essential features of the Road Test was the establishment of a definition of pavement performance and a means of measuring this, known as the Pavement Serviceability-Performance System. A term called "present serviceability" was used to objectively evaluate the performance of the pavement sections in terms of their ability to serve high volume, high speed mixed traffic. A rating system (Present Serviceability Rating - PSR) of 0 to 5, where a rating of 5 indicated a perfect road and 0 an impassable road, was developed (Carey and Irick, 1960) to evaluate pavement deterioration under repetitive loading in terms of the:

(a) Longitudinal profile variations using slope variance measured with the CHLOE profilometer to present roughness;

(b) Rutting;

(c) Degree of cracking; and

(d) Patching.

The individual ratings for each of the sections were averaged and converted to a Present Serviceability Index (PSI) to reflect objective measurements by equipment and evaluators, through use of the following equations:

For flexible pavements:

$$PSI = 5{,}03 - 1{,}91 \log (1 + \overline{SV}) - 0{,}01 \sqrt{C + P} - 1{,}38 \overline{RD}^2 \qquad \text{... Eq. (2-1)}$$

For rigid pavements:

$$PSI = 5{,}41 - 1{,}80 \log (1 + \overline{SV}) - 0{,}09 \sqrt{C + P} \qquad \text{... Eq. (2-2)}$$

where: SV = slope variance

C = degree of cracking

P = degree of patching

RD = average rut depth

The Present Serviceability Index (PSI) was then used to evaluate and compare the performance of the respective sections.

The general design equation developed from the AASHO Road Test, which incorporates the serviceability-performance concept (HRB 1962a; Langsner et al, 1962; Bartelsmeyer and Finney, 1962), is given by:

$$G = \log \left(\frac{c_0 - p}{c_0 - c_1} \right) = \beta \left(\log W - \log \rho \right) \qquad \text{... Eq. (2-3)}$$

or if re-written, denotes the following:

$$\log W = \log \rho + \frac{G}{\beta} \qquad \text{... Eq. (2-4)}$$

where: G = a function (the logarithm) of the ratio of loss in serviceability at any time to the potential loss taken to a point where p = 1,5;

β = a function of design and load variables that influences the shape of the p vs. W serviceability curve;

W = weighted traffic factor (the number of seasonal load applications multiplied by a seasonal weighting function);

ρ	=	a function of design and load variables that denotes the expected number of load applications to a serviceability index of 1,5;
p	=	serviceability at a given time;
c_0	=	initial serviceability value; and
c_t	=	terminal serviceability level - serviceability level (1,5) at which test sections were removed from the experiment.

For weighted load applications (seasonal effects are accounted for by multiplying the number of applications by a seasonal factor) and for flexible pavements:

$$\beta = 0{,}40 + \frac{0{,}081\ (L_1 + L_2)^{3{,}23}}{(SN + 1)^{5{,}19}\ L_2^{3{,}23}} \qquad \text{... Eq. (2-5)}$$

and

$$\log \rho = 5{,}93 + 9{,}36 \log (SN + 1) - 4{,}79 \log (L_1 + L_2) + 4{,}33 \log L_2 \qquad \text{... Eq. (2-6)}$$

where:	L_1	=	load on one single-load axle or on one tandem-axle set, in kips;
	L_2	=	axle code, 1 for single; 2 for tandem;
	SN	=	structural number = $a_1 D_1 + a_2 D_2 + a_3 D_3$;
	a_1, a_2, a_3	=	material coefficients determined in the Road Test;
	D_1	=	thickness of bituminous surface course, in inches;
	D_2	=	thickness of base course, in inches;
	D_3	=	thickness of subbase, in inches;

For weighted load applications and for rigid pavements:

$$\beta = 1 + \frac{3{,}63\ (L_1 + L_2)^{5{,}20}}{(D + 1)^{8{,}46}\ L_2^{3{,}52}} \qquad \text{... Eq. (2-7)}$$

and

$$\log \rho = 5{,}85 + 7{,}35 \log (D + 1) - 4{,}62 \log (L_1 + L_2) + 3{,}28 \log L_2 \qquad \text{... Eq. (2-8)}$$

where: L_1 = load on one single-load axle or on one tandem-axle set, in kips;
L_2 = axle code, 1 for single; 2 for tandem;
D = thickness of rigid (concrete) slab, in inches.

Determination of equivalence factors for flexible pavements were then obtained by using the general AASHO design formula and substituting the corresponding mathematical values of ρ into it, which gives the following:

$$\log W = 5{,}93 + 9{,}36 \log (SN + 1) - 4{,}79 \log (L_1 + L_2) + 4{,}33 \log L_2 + \frac{G}{\beta} \qquad \text{... Eq. (2-9)}$$

When L_1 = 18 kips (18 000 pound reference axle load) and L_2 = 1 (single axles):

$$\log W_{18} = 5{,}93 + 9{,}36 \log (SN + 1) - 4{,}79 \log (18 + 1) + \frac{G}{\beta_{18}} \qquad \text{... Eq. (2-10)}$$

When $L_1 = L_y$ (any other axle load) and L_2 = 1 (single axle):

$$\log W_y = 5{,}93 + 9{,}36 \log (SN + 1) - 4{,}79 \log (L_y + 1) + \frac{G}{\beta_y} \qquad \text{... Eq. (2-11)}$$

Subtracting log W_y from log W_{18}, for single axles:

$$\log \frac{W_{18}}{W_y} = 4{,}79 \log (L_y + 1) - 4{,}79 \log (18 + 1) + \frac{G}{\beta_{18}} - \frac{G}{\beta_y} \qquad \text{... Eq. (2-12)}$$

Similarly, when $L_1 = L_y$ and $L_2 = 2$ (tandem axles):

$$\log W_y = 5{,}93 + 9{,}36 \log (SN + 1) - 4{,}79 \log (L_y + 2) + 4{,}33 \log 2 + \frac{G}{\beta_y} \qquad \text{... Eq. (2-13)}$$

Subtracting log W_y (tandem axles) from log W_{18} (single axles):

$$\log \frac{W_{18}}{W_y} = 4{,}79 \log (L_y + 2) - 4{,}79 \log (18 + 1) - 4{,}33 \log 2 + \frac{G}{\beta_{18}} - \frac{G}{\beta_y} \qquad \text{... Eq. (2-14)}$$

Using the same procedure as the preceding, the subsequent equation for rigid pavements is given by:

$$\log \frac{W_{18}}{W_y} = 4{,}62 \log (L_y + 2) - 4{,}62 \log (18 + 1) - 3{,}28 \log 2 + \frac{G}{\beta_{18}} - \frac{G}{\beta_y} \qquad \text{... Eq. (2-15)}$$

The ratio between W_{18} and W_y expresses the relationship between the 18 000-lb (80 kN) single axle load and any other axle load L_y, single or tandem. The ratio becomes the equivalence factor and may be evaluated by solving the respective equations for different values of L_y. Since ρ and β vary with SN, L_y and L_2, and G depends on both the initial (p) and the chosen terminal serviceability level (c_1), solving of the respective equations to derive equivalence factors may become an elaborated procedure. However, the AASHTO Interim Guide for Design of Pavement Structures 1972 (AASHTO, 1974) facilitates this procedure by tabulating calculated LEFs for a range of loads (L_y) on both single and tandem axles, for a range of pavement structures (SN = 1 to 6) and terminal serviceability (c_1) indices of 2,5 and 2,0. Thus, LEF is obtained for different pavement structures, single or tandem axle loads, and for two possible terminal serviceability levels. The AASHTO

Guide for Design of Pavement Structures 1986 (AASHTO, 1986) extends the table to three terminal serviceability values (2,0; 2,5 and 3,0), single, tandem and tridem axles and six pavement structural numbers.

2.1.2 The Fourth Power Law

A subsequent analysis of the AASHO Road Test data (Irick and Hudson, 1964) probably initiated one of the most widely used and accepted equivalency factor formulae. From their analysis they indicated that the effects of pavement structure and decrease in PSI were comparatively minor and that the following relations gave satisfactory approximations to the LEFs:

$$F = \left(\frac{L}{80}\right)^4 \qquad \textit{for dual-tyred single axles} \qquad \text{... Eq. (2-16)}$$

and

$$F = \left(\frac{L}{147}\right)^4 \qquad \textit{for dual-tyred tandem axles} \qquad \text{... Eq. (2-17)}$$

where:	F	=	the load equivalence factor (LEF)
	L	=	applied axle load
	80	=	reference dual-tyre single axle load in kN at the Road Test
	147	=	reference dual-tyre tandem axle load in kN at the Road Test

These relations were recommended as being sufficiently precise for use in proposed satellite studies following the AASHO Road Test (Irick and Hudson, 1964) and have since received very wide acceptance, being referred to as the "fourth power law".

2.1.3 California Division of Highways (CDH)

Hveem and Sherman (1963) developed a similar power law relationship which is based on the equivalent gravel concept (EG) (design method used by the California Division of Highways) and data obtained from the AASHO Road Test. In their approach adopted, the equivalent thickness of gravel which could result in the same cohesiveness as the pavement structure being analyzed is calculated from the following general formula:

$$GE = \left(\frac{c}{cohesion\ of\ gravel} \right)^{0,2} \qquad \text{... Eq. (2-18)}$$

where: c = equivalent cohesion

Relating the gravel equivalent of the individual AASHO Road Test sections to the number of applications at present serviceability index (PSI) = 2,5, some adjustments to the coefficients of the 1957 California formula for calculating traffic index was necessary and the subsequent formula[1] reduced to:

$$T = C \times W^{0,5} \times r^{0,119} \qquad \text{... Eq. (2-19)}$$

where: T = thickness (in inches)

C = constant

W = wheel load (in kips)

r = repetitions

---

[1]0,119: approximate "reasonable value" based on slope of Number of Applications at Present Serviceability Index ($PSI_{2,5}$) vs. Gravel Equivalent curve.
0,5: "theoretical value" (actual value = 0,48) obtained from slope of Wheel Load vs. Gravel Equivalent curves.

Thus for different wheel loads (mixed traffic) the following apply:

$$\frac{T_1}{T_2} = \left(\frac{W_1}{W_2}\right)^{0,5} \times \left(\frac{r_1}{r_2}\right)^{0,119} \qquad \text{... Eq. (2-20)}$$

If $T_1 = T_2$; $W_1 = 5000$ lb (reference wheel load used by the CDH) and r_2 = one repetition of wheel load W_2, then:

$$r_1 = \left(\frac{W_2}{5}\right)^{4,2} \qquad \textit{Equivalent 5 kips wheel load (EWL)} \qquad \text{... Eq. (2-21)}$$

This designated load of approximately 22 kN represents a single wheel load of 20 kN (standard load on each of the individual wheels of a 80 kN single axle - dual wheel configuration) with a corresponding arbitrary 10 per cent increase to allow for tandem effect. As the method is basically a load ratio, any reference load may be used.

For single axles (reference axle load of 80 kN), pavements with a structural number (SN) of 3 and a terminal serviceability level of 2,0, the "Power Law" (n = 4,2) provides an accurate representation of the equivalence factors determined from the AASHO Road Test. Similarly, for a terminal serviceability level of 2,5 and a structural number of 4. In general, the "Power Law" provides a good estimation of the AASHO equivalence factors, regardless of the pavement structural number and the terminal serviceability level, viz values for the relative damage coefficient (n) vary between 3,8 and 4,7 (average ≈ 4,2). However, this is only applicable to single axles and subsequently, the corresponding values for tandem axles differ substantially.

2.1.4 U.S. Corps of Engineers - Equivalent thickness design

The U.S. Corps of Engineers has in the past, undertaken a considerable number of investigations into the relative damaging effect of wheel loads. The results of these studies have been incorporated into the following equation for airfield pavement thickness design (Brown and Ahlvin, 1961), i.e.:

$$t = (30,8 \log W + 20,1) \sqrt{P\left(\frac{1}{CBR} - \frac{17,8}{\rho}\right)} \qquad \text{... Eq. (2-22)}$$

where:

	t	=	thickness in mm
	W	=	number of traffic repetitions
	P	=	single or equivalent single wheel load in kN
	CBR	=	California Bearing Ratio of subgrade
	ρ	=	tyre pressure in kPa

Re-arrangement of the above equation allows the relative damaging effects of different wheel loads and tyre pressures on the same pavement ($t_1 = t_2$) to be assessed. Good correlation between LEFs calculated from this method and LEFs calculated from the California Division of Highways method (Hveem and Sherman, 1964) were reported (van Vuuren, 1972).

2.1.5 Discussion on the empirical approach

Empirically derived equations are obtained by applying "best fit" mathematical relationships to measured data. The variables in the equations are measured, and the constants, determining their mathematical dependency, are calculated by regression analysis. The main benefit of such methods, as opposed to theoretical methods, generally lies in the subsequent inclusion of the effects of environmental influences, although the general aim is to control these factors to some extent. However, it is also in this apparent benefit where the danger lies, as environmental effects may differ substantially from region to region, forcing the adaption or verification of the results obtained and the equations developed for application elsewhere. These methods, and relationships established, also need constant upgrading and modification (even for the regions for which they were developed) as changes in current practice (construction, materials used, vehicle design, computerised software, etc.) become apparent. Therefore, models developed entirely within one region typically lack a basis for extrapolation to conditions applying in other regions unless the empirical base embraces a wide range of conditions and factors.

Most of the methods discussed and represented in this section rely heavily on the data and findings of the AASHO Road Test. Some aspects regarding the applicability of the AASHO Road Test findings to conditions prevailing in South Africa therefore need some clarification. The AASHO Road Test was constructed in 1958, some 35 years ago and considerable improvements in construction techniques and equipment have taken place since then. Vast differences in the quality of the materials used for the construction of the AASHO Road Test sections and those commonly used in South Africa exist (Otté, 1972). Most of the AASHO Road Test sections failed while the pavement was thawing after the winter, a phenomenon not encountered locally.

Major limitations of the AASHO Road Test were identified. The test was conducted at a single site, in a single environment on a single subgrade (Hudson and Irick, 1985). The AASHO LEFs were derived from statistical analysis of empirical data. No attempt was made to distinguish between different modes of distress: the equivalency factors were related solely to performance as measured by the present serviceability index (PSI) (Deacon, 1963). Furthermore, the composition of the equations which were used to express the PSI is such that the calculated PSI is extremely reliant on the first term, i.e. slope variance. Curtayne and Walker (1972) found the cracking and patching term $\sqrt{(C + P)}$ to be statistically insignificant for flexible pavements. The rutting term (RD) was found to be significant, however, the rut depth on the road test was in general less than 5 mm when trafficking was terminated and therefore inaccuracies may be incorporated in the regression when large rut depths exist.

A simplified evaluation of the AASHO LEFs shows that these values are mainly influenced by the magnitude of the applied load regardless of the structural number and terminal serviceability level. A reasonable approximation can be obtained (for single axles) by applying the power law functions (Hveem and Sherman, 1963; Irick and Hudson, 1964), with values for the damage coefficient found, in essence, to be constant ($n = 4{,}2$ or $n = 4$ - actual determined values range between 3,8 and 4,6).

In general, simplification of the AASHO equations, as expressed by the 4^{th} power law relationship (Irick and Hudson, 1964) provides a reasonable approximation of the AASHO LEFs, however, a more recent evaluation of the AASHO Road Test results reported the statistical analysis to be erroneous (Irick and ARE Inc, 1989; Small et al, 1989). The revised value of the load based relative damage coefficient (n) in the relative damage equation (previously determined as 4) was reported to be three (3).

2.2 THEORETICAL APPROACH

In order to gain better insight into the mechanisms causing pavement damage, a considerable amount of laboratory work was carried out (Kennis, 1977; Barker et al, 1977; Brown et al, 1977; Brown and Bell, 1977) where specimens of pavement materials were subjected to repeated applications of controlled stress (or strain). Data of several experimental sections (of which the AASHO Road Test is the most prominent) was used to assist in developing computerised packages from which pavement behaviour and performance are modelled. Various parameters were used in the modelling efforts which relate common pavement failure mechanisms to the respective calculated parameters.

2.2.1 LEFs using tensile strains as the fatigue-damage parameter

Deacon (1969) computed maximum principle tensile strain under the bituminous surface layer and used these tensile strains as the fatigue damage parameter in calculating theoretical load equivalency factors (LEF), viz under simple loading conditions the fatigue life can be expressed by:

$$N = K\left(\frac{1}{\epsilon}\right)^{C} \qquad \text{... Eq. (2-23)}$$

where: N = number of applications to failure

ϵ = maximum principle tensile strain at bottom of bituminous layer

K and C = material constants (C generally ranges between 5 and 6 depending on the mixture composition)

Assuming that a single load, L_i, when repetitively applied to the pavement will cause fatigue failure after N_i applications and if the load were changed to the base load, L_b, fatigue failure would occur after N_b applications, the load equivalency factor for L_i is:

$$F_i = \frac{N_b}{N_i} \qquad \text{... Eq. (2-24)}$$

If many different loads were applied to the pavement, the total equivalent number of applications of the base load, E, becomes:

$$E = \sum_i F_i n_i \qquad \text{... Eq. (2-25)}$$

where: E = total number of equivalent applications of the base load

F_i = load equivalency factor for load L_i

n_i = number of applications of load L_i

A standard pavement section was chosen (corresponding structural number of 4), which served as the basis of comparison between three different axle configurations (single axles with single tyres, single axles with dual tyres and tandem axles with dual tyres) and a range in axle loads (1 to 17 kips). The analysis was performed by means of a computerised linear elastic multi-layer program, developed by the Chevron Research Company, and the effect of the dual, and dual-tandem configuration, was modelled by means of superposition of the stresses. These theoretical load equivalency factors were found to agree remarkably well with those established during the AASHO Road Test.

Several pavement performance models exist which are similar to the approach adopted by Deacon (1969), i.e. calculated tensile strains are used as the fatigue-damage parameter. These relationships are generally expressed in the form:

$$N \propto \varepsilon^{-c} \qquad \text{... Eq. (2-26)}$$

where: N = the number of load repetitions to fracture (fatigue)

ε = tensile strain

c = coefficient depicting mathematical dependency of parameter in the equation

In each case different values for the coefficient c were found, depending on the assumptions pertinent to the specific damage model and the approach adopted, viz laboratory test results, linear or non-linear material behaviour characterization, etc.. Different determined values for the coefficients c in the tensile strain-damage relationships are summarised in Table 2-1.

Table 2-1: Summary of tensile strain-damage coefficients used by various authors.

Source	Method of derivation	Value of exponent (c)
Finn and others (1977)	*Laboratory testing of sample specimens and data obtained from AASHO Road Test sections; developed computer program based on linear elastic theory with provision for stress sensitivity of unbound materials; traffic predictions in terms of single axle load applications; conversion of actual mixed traffic by means of AASHO LEFs.*	3.3
Carmichael and others (1977)	*AASHO Road Test pavements and other selected pavement trial sections; linear elastic theoretical models*	5.1
Claessen and others (1977)	*Laboratory testing and field trials; developed linear elastic, multi-layer system computer program (BISAR); spectrum of traffic converted to standard axles (80 kN single axle) using 4th power law*	6
Kennis (1977)	*Laboratory and field studies (mainly AASHO Road Test); probabilistic elastic and/or visco-elastic computer program (VESYS); mixed traffic converted by means of AASHO LEFs*	2.6 (example)
Brown and others (1977)	*Laboratory testing; elastic computer simulation (BISTRO); mixed traffic converted to equivalent single axle loads by means of AASHO LEFs*	2.9 - 4.9
Deacon (1969)	*Laboratory and field studies; used linear elastic based computer program; calculated LEFs*	5 - 6
Ullidtz (1977)	*In situ tests performed on selected pavements; uses elastic theory; mixed traffic converted to standard axle loads by means of any preferred method.*	5.6
Célard (1977)	*Laboratory testing and experimental test track results; apply theory of elasticity; traffic in terms of number of passes of simulated truck wheel connected to a rotating arm*	4.3
Mitchell and Monismith (1977)	*Laboratory tests and analysis of a single pavement structure; computer based layer elastic principles (ELSYM)*	20.3

Although these models predict pavement life to a specified level of distress to occur, they do not encompass the derivation of load equivalence factors. In general, to evaluate the accuracy and the validity of the models, the predicted traffic estimated with these models were compared with field studies in which mixed traffic load applications were converted to "equivalent axle loads" by means of previously established LEFs such as those developed during the AASHO Road Test (AASHTO, 1974) or AASHO-based LEFs (RRL, 1970).

Even though consideration of the proposition that equivalent pavement response implies equivalent pavement damage (Scala and Potter, 1981) does not yield to LEF calculations, extrapolation of these damage models may facilitate this determination, viz by simply expressing the calculated LEF as the ratio of damage predicted under an axle load of any magnitude and configuration to the damage predicted under a standard axle load.

Thus, if the number of applications (N_2) of a given load configuration to a specified level of distress (or response) can be expressed in terms of the number of applications (N_1) of any other reference load (to the same level of distress or response), the LEF can be calculated from:

$$LEF = \frac{N_1}{N_2} \qquad \text{... Eq. (2-27)}$$

Therefore, if a damage parameter and consequently a specific load-damage model is chosen in the form of Equation 2-26, substitution into Equation 2-27 represents the determination of load equivalency factors , i.e.:

$$LEF = \frac{N_1}{N_2} = \vartheta \left(\frac{\varepsilon_2}{\varepsilon_1} \right)^c \qquad \text{... Eq. (2-28)}$$

in which all notations have their previous meanings (as in Equation 2-26) and ϑ represents a factor which includes any other design variable(s).

2.2.2 LEFs using subgrade compressive strains as the deformation-damage parameter

Similarly, several damage models were developed in which permanent (plastic) deformation was considered as the primary failure mechanism. Laboratory and field observations revealed that good correlation exists between calculated subgrade compressive strains and level of deformation occurring and subsequently, these calculated strains were used as the damage parameter in the load-damage models.

The relationships are generally expressed in the form:

$$N \propto \varepsilon^{-c} \qquad \text{... Eq. (2-29)}$$

where: N = the number of load repetitions to the specified level of deformation

ε = vertical elastic (recoverable) strain

c = coefficient depicting mathematical dependency of parameter

Different values for the coefficient c were found, depending on the assumptions pertinent to the specific damage model and the approach adopted in the specific model (laboratory test results, linear or non-linear material behaviour characterization, etc.). The results of various authors are summarised in Table 2-2.

These models predict pavement life to a specified level of distress to occur but do not directly allow the derivation of load equivalence factors. Verification of the models in terms of predicted performance vs. actual performance requires the assumption of previously established relationships to convert mixed traffic load applications to equivalent axle load applications.

Table 2-2: Summary of subgrade vertical compressive strain-damage coefficients used by various authors.

Source	Method of derivation	Value of exponent (c)
Claessen and others (1977)	*Laboratory testing and field trials; linear elastic, multi-layer computer program (BISAR); conversion of mixed traffic to equivalent number of standard wheel passes by means of LEF equation developed by van de Loo (1976)*	4.0
Brown and others (1977)	*Laboratory testing; elastic computer simulation (BISTRO); mixed traffic converted to equivalent single axle loads by means of AASHO LEFs*	3.57
Kirwin and others (1977)	*Laboratory testing and field studies; used non-linear computer based model (DEFPAV) for analysis; used AASHTO LEFs to relate mixed traffic to laboratory load applications*	5.1

In their approach Claessen et al (1977) used a method developed by van de Loo (1976) whereby a relationship expressing mixed traffic in terms of the total number of wheel passes of the standard wheel is calculated by the following equation i.e.:

$$W_{eq} = 1{,}4\ W_{tot} \sum_{i=1}^{k} \left(\frac{\sigma_i}{\sigma_0} \right)^{\frac{1}{s}} \frac{n_i}{n_{tot}} \qquad \text{... Eq. (2-30)}$$

where: W_{eq} = total number of wheel passes of the standard wheel

W_{tot} = total number of axle loads over period

σ_i = contact stress between tyre and pavement of wheel load class I, in kPa

σ_0 = contact stress between tyre and pavement of the standard wheel (600 kPa)

n_i/n_{tot} = ratio of the number of wheel loads in class I to the total number of wheel

s = slope of the log S_{mix} - log S_{bit} curve determined by static creep testing

S_{mix} = stiffness of asphalt mix

S_{bit} = stiffness of bitumen

2.2.3 LEFs using subgrade deflection as the damage parameter

Jung and Phang (1974) used linear elastic multi-layer theory to estimate subgrade deflections for the AASHO Road Test sections and derived the following relationship:

$$N = \frac{1}{d^6 \times 10^{(K - 0{,}09\,P)}} \qquad \text{... Eq. (2-31)}$$

where: N = number of load repetitions to a specified serviceability index

d = vertical subgrade deflection under applied load in inches

P = wheel load in lb

K = a constant whose value depends on the terminal PSI value chosen

Using this relationship and applying the same method as outlined above, they derived the following equation for the calculation of load equivalency factors:

$$LEF = \left(\frac{d}{d_s} \right)^6 \times 10^{-0,09\,(P - P_s)} \qquad \text{... Eq. (2-32)}$$

where: d = vertical subgrade deflection under applied load (in inches)

d_s = vertical subgrade deflection under standard 18 kips load (in inches)

P = applied wheel load in kips

P_s = standard 18 kip wheel load[2]

2.2.4 LEFs using calculated stresses as the damage parameter

Apart from the magnitude of the applied force, Govind and Walton (1989) identified the rate at which the force is applied and removed to be of major importance in determining the extend of fatigue damage on asphalt mixes. Using the relationships developed and data from the AASHO Road Test and applying a computerized dimensional analysis technique, a damage model (D) was constructed based on the rate of change of applied energy (δσ/dt), viz:

$$D = \int_{t_0}^{t_1} \frac{1}{t_1 - t_0} \left| \frac{\delta\sigma}{dt} \right| dt \qquad \text{... Eq. (2-33)}$$

The load equivalence factors were determined by:

$$LEF = \left(\frac{D_a}{D_b} \right)^n = \left(\frac{L_b}{L_a} \right)^n \qquad \text{... Eq. (2-34)}$$

where D_a and D_b = damage (fatigue) transforms for events a and b, and

L_a and L_b = lives of the events a and b respectively

[2]SI-conversion of Equation 2-32 for P and P_s measured in metric tons (1000 kg) the coefficient -0,09 changes to -0,02.

The subsequent range in values for the damage coefficient (n) was calculated to vary between 3 and 7, based on the sensitivity of the damage transforms used in the analysis.

2.2.5 Discussion on the theoretical approach

Linear elastic multi-layer theory is the most widely used model for analysing flexible pavements. The pavement is modelled as a series of layers on an infinite subgrade or "rigid" base, each layer being of uniform thickness and of infinite extent in the horizontal plane. Each layer is composed of homogeneous, linear elastic, isotropic material and hence is fully characterised by two elastic constraints, conventionally Young's Modulus of elasticity (E) and Poisson's Ratio (υ). Deacon (1969) for example, based his calculation of load equivalence factors on the linear elastic theory and the assumptions pertinent thereto. Although his LEFs were basically relative comparisons, bituminous materials exhibit visco-elastic behaviour and consequently the calculated stresses would be highly dependent on factors such as temperature and load application rate. Several others (Tables 2-1 and 2-2) developed more sophisticated models from which materials exhibiting visco-elastic or stress-dependent behaviour can be characterised, but, due to the level of sophistication required in characterizing the various materials used in these models, widespread use of these models are restricted to specialised applications.

The reliability of predictions obtained from theoretical models is dependent on three sources of variation, namely; the inherently stochastic behaviour of materials under natural conditions, the inability of parameters in a model to fully represent all factors influencing pavement behaviour, and the measurement errors arising from differences between the observed and actual pavements behaviour. As the variations of behaviour within a normally homogeneous length of pavement alone may be as great as a factor of three to ten (Paterson, 1987), a concept of reliability or probability needs to be incorporated as an essential part of the modelling effort. To develop a theoretical evaluation procedure, the response variables must be related to future performance of the pavement, and the location in the pavement structure where this response will be critical must be known. The problem is that the failure mechanism in a given pavement structure may vary for different loadings. Thus the critical parameters do not remain constant for the same pavement structure with a change in axle load (Treybig, 1983).

In general, the laboratory techniques used to measure and define material strengths, do not give a satisfactory indication of the performance potential of a material as related to the performance of the material as a pavement layer (Ahlberg and Barenberg, 1963). The lack of applicability of tensile strain-damage relations to pavements with thin bituminous surfaces (< 75 mm) poses a problem as these type of pavements represent a vast majority of pavements in South Africa. Finally, very little corroborative evidence exist as to the predicative accuracy of the models using subgrade compressive strain as the damage parameter.

2.3 MECHANISTIC APPROACH

The mechanistic approach is similar to the theoretical approach, with the primary difference being that the response and distress parameters used are measured in situ and not calculated. The most accurate method of assessing pavement response, and hence damage caused by the passage of vehicle loads is by direct measurement, i.e. by allowing sufficient passages of a vehicle over a pavement structure to enable the damage to be measured (empirical approach). However, the impracticability of this is obvious especially with regard to time and cost. A solution is to compromise, viz, to measure pavement response and to model the behaviour. Thus, in the mechanistic approach, in contrast to the theoretical approach, the actual measured pavement response parameters are used in the modelling effort. Models are verified from the observed structural performance of the pavement and these models form the basis of developing LEFs.

2.3.1 Deflection based LEFs

2.3.1.1 Christison study (1986)

Christison (1986) used the measured vertical surface deflection to predict LEF, by developing LEF functions for both single and tandem axles i.e.:

Single axles:

$$LEF = \left(\frac{D_l}{D_b}\right)^c \qquad \text{... Eq. (2-35)}$$

Multiple axle configurations:

$$LEF = \left(\frac{D_l}{D_b}\right)^c + \sum_{i=1}^{n-1}\left(\frac{\Delta_i}{D_b}\right)^c \qquad \text{... Eq. (2-36)}$$

where: LEF = load equivalency factor

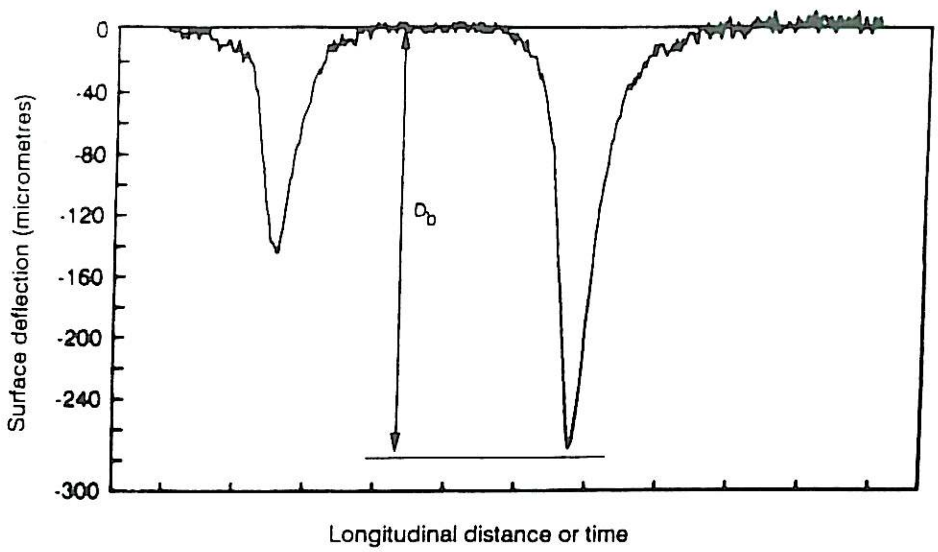
D_b = deflection under 18 kip (80 kN) single axle

D_l = deflection under various single axle loads, or deflection under leading axle in the case of tandem axles (Figure 2-1b)

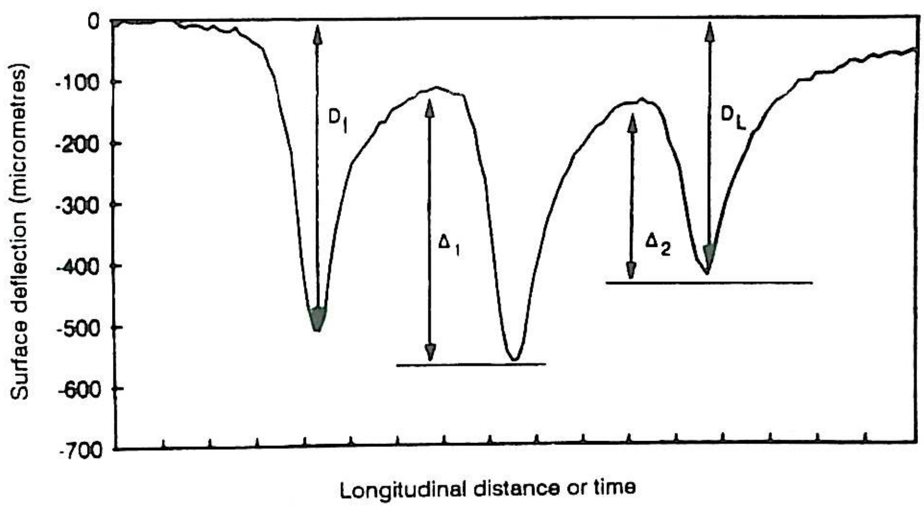
Δ_i = difference between maximum deflection under the second axle and the intermediate deflection between axles (Figure 2-1b)

c = slope of the deflection-anticipated traffic loading relationship. This was set equal to 3.8 following the recommendations by the Pavement Advisory Council of the Canroad Study (Christison, 1986).

The corresponding values of the calculated LEF were obtained from field measurements (surface deflection measurements) conducted on some fourteen pavement sections throughout Canada, known as the RTAC Vehicle Weight and Dimension Study (RTAC VW & D Study).



a) Deflection profile under single axle loads



b) Deflection profile under a tridem axle load

Figure 2-1: Typical deflection profiles under different axle configuration loads.

In a review of the data on multiple axles obtained during the RTAC VW & D Study, Prakash and Agarwal (1988) observed that the pavement behind the last axle in a axle-group nearly returned to its original position. Therefore they concluded that the damaging effect under the last axle would be determined by the total deflection under that axle rather than the relative deflection as used in the equation developed by Christison (1986). Subsequently a revised form of the equation by Prakash and Agarwal (1988), was proposed i.e.:

$$LEF = \left(\frac{D_l}{D_b}\right)^c + \sum_{i=1}^{n-2}\left(\frac{\Delta_i}{D_b}\right)^c + \left(\frac{D_L}{D_b}\right)^c \qquad \text{... Eq. (2-37)}$$

where;	D_b	=	maximum surface deflection under standard 18kip (80kN) axle load
	D_l	=	maximum surface deflection under leading axle (Figure 2-1b)
	D_L	=	maximum surface deflection under proceeding axles (Figure 2-1b)
	Δ_i	=	difference in magnitude of the maximum deflection recorded under axle (excluding leading axle) and the maximum residual deflection proceeding the axle as shown in Figure 2-1b
	c	=	slope of the deflection anticipated traffic loading relationship (3,8)

The proposed revision to the equation resulted in significantly higher values in the obtained LEFs for multiple axle configurations (Prakash and Agarwal, 1988).

Rilett and Hutchinson (1988) developed response type LEFs, based also on the fatigue analysis principles adopted by Christison (1986), for different axle group configurations. The form of the LEF function was estimated as:

$$LEF = CONSTANT \times LOAD^{l} \times TEMP^{t} \times SPEED^{s} \times AXLE\ SPACING^{a} \qquad \text{... Eq. (2-38)}$$

where:	CONSTANT	=	constant determined from a regression analysis
	LOAD	=	load on tandem axle group
	TEMP	=	average temperature recorded during test run (°C)
	SPEED	=	speed of test vehicle (km/h)
	AXLE SPACING	=	front to rear axle spacing in tandems and tridems
	l, t, s, a	=	coefficients determined by regression

LEFs were calculated from deflection ratios, i.e. the ratio of deflections measured under the test vehicles to those measured under the Benkelman beam vehicle. Variables such as speed, temperature and pavement type (as reflected by the pavement structural number) were kept constant, and the magnitude of the coefficients in the above equation were determined by regression. The aim of the study was to quantify the relative influence of load, axle spacing, vehicle speed and temperature on surface deflection based LEF functions. Later an additional parameter, namely structural number, was found to influence deflection based LEFs, and was therefore added as an additional input into the equation. However, insufficient data for the single and tridem axle configurations necessitated that data for the respective axle groups had to be pooled for the purposes of the analysis, some of which resulted in statistically questionable correlations (R^2 values of 0,43) for single axles, although good correlating values (R^2 of 0,74) were obtained for tridem axles.

2.3.1.2 Scala and Potter study (1981)

In a study conducted for the Australian Road Research Board, Scala and Potter (1981) proposed a method whereby load equivalence factors for specialised vehicles are predicted. The method is based on the assumption that equal response, i.e. maximum surface deflection, implies equal damage. Previous deflection studies conducted by Scala (1970) on typical pavements found in Australia concluded that the load on a single axle with single tyres which produces the same deflection, and subsequently the same damage, as a standard axle (80 kN dual-tyre single axle) is 53 kN. Apparently this assumption was also adopted by the study team of the National Association of Australian State Road Authorities Economics on Road Vehicle Limits (NAASRA ERVL) to assess the damaging effect of single axles with single tyres and was later incorporated into the NAASRA Interim Guide to Pavement Thickness Design (Scala and Potter, 1981).

The method further assumes that for any number of axles in a specific axle arrangement, i.e. tandem, tridem, or other, the total load on the axle group is equally distributed amongst the respective axles and subsequently, equally amongst all tyres on the axles. The individual axles are analyzed in isolation and the LEF for the combined axle configuration, is a summation of the LEFs calculated for the respective individual axles. Adoption of a relationship between tyre size, tyre load and pavement deflection permits the determination of the tyre load for any given tyre size, to produce the same pavement deflection as the standard axle. Knowledge of the transverse deflection profile for any given tyre size and load, in turn, enables the determination of the maximum deflection under this axle by means of the principle of superposition. Furthermore, adoption of a relation between axle load and maximum pavement deflection under the axle enables the determination of the axle load which will produce the same maximum pavement deflection as the standard axle. The corresponding LEF is then determined by the ratio of axle load to the load of the axle which produces the same damage as the standard axle, raised to the power of 4 (4^{th} power law).

2.3.2 LEFs based on strain measurements

Christison (1986) also used the measured interfacial horizontal tensile strain to predict LEFs. The strains were measured from embedded strain gauges positioned across the outer wheel path and placed at the asphalt-base layer interface. LEFs based on these measured interfacial strains were computed by using the following equation:

$$LEF = \sum_{i=1}^{n} \left(\frac{S_i}{S_h} \right)^c \qquad \text{... Eq. (2-39)}$$

where: S_h = maximum longitudinal interfacial strain measured under standard 8160 kg axle load (Figure 2-2)

S_i = maximum longitudinal interfacial strain measured under the ith axle in the axle configuration as shown in Figure 2-2

c = slope of the fatigue life-tensile strain relationship (c = 3,8)

The corresponding values of the calculated LEF were obtained from field measurements conducted on some fourteen pavement sections throughout Canada. In the approach adopted, the respective tensile strains measured under each axle were used in the equation as opposed to the intermediate values used in the deflection-based method.

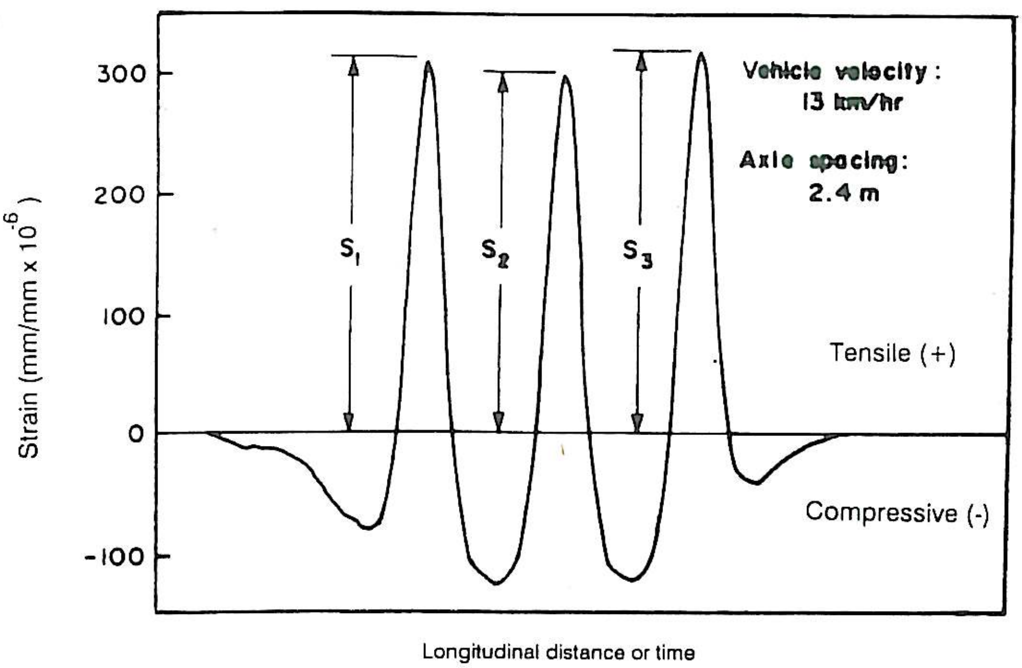


Figure 2-2: Longitudinal interfacial strain profile measured under a tridem axle configuration load (updated from Prakash and Agarwal, 1988).

2.3.3 Discussion of the mechanistic approach

The mechanistic approach differs from the theoretical approach in that the parameters used in the calculations are obtained from actual measurements and not determined from laboratory testing of specimens. The validity of the results will therefore depend on the correctness of the assumptions contained in the theoretical procedure on which it is based, the parameter(s) used to define pavement distress, the variation in the range of data used in the analysis (pavement types, load configurations, etc.) and the method and accuracy of the measuring technique.

Most of the methods presented and discussed previously use surface deflection as the response parameter. The primary reason being that deflection is a direct, easy, accurate and widely used measurable parameter. Furthermore, adoption of deflection as the response parameter obviates the use of elastic-layer theory (or any other) to obtain LEFs.

Deflection-based methods are all based on the hypothesis that "equivalent deflection response implies equivalent damage". Although this hypothesis, in general, applies to all response-based methods, deflection is a response which does not assign specific damage criteria. Therefore, any damage model can be used in conjunction with deflection-based methods, be it cracking, deformation or any other. Furthermore, the possibilities of wheel and axle configurations that can be evaluated with deflection-based methods are unlimited, therefore deflection based LEFs are regarded as a good starting point to improve relative damage models.

The basic assumptions pertaining to the calculation methods used in the models should be carefully considered as vast differences in obtained LEFs may occur. Christison (1986) used the difference of the maximum deflection under the axle and the residual deflection of the preceding axle to compute the LEF. Thus the relative effect of each axle in a group of axles was calculated, and the LEF for the total axle configuration was the summation of the effects of the individual axles of the group. The proposed revision of the method (Prakash and Agarwal, 1988) is a more conservative approach to calculate LEFs since the inclusion of the total deflection measured under the last axle, in fact already contains and reflects the effect of any preceding axle(s) in an axle arrangement. Therefore double calculation of the effects occurs to some extent when using this revised method, and the subsequent result is higher calculated LEF values.

2.4 THE EMPIRICAL-MECHANISTIC METHOD USED IN SOUTH AFRICA

The method currently used in South Africa to calculate LEFs is a load based power law relationship, which was derived at the AASHO Road Test (Hveem and Sherman, 1963; Irick and Hudson, 1964) but differs in the definition of damage used, namely deformation as opposed to serviceability (Walker et al, 1977) i.e.:

$$LEF = \left(\frac{P}{80}\right)^{d} \qquad \text{... Eq. (2-40)}$$

where: P = axle load in kN

80 = standard single axle load in kN

d = relative damage coefficient (found to vary according to pavement type and state)

The customary parameter used to relate equivalent damage of various single axle loads, is permanent surface deformation (rutting). A terminal rut depth of 20 mm (severe condition), as measured under a 2 m straight edge, is used as the basis for comparing the effects of different loads in relation to that of a standard 80 kN axle load. This rutting limit is strictly applicable to conventional flexible pavements, for which deformation is a major form of distress, and represents a level at which wheel path pounding of water may become a hazard to road users. A warning limit for rut depth of 10 mm have been suggested for rehabilitation investigations (CSRA, 1991).

From Heavy Vehicle Simulator (HVS) tests conducted on existing pavements throughout the country and on a variety of pavement types, it was possible to compare the effect of different single axle loads. These HVS tests further permitted the calculation of the damage coefficient (d) used in the LEF formula, given above. In general, the premise is that the number of applications (N_2) of any given wheel load (P_2) to a terminal deformation level of 20 mm can be related to the number of applications (N_1) of the standard or base load (P_1) to the same terminal deformation level, or:

$$N_1 = LEF \times N_2 \qquad \text{... Eq. (2-41)}$$

The LEF can therefore be expressed as the ratio of the number of applications (N_1) of a standard wheel load (P_1) to an end deformation of 20 mm, to the number of applications (N) of any other wheel load (P_2) to an end deformation of 20 mm, or:

$$LEF = \frac{N_1}{N_2} \qquad \text{... Eq. (2-42)}$$

By plotting the deformation or rut (D) against the number of applications (N) of a wheel load (P) (Figure 2-3), the value of d pertaining to the specific test can be calculated. A linear relation can be assumed based on past HVS test experience (Kekwick, 1985) and the respective slopes of the curves represent the rate of deformation (R_d), viz:

$$N_1 \times R_{d_1} = D_1 \qquad \text{... Eq. (2-43)}$$

and,

$$N_2 \times R_{d_2} = D_2 \qquad \text{... Eq. (2-44)}$$

Therefore for the two wheel loads to be equivalent in damage, they must cause the same amount of damage, i.e. $D_1 = D_2$; at the same terminal level in deformation (20 mm), therefore:

$$\frac{N_1}{N_2} = \frac{R_{d_2}}{R_{d_1}} \qquad \text{... Eq. (2-45)}$$

and from Equation 2-41, it follows that:

$$LEF = \frac{R_{d_2}}{R_{d_1}} \qquad \text{... Eq. (2-46)}$$

The generalised AASHO load damage formula (using the same notation) is:

$$LEF = \left(\frac{P_2}{P_1}\right)^d \qquad \text{... Eq. (2-47)}$$

Thus:

$$\left(\frac{P_2}{P_1}\right)^d = \frac{R_{d_2}}{R_{d_1}} \qquad \text{... Eq. (2-48)}$$

When transformed to a logarithmic function and with some mathematical manipulation, the relative damage coefficient (d) can be determined by:

$$d = \frac{\log\left(\frac{R_{d_2}}{R_{d_1}}\right)}{\log\left(\frac{P_2}{P_1}\right)} \qquad \text{... Eq. (2-49)}$$

Thus, damage coefficients (d) for different pavements and pavement types were determined and were found to be dependent on both the type and state of the pavement.

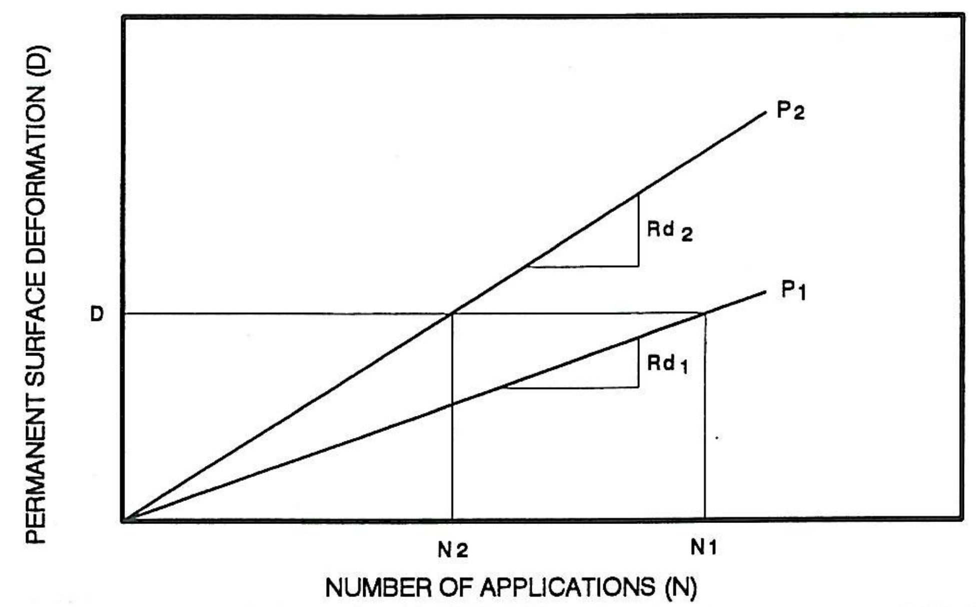


Figure 2-3: Rate of deformation (Rd) used in calculating the relative damage coefficient (d).

2.4.1 Discussion on the South African HVS based damage model

The Heavy Vehicle Simulator (HVS) fleet consists on three test machines operating in various regions in South Africa on test sections which are part of existing pavements. The HVS is an accelerated testing facility which can apply single or dual wheels (half axle loads up to 200 kN). Relative damage of all axle loads are expressed in terms of a standard axle load of 80 kN. This standard axle load also represents the legal axle load limit currently employed in the RSA.

Viljoen (1984) proposed the notation of *d* (referred to as the relative damage coefficient) as opposed to n (AASHO damage coefficient) to distinguish between the difference in approach and criteria used to developed the AASHO and the HVS coefficients. From HVS test results it was possible to determine the value of the damage coefficient (d) and was found to vary between < 1 and 8,5 for deformation, and up to 13 for cracking, according with various factors such as pavement type and pavement state (Freeme, 1983; Maree, 1982; Kleyn and Savage, 1982). Results from the recently completed Force project (OECD, 1991) appear to support this found variation in LEFs. The corresponding ranges in damage coefficients (d) found for rutting were 1,47 to 5,74 and for cracking values ranged between 1,80 and 6,68.

Although damage coefficients were determined based on rutting, damage coefficients based on cracking were also determined in a limited number of pavements (van Zyl and Freeme, 1984). Table 2-3 gives a summary of such coefficients determined for various pavement types. Kekwick (1984b) applied the same approach to derive d-values for concrete pavements by applying cracking criteria as the parameter to evaluate pavement life as opposed to deformation as used for flexible pavements. These findings are however based on the results of only a limited number of test sections. Bosman and Viljoen (1986) state that any pavement performance parameter could be used in similar fashion to derive equivalency factors. They based their approach on the mechanistic design method used in South Africa and defined failure as that point in the life of a pavement when it reaches the terminal service level set for the specific pavement category (Road category A to C as defined in TRH 4 (CSRA, 1989)) and therefore major rehabilitation is required. This terminal service level can be ascertained from a variety of parameters, be it excessively high deflections, a severe degree of cracking, inadequate riding quality, deformation, surface disintegration, such as ravelling, potholing, bleeding etc.

Kleyn et al (1985) found the rate of deformation (R_d) of a pavement in terms of the number of application of a particular wheel load (expressed in millimetres per million load applications - mm/10^6) affords a very useful mean of characterizing the pavement. In essence, the value of d represents the load sensitivity of a pavement in a specific state which in turn is indicative of the strength balance of the pavement, i.e. the shallower the strength distribution in a pavement the more sensitive it can be to overloading.

Shackleton (1990) used several HVS test results conducted since 1976 on granular base and subbase pavements to evaluate the validity of the load based "power law" in the calculation of relative damage on these pavements. Although it was not the objective of his study, he concluded that in general the power law seems to hold true for pavements with granular bases and subbases with d varying between 2 and 3.

Uncertainty still exists regarding the value of the damage coefficient (d) to be used. Originally a value of 4,2 was assumed (probably originated from the value obtained and used by the California Division of Highways) later, a value of 4 was used (which stems from the AASHO Road Test).

Table 2-3: Summary of relative damage coefficients derived from HVS tests

BASE/SUBBASE TYPE	ROAD CATEGORY	PAVEMENT STATE	LAYER TYPE AND STATE	DAMAGE CRITERIA	LOADS COMPARED	d
Crushed stone/ cemented	A	Dry		Deformation	40†/55† 65†/75†	4.5
Crushed stone/ cemented	A	Dry		Deformation	40†/55† 65†/75†	6.0 6.7
Gravel/ gravel	C	Dry		Deformation	40/60 40/80 40/100 80/100 60/100	< 1 3.3 4.6 to 6.8 8.5 4.1
Gravel/ gravel	C	Wet		Deformation	40/60 40/80 40/100 60/100 80/100	< 1 < 1 < 1 1.9 1.4
Gravel/ gravel	C	Dry	Surfacing	Visual cracking	40/60 40/80 40/100 60/100 80/100	< 1 1.7 2.5 to 2.8 6.3 4.9
Crushed stone/ cemented	A		 Base dry Base dry and fatigued Base wet and fatigued	Deformation Mechanistic analysis Mechanistic analysis Mechanistic analysis	70/100 40/70 40/70 40/70 40/100	4.7 4.8 2.4 0 0
Crushed stone/ cemented	B	Wet		Deformation	70/100	2.3
Cemented/ crushed stone	A	Dry	Base un-cracked	Cracking Mech analys	40†/60† 40†/80†	4.5 to 10 6 to 13
Cemented/ cemented	B	Dry, cracked; wet		Deformation deformation	40/100 40/100	1.0 3.0
Cemented/ gravel	C	Cracked, dry		Deformation	40/70 40/100	3.3 3.0
Cemented/ cemented	A	Cracked - uncracked, dry		Deformation	40/60	2.0
Recycled asphalt/ cemented Recycled asphalt/ cemented	A A	Base brittle, subbase varies between cemented and granular.	Base uncracked Base uncracked	Crack initiation - Mech analys Visual crack-ing	40/70 100 40/70 70/100 70/100	1.5 to 4.7 2.5 1.8 0.8
Asphalt/cemented	A	New structure		Deformation	80/100	1.4

† Trafficked with a single wheel (after Van Zyl and Freeme, 1984)

Damage coefficients developed from Heavy Vehicle Simulator (HVS) tests results (Table 2-3) show a fairly wide range of calculated values, even for similar pavement structures.

The method used in South Africa, although based on a range of different wheel loads and a variety of different pavement types, may not reflect the failure mechanism encountered on all pavements and pavement types. For further concern is the method used in the determination of LEFs, i.e. the assumption of the "power law"; and the method of applying the load, i.e. uni-directional creep speed single or dual half axle load, may not reflect actual traffic conditions. Furthermore, the HVS-based damage model only applies to single axle loading conditions. This is a major limitation, as mixed traffic comprises vehicles encompassing a wide spectrum of wheel and axle configurations. Thus, although the HVS-based model may be regarded as a suitable damage model for most pavements encountered in South Africa, it does not facilitate, per se, the calculation of LEFs for wheel and axle configurations other than the trafficking wheel of the HVS, viz single axle, with single or dual wheels.

2.5 COMMENTS ON THE LITERATURE REVIEW

Several methods which attempt to quantify load-associated damage on pavements were evaluated and discussed in the preceding paragraphs. As these methods were developed for different regions and used different parameters to quantify damage, they all have their own inherent assumptions and limitations and subsequently, provide different calculated LEFs. The results and findings of the AASHO Road Test plays a prominent role in most of the discussed methods and of specific importance, is the relevancy of these results to circumstances and conditions pertaining to South Africa. Factors to take cognizance of when evaluating the suitability and applicability of the AASHO Road Test results for use in South Africa were also presented and discussed. A summary of the discussed models, together with a reference to their base of derivation, the parameters used and the general limitations regarding applicability of the methods is presented in Table 2-4.

The damage model currently used in South Africa is based on extensive research conducted on actual pavements constructed over the years (approximately 15 years) since the development of the first prototype HVS.

Table 2-4: Summary of methods discussed in previous paragraphs with reference to the general applicability of these methods.

Method	Base of derivation	Principle source of information	Damage criteria	Parameter(s)	Inputs	Applicability
1) Theoretical	Computer based analysis based on linear elastic theory	AASHO Road Test data; laboratory testing of selected pavement specimens; traffic studies conducted in Kentucky (USA)	Fatigue and deformation	Calculated strains, stresses and deflections at various pre-selected locations within the pavement.	Depends on the model used. Main variables are Modulus of elasticity (E); Poisson's ratio (υ); load; axle configuration and pavement composition	Predictive accuracy of models depend on modelling technique used and the assumptions pertinent thereto, i.e. linear or non-linear material characterization. Applicable to all pavement types and any region for which required input parameters are known, or for which they can be determined.
2) Empirical	Empirical relationships established from field observations	Measurements conducted on AASHO Road Test experimental sections and experience gained from other field observations	Serviceability	Slope variance as measured by the CHLOE profilometer indicates roughness or riding quality	Wheel load and number of axles; pavement structural number; terminal serviceability level	Applicable to regions where prevailing conditions and material properties are similar to those for which models were developed. Simplification of original relationship, i.e. power law relationships, limited to specific pavement types (SN = 4), wheel and axle configurations.
3) Mechanistic	Theoretical models calibrated and verified by field observations	Canroad studies, RTAC VW & D Study and measurements conducted on other selected pavement sections.	Fatigue Cracking	Surface deflection; interfacial strain	Measured surface deflection or measured horizontal strains at selected layer interfaces	Depend on both the accuracy of the model used and the accuracy of the measurements used during the modelling effort. Models exist from which LEFs for any pavement type, wheel and axle configuration can be calculated.
4) HVS	Empirical relationship adopted from AASHO Road Test and verified by HVS testing.	Extensive HVS based research on actual in use pavements.	Deformation; fatigue Cracking	Deformation (rutting); crack propagation	Pavement type and state, wheel load	All pavements types - distinction by means of different damage coefficients for different pavement types. Limited to single axle dual tyre loads and region for which it was developed.

Although the common shortcomings of other damage models used elsewhere and discussed previously were overcome, the HVS-based damage model only applies to single axle loading conditions. This is a major limitation, as traffic comprises vehicles encompassing a wide spectrum of wheel and axle configurations. Thus, although the HVS-model may be regarded as a suitable damage model for most pavements encountered in South Africa, it does not facilitate the calculation of LEFs for wheel and axle configuration loads other than that used by the trafficking wheel of the HVS, viz single axle, single or dual wheeled. A suitable method is therefore needed to calculate LEFs for all axle configurations commonly encountered on South African road networks. However, a prerequisite for the general acceptance of a method for use in South Africa is the development of a model to suit circumstances and conditions generally encountered in the country.

The slow speed of the HVS wheel/s (1 to 8 km/h) is another major limitation of HVS-based damage models for the determination of LEFs. This dynamic effect has not been quantified for all road building materials used in South Africa, and therefore a method for the determination of LEFs at various speeds cannot be implemented at this stage. Research is currently being carried out to apply dynamic analysis in pavement design (SARB, 1992).

2.6 CONSIDERATIONS ON LOAD-DAMAGE RELATIONSHIP

Load equivalency factors are significantly influenced by the specific pavement response parameters on which they are based (deflection, strain, stress, riding quality, deformation, etc.), and the methods used for their calculation. From the methods to determine load associated pavement damage discussed above, it is evident that the definition of damage is crucial to the results and findings of all the studies performed to quantify this concept.

Paterson (1987) states that as the definition of damage could be related to the rehabilitation or maintenance required to rectify a specific time related distress, such as cracking, deformation or surface deterioration, the definition of damage may become more of an economic than an engineering related consideration. It is recognised for example that the different maintenance and rehabilitation alternatives, and thus different cost responsibilities, are necessitated by different types of distress. For instance, the need for resealing is related to the amount of cracking, ravelling or potholes, the need for a smoothing overlay to the roughness, and the need for rehabilitation is related to various combinations of all distress types.

Damage does not necessarily result only from load applications, but damage could in some instances be attributed primarily to the effect of environmental influences. Therefore, environmental and traffic effects should be evaluated jointly, and the level of distress resulting from either one or the combined effect of both, should be reflected in the parameter(s) used to construct such a prediction model.

The criteria used to evaluate distress, such as fatigue cracking, deformation, riding quality, etc., change during pavement life and therefore, a single function with a single parameter may not be applicable at all times. Prediction of the future performance of a pavement is at best only an approximation and is subject to considerable variation owing to changes in traffic loading conditions, changes in the state of the pavement materials and environmental influences. Hveem and Sherman (1963) identified some 30 variables which could affect the performance of asphalt pavements (shown in Figure 2-4). Because of the complexity of the problem, a simple relation which can accurately predict future performance and hence, accurately predict the parameter required to quantify damage due to wheel loads, seems unlikely. It appears more feasible that a complex model, which contains axle loads and axle spacings together with other explanatory variables, would be required.

A list of variables affecting load equivalency factors is given in Table 2-5 (Deacon, 1969) and mainly consists of load, pavement and failure variables. Many others variables have been identified since then such as environmental (temperature, moisture), load (speed, dynamics effects), and pavements (pavement state) variables. Even though this is not an exhaustive list of variables affecting LEFs, it does indicate the number of different aspects to be taken into account when referring to "equivalent loading" conditions, i.e. it is too simplistic to define LEFs based upon a simple load ratio.

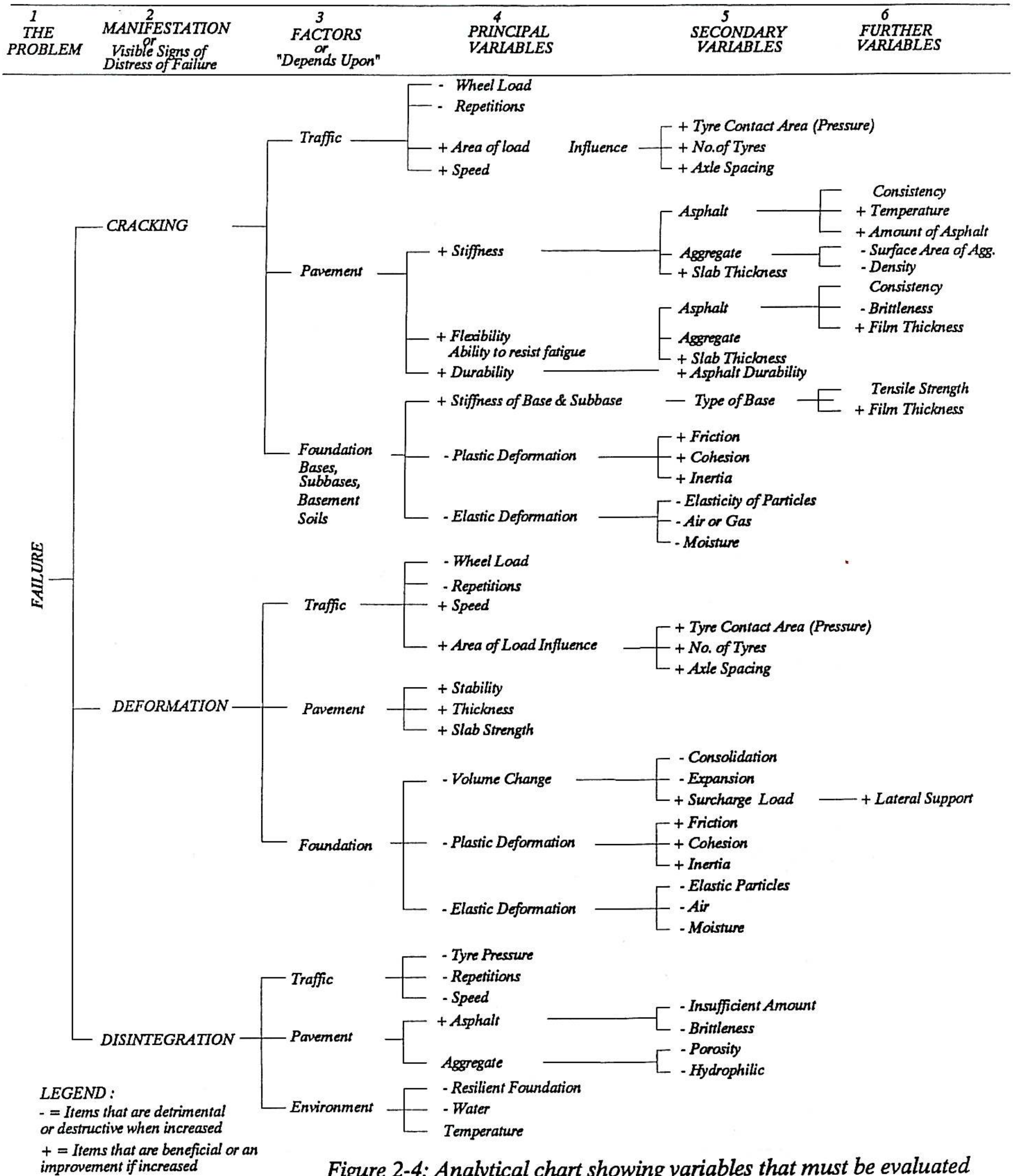


Figure 2-4: Analytical chart showing variables that must be evaluated for structural design of asphalt pavements (Hveem and Sherman, 1963)

Table 2-5: Variables affecting load equivalence factors (Deacon, 1969).

1. Load variables	1.1 Magnitude	
	1.2 Contact pressure	
	1.3 Axle configuration	a. Single tyres on single axles b. Dual tyres on single axles c. Dual tyres on tandem axles d. Super single tyres e. Dual tyres on tridem axles
	1.4 Axle and tyre spacing	a. Spacing between dual tyres b. Spacing within tandem axles c. Spacing within tridem axles
2. Pavement variables	2.1 Surface	a. Mechanical response b. Failure response c. Thickness
	2.2 Base	a. Mechanical response b. Failure response c. Thickness
	2.3 Subgrade	a. Mechanical response b. Failure response
	2.4 Composite mechanical response	
3. Failure variables	3.1 Mode of distress	
	3.2 Definition of distress	
	3.3 Selection of damage parameter	

Information is available from which several parameters can directly be related to specific failure mechanisms, viz rutting is generally governed by the vertical compressive strain at the top of the subgrade (Treybig, 1983; Hajek and Agarwal, 1989; Claessen et al, 1977; Brown et al, 1977) or as in some instances, on top of the crushed aggregate base (Bonaquist et al, 1989) and lightly cemented layers (De Beer, 1989b). Tensile strain at the bottom of asphalt layers can be directly related to fatigue cracking (Deacon, 1969; Kennis, 1977; Sebaaly et al, 1989; Hajek and Agarwal, 1989; Treybig, 1983). Thus, identifying the primary mode of failure and the parameter(s) which best describes that mechanism of failure, is therefore essential to quantify damage.

The above considerations lead to the inapplicability of LEF determination methods based solely on load-damage considerations, mainly when some kind of extrapolation needs to be done in order to accommodate different loading, pavement or environmental conditions. Therefore a methodology that is based on pavement response rather than load consideration is suggested. At this stage the suggested method is believed to be able to overcome the shortcomings associated with load-damage based methods and will implicitly incorporate local condition as it is based on pavement response.

A provisional suggestion is to use the deflection-based method proposed by Christison (1986). The analysis of the output will serve as verification of the applicability of deflection-based methods to South African conditions and experience. The possible advantages and disadvantages of the method are discussed and are used as the basis for the formulation of a more comprehensive method for the determination of Load Equivalency Factors (LEFs). The latter method is based on pavement response parameters and relates them to load characteristics, pavement properties and environmental conditions.

3. APPLICABILITY OF DEFLECTION-BASED METHODS ON THE DETERMINATION OF LOAD EQUIVALENCY FACTOR (LEFs)

3.1. SUGGESTED METHOD TO CALCULATE LEFs

In the preceding section, amongst others, the shortcomings of the HVS-based method used in South Africa were highlighted. In this chapter, a deflection-based method is used in order to assess these shortcomings, and the applicability of the method to South African conditions is investigated. This method permits the calculation of LEFs for multiple wheel and axle configuration loads based on the measured surface deflection response of the pavement. This approach was developed by Christison (1986) for the Canadian Roads Department. The fundamental principle on which the method is based, is that equivalent response implies equivalent damage (Scala and Potter, 1981), viz equal deflection implies equal damage. The basic equation to derive LEFs is:

$$LEF = \left(\frac{D_l}{D_b}\right)^c + \sum_{i=1}^{n-1}\left(\frac{\Delta_i}{D_b}\right)^c \qquad \text{... Eq. (3-1)}$$

where : LEF = load equivalency factor

D_b = deflection under the standard reference axle (80 kN dual-tyre single axle) (Figure 2-1a)

D_l = deflection under a single axle load, or deflection under leading axle in the case of multiple axle arrangements (Figure 2-1b)

Δ_i = difference between maximum deflection under each succeeding axle and the intermediate deflection of the preceding axle (Figure 2-1b)

c = slope of the deflection anticipated traffic relationship (set equal to 3,8 for this study)

As no information is available regarding the value of c, a value of c = 3,8 was adopted for the purposes of this study. This follows recommendations (Christison, 1986) made by the Pavement

Advisory Council of the Canroad Study. Evaluation of HVS test data might provide sufficient information from which a more appropriate value of c can be derived for local conditions.

Surface deflection is not the only response parameter which could be used. The basis for determining response based LEFs may be one of several such as equal maximum principal tensile strains at various locations in a pavement (usually at the bottom of surfacing or bound layers), equal vertical stresses (usually on top of base or subgrade), or equal vertical deflections (Deacon, 1969, Boussinesq, 1885 and van Vuuren, 1972). The use of the latter parameter, i.e. surface deflections, is however preferred to other response parameters such as stress and strain, for the simple reason that surface deflection is model independent, i.e. independent of the model used to characterise the materials, viz, linear elastic, stress dependent or any other material specific model. Owing to the state of the art in technological developments deflection is a directly, easily, accurately and widely used measurable parameter. Furthermore, it also accounts for a number of factors which influence pavement behaviour characteristics, such as load variables and pavement variables.

Deflection, as a parameter, does not directly measure damage, as the case may be if stresses or strains were used. However, the measured deflection can be related to damage in a number of ways. As a design parameter, considerable experience has been gained with surface deflections especially on pavements with asphalt surfaces and granular bases. Several pavement design methods, such as the TRRL surface deflection method (Kennedy and Lister, 1978) and the Asphalt Institute design method (The Asphalt Institute, 1969) use surface deflections as the primary response parameter to evaluate the structural condition of a pavement. One of the prime objectives of the recently completed FORCE project (OECD, 1989) was to insure accurate and reliable measurements of surface deflections, as this was regarded as an important and widely used indicator of pavement condition.

Nijboer and van der Poel (1953) identified load-deflection interaction to be a valuable relationship to express pavement response and subsequently included both these parameters in a term they defined as "stiffness" (resistance to bending) i.e.:

$$S = \frac{F_P}{X_P} \qquad \text{... Eq. (3-2)}$$

where : F_P = force acting on a pavement in Newton and,

X_P = deflection of the pavement in micron

The fundamental principle on which the proposed deflection-based method is based is the hypothesis that equivalent response implies equivalent damage, i.e. that equal deflections implies equal damage. Consideration of this hypothesis therefore needs to be evaluated.

From the correlations obtained from the WASHO Road Test and similar findings of others, Benkelman and Carey (1962) used the basic premise that deflection (surface deflection as measured by the Benkelman beam - Benkelman and others, 1962) of a given pavement under a particular load would serve as a better measure of the pavements ability to withstand repetitive load application, than knowledge of its structure alone, viz that the magnitude of the defection would reflect the strength of the pavement system as actually constructed, regardless of the construction as specified.

Subsequently they constructed a mathematical model from which the life of a pavement to a given level of serviceability could be estimated satisfactorily using both load and deflection as input parameters i.e.:

$$W_P = \frac{A_0 \; L_1^{A_1}}{d^{A_2}} \qquad \text{... Eq. (3-3)}$$

where : W_P = applications of axle load L_1 to serviceability level p

L_1 = single axle load in kips

d = deflection under wheel load, and

A_0; A_1; A_2 = constants obtained by regression

The relationship established by Benkelman and Carey (1962) served as basis for the load-deflection relationships developed during the AASHO Road Test (HRB, 1962a), which was further modified to accommodate tandem axle groups.

In the literature several references to deflection-damage relations are made (Lister and Kennedy, 1977; Finn and others; Jung and Phang, 1974). Several references to deflection-based methods to derive LEFs are also found in the literature (Scala and Potter, 1981; Christison, 1986; Prakash and Agarwal, 1988; Rillet and Hutchinson, 1988) and LEF equations based entirely on surface deflections were developed to express the damaging effects of different wheel and axle load configurations on pavement structures.

In South Africa, deformation is generally used as the damage criteria, i.e. terminal surface rut level of 20 mm as measured with a 2 m straight edge. Sebaaly and Tabatabaee (1989) found significant correlations between surface deflection and the rate of rutting during their investigation into the effect of tyre pressure and tyre type on the response of flexible pavements. From a selection of several HVS test sections (based on findings of Maree (1982) and which were further updated for this study) the initial measured deflection was plotted against the number of load applications (standard axle loads) to failure. The results are shown in Figure 3-1. The apparently poor (statistical) relationship (R^2 = 0,51) can be attributed to the limited range of deflection. The relationship between initial deflection and pavement life (using rut depth as the failure criterion) is based on the high sensitivity of deflection measurements to changes in the subgrade materials. A relatively weak subgrade would be indicated by high deflections which would, with an increase on loading, result in high rut depths. Weak base or subbase material would only result in a relatively small increase in deflection which would, due to natural variation in pavement materials, result in a poor correlation between deflection and pavement life (Jordaan, 1988).

3.2. FACTORS AFFECTING PAVEMENT RESPONSE AND PERFORMANCE AND THEIR INFLUENCE ON SURFACE DEFLECTION

From studies conducted both locally and internationally, several factors which influence surface deflection, and its ability to predict pavement response, were identified. The great advantage in using surface deflection, lies in its ability to reflect such a wide range of factors by a single, easily measurable, and widely understood parameter. Several studies, such as: the AASHO Road Test (HRB, 1962c), the Brazil-UNDP Study (GEIPOT, 1982), the South African HVS testing programme (since 1974 to 1991), the recently completed FORCE project (OECD, 1991), to name but a few, provide valuable data to upgrade, develop and extrapolate current pavement design and damage models.

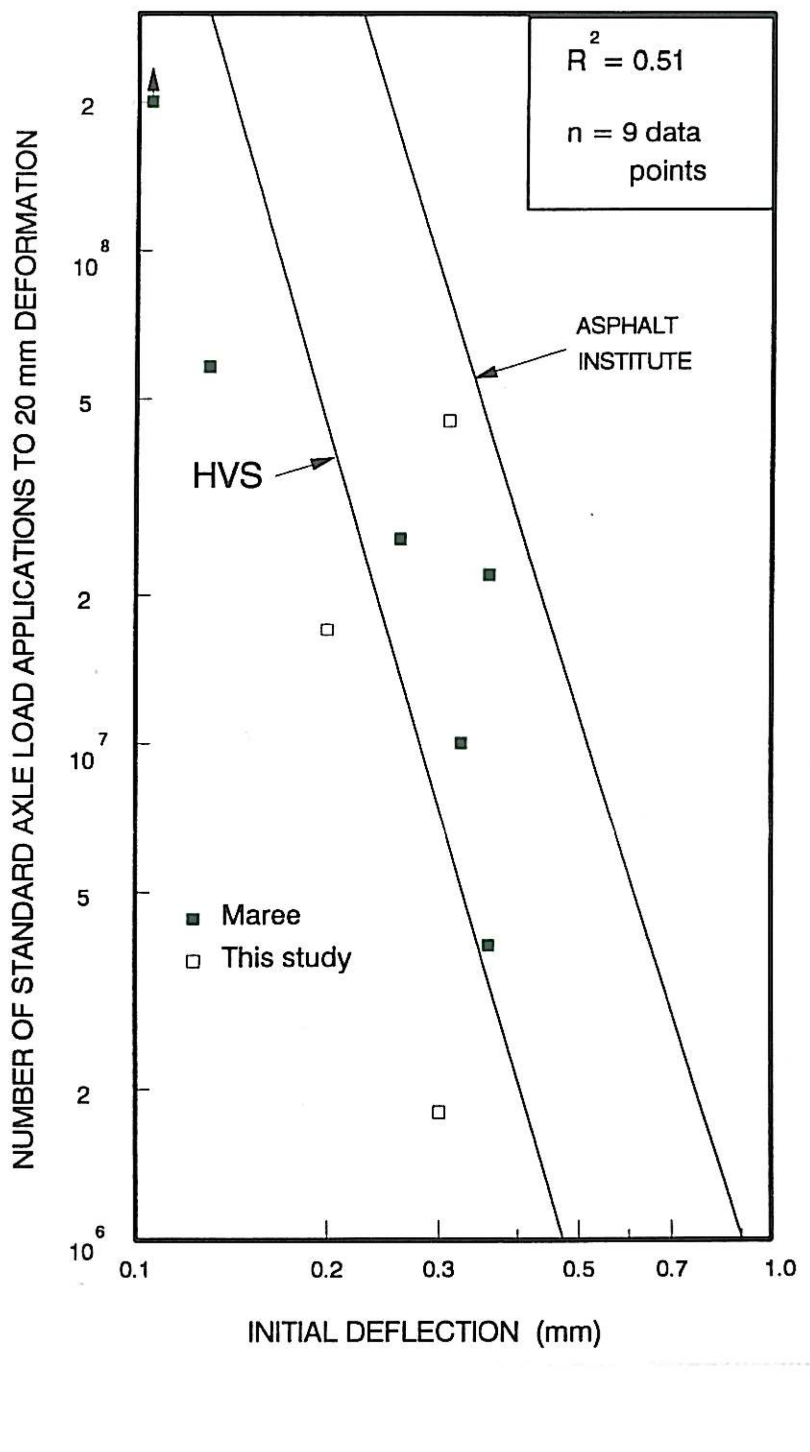


Figure 3-1: Plot of Initial Surface Deflection measurements vs. Number of Load Applications to Failure (20 mm permanent deformation), based on selected HVS test results (updated from Maree, 1982).

The quantitative effect of any of these variables on pavement performance has not been clearly established. This is mainly due to the difficulty in isolating the effect of any given variable under the fluctuating conditions to which a highway pavement is subjected. There is a danger of misinterpreting the results if one of a number of interrelated variables is isolated and studied independently of the others (Ahlberg and Barenberg, 1963). However, the variables that affect pavement performance must be isolated and analyzed independently if the effects of each are to be put into proper perspective. Once this has been done they can be integrated into pavement design procedures.

Some of the factors which influence pavement behaviour and are reflected in deflection measurements, include:

(a) ***Pavement type:*** An increase in the pavement strength, as reflected by the structural number of the pavement, has the effect of reducing surface deflection (Rilett and Hutchinson, 1988). The complete deflection basin would most probably be a more accurate measure of the structural strength of a pavement, as it shows the relative sharpness of bend (curvature). From modelled studies (Elliot and Lourdesnathan, 1989), using the principles of stress dependency of unbound granular layers ($k\ \theta^n$ theory), an increase in deflection with decrease in pavement strength was observed. Furthermore, an increase in the strength of a pavement generally causes an increase in the stiffness of a pavement. As the stiffness can be directly related to deflection (Nijboer and van der Poel, 1953), it follows that deflection increases as pavement stiffness decreases, or subsequently, as pavement strength decreases. It is however important to note that the stiffness of a pavement is highly dependent on the state and type of the material used in the pavement layers; pavement type, material type and condition, are strongly interrelated.

(b) ***Material type, condition and load history***: the type of material used, i.e. asphaltic, granular or stabilized, plays a major role in terms of classifying the different pavement types, for example : deep granular pavements, pavements with stabilized bases and/or stabilized subbases, etc. The "relative stiffness" of pavement layers are primarily influenced by the type of material used in the various pavement layers, and the manner of composition of the different pavement layers within the pavement structure. In general pavements with stabilized base and subbase layers tend to exhibit lower resilient deflections as opposed to natural gravel base pavements, but this generalised statement, as mentioned previously, is highly dependent on the state, i.e. pre- or post cracked phase of the stabilised layer. This phase of a stabilised layer is again dependent on the history of load applications, i.e. a newly constructed or existing pavement. Layer thickness and density are two construction variables that have a significant impact on flexible pavement performance. Both these variables have a direct bearing on the relative stiffness of a pavement layer, which in turn is a function of the magnitude of the load and the deflection measured in the pavement layer (Nijboer and van der Poel, 1953). Grivas (1985) stated that each material has a distinct stress-strain, strength, and fatigue behaviour under repetitive loading. The damping characteristics of the materials have a deciding influence on the deflection under a moving load. The damping ratio of a material is stress dependent and therefore also changes as the speed of the vehicle changes (SARB, 1992a, 1992b, 1995).

(c) ***Moisture condition***: The degree of saturation of pavement layers could have a marked influence on deflection, as evident from the AASHO findings. Effects of freezing and thawing contribute to the in situ moisture content, which proved to be the critical periods in the level of deflections during the execution of the AASHO Road Test (HRB, 1962a). Moisture condition is therefore dependent on the geographical region and could be adversely affected by seasonal fluctuations. Granular base and subbase pavement deflections tend to be more sensitive to in situ moisture conditions than stabilized or asphaltic materials (HRB, 1962a; Freeme and de Beer, 1987). The factors which cause a change in pavement state are either load-associated or water-associated or both, and provided the materials remain in a dry state, peak loading is normally not a problem (Freeme and Servas, 1985). This is also reflected in lower values in the HVS based relative damage coefficients.

(d) ***Temperature***: Since bituminous materials are visco-elastic, deflections are highly dependant on the temperature and rate of loading (Freeme and de Beer, 1987). Deflections in these materials increase with an increase in pavement temperature, but within a range in temperature of between 80 and 120 °F, deflections were observed to remain constant (HRB, 1962a). Granular and stabilized materials exhibit stress dependent and/or elastic behaviour (Thompson, 1974; de Beer, 1989a) and the behaviour of these materials under loading conditions are mainly influenced by the state and condition of the materials. Therefore temperature has a minimal influence on the overall response of these type of materials.

(e) ***Load magnitude***: Deflections increase as the magnitude of the applied load increases (Sebaaly and Tabatabaee, 1989). Good correlations between load-deflection relationships were reported by several authors (HRB, 1962a; Sebaaly and Tabatabaee, 1989; and Queiroz and others, 1991). However, the magnitude of the applied load and the repetitive application of this, or any other load of lesser magnitude, are equally important when assessing the damage to pavements due to load applications (Deacon, 1963; Govind and Walton, 1989). Although the relationship between deflection and number of load applications are governed by several factors, deflections generally increase with an increase in number of load applications.

(f) ***Tyre pressure and tyre type:*** Studies done by Bonaquist and others (1989) and Sebaaly and Tabatabaee (1989) showed that an increase in tyre pressure had a minimal effect on the measured response of a pavement (range in tyre pressures evaluated varied between approximately 75 psi (525 kPa) to 145 psi (1000 kPa). A hypothetical analysis by Scala and Potter (1981) indicated a 25 percent increase in the deflection with a concomitant increase in tyre pressure from 200 kPa to 1000 kPa. From special studies conducted during the AASHO Road Test (HRB, 1962b) it was concluded that tyre pressures had no or minimal effect on pavement deflections. While Bonaquist and others (1989) used only two different tyre types and found no consistent trend in the influence of tyre type on the magnitude of deflection, Sebaaly and Tabatabaee (1989) found the deflections under bias tires were 20 and 15 per cent higher than those under wide-base radial single tires respectively.

(g) ***Contact area and load intensity***: These two parameters are both directly related to the tyre pressure and tyre type. For a specific type of tyre, changes in the tyre pressure would alter the contact area of the tyre and subsequently the load intensity would change, i.e.

$$Load\ Intensity = \frac{Wheel\ load}{Contact\ Area} \qquad \text{... Eq. (3-4)}$$

In practice, conventional road tyres have reasonably stiff walls, resulting in negligible change in contact width but considerable change in contact length as tyre pressure is varied (Scala, 1970). As the effect of an increase in tyre pressure revealed only a minimal effect on the measured deflection (HRB, 1962a; Bonaquist et al, 1989; Sebaaly and Tabatabaee, 1989), the effect of these factors should therefore also have a minimal effect on deflections, but needs to be investigated further under local conditions in South Africa.

(h) ***Vehicle travelling speed***: During the testing of experimental test sections (HRB, 1962a), a pronounced reduction in deflection with an increase in vehicle travel speed was found to exist. A much greater reduction in curvature was observed with an increase in speed, than that observed in deflection. A formula constructed to relate the percentage reduction in deflection to travel speed was:

$$d_{\%} = 100\ (1 - 10^{33\ A_1}) \qquad \text{... Eq. (3-5)}$$

where: $d_{\%}$ = percentage reduction in deflection

A_1 = speed coefficient determined from regression

Results in general showed that a concomitant increase in axle load with vehicle speed, resulted in lower percentage reductions in deflections ($d_{\%}$) for the heavier axle loads. Therefore the speed coefficient (A_1) subsequently reduces as axle load increases.

De Beer (1991) also measured a decrease in deflection with an increase in vehicle speed on a smooth pavement. This aspect, however, also needs to be further investigated in South Africa, especially towards the effects of different loads on pavement response and hence pavement damage.

Investigations by Lourens (SARB, 1992b) showed that the difference in maximum surface deflections between tandem axles at different speeds is lower than the difference in stresses in the asphalt layer at the same speeds, thus, the surface deflection tends to mask the magnitude of distress which may be induced in certain layers by a change in speed.

(i) ***Load application rate***: The rate at which a particular load is applied and withdrawn was recognised (Govind and Walton, 1989) to have a marked influence on the performance of especially bituminous materials, i.e. materials exhibiting visco-elastic behaviour (Freeme and de Beer, 1987).

(j) ***Wheel and axle configuration***: AASHO Road Test results (Irick and Hudson, 1964) indicated that for two single axles brought together as a tandem to produce the same damage (equal LEF) as the standard 80 kN (18 kip) axle, the load per axle is 73,5 kN, i.e. a load per axle decrease of 8 %. Hajek and Agarwal (1989) concluded that within the practical range of axle spacings, pavement damage can be significantly reduced by increasing axle spacings. Calculated LEFs based on surface deflections (Hajek and Agarwal, 1989) showed that a dual-axle, spaced at 1 m, causes the same damage (equal LEF) as two 10000 kg single axles for which the corresponding load per axle for the dual-axle configuration is 7450 kg. By increasing the axle spacing to 1,8 m, the resultant permissible load per axle increases to 8350 kg. Maximum deflections would therefore be lower in cases where large axle spacings are encountered, while the intermediate deflections would be higher for larger axle spacings. The reason being that for larger spacings, the pavement would have time to recover from the deformation caused by the first wheel load before the arrival of the second wheel. This effect seem to be more pronounced in thinner pavement structures. Scala (1970) concluded from his study, that the load on a single axle with single tyres which produce the same deflection as a standard axle (80 kN dual-tyre single axle) is 53 kN. Thus, both the wheel and axle configuration influence the magnitude of deflections and subsequently, the LEFs calculated using deflections as the damage parameter. Due to the inertia and damping characteristics of the asphalt pavement, a remnant stress is retained in the pavement a short while after the leading wheel of a tandem axle has passed a ceratin point, and is therefore added to the stress induced by the trailing wheel as it passes the same point (SARB, 1992b). This effect is highly dependent on the speed of the vehicle, i.e. the higher the speed the greater the overlapping stress. However, an increase in speed has the effect of reducing stresses, being this effect of overriding importance over the former one (SARB, 1992b).

(k) ***Measuring technique***: The technique used to measure deflections could have a marked influence on the magnitude of deflection, as some techniques measure creep speed deflections (Benkelman Beam, Lacroix Deflectometer, Road Surface Deflectometer (RSD)) or dynamic impact deflections (Impulse Deflectometer (IDM), Falling Weight Deflectometer (FWD) and Dynaflect). In general deflection surveys refer to Benkelman beam deflections, which is used as the standard method of measuring deflections in the past, although the measurement of dynamic deflections are viewed by some to be a more accurate means of expressing actual traffic conditions (HRB, 1962a; Bonaquist et al, 1989; Queiroz et al, 1982).

3.3. LEF PREDICTIONS FROM DEFLECTION-BASED MODELS USING LOCAL DATA

From a recent deflection and axle load study conducted on the N4 near Pretoria (detail given in Appendix A), LEFs for several heavy vehicles were calculated using the proposed deflection-based method (Christison, 1986). The results were compared with LEFs calculated from other widely used and accepted models such as the AASHO LEFs (AASHTO, 1974), simplifications of the AASHO LEFs i.e. the "fourth power law" ($n = 4$)(Irick and Hudson, 1964), the method developed by the California Division of Highways ($n = 4,2$)(Hveem and Sherman, 1963) and the HVS-based method currently used in South Africa. Separate comparisons are made for the following axle configurations:

(a) Single axles with conventional single tyres;

(b) Single axles with conventional dual tyres; and

(c) Tandem axles with conventional dual tyres.

Comparisons are made on the basis of LEF for the axle group vs. loading on the axle group and are presented as a series of graphs shown in Figures 3-2 to 3-4. The objective of this study was to compare results obtained using deflection-based methods to those obtained from other widely used and accepted methods which includes the HVS-based method currently used in South Africa. Factors such as axle spacing, vehicle speed and wheel spacing were not included as part of the field investigation discussed here, although these variables were recorded to ensure control and/or to minimize the effect these variables have on the calculation of LEFs. However, variation in wheel and axle spacings within an axle group, i.e. single or tandem axle arrangements, were found to be minimal for this investigation. Axle spacings within a tandem arrangement varied between 1,3 m to 1,4 m (centre to centre) and the wheel spacings between 350 and 360 mm (centre to centre). For single axles with dual tyres, the range in wheel spacings was somewhat wider, i.e. 280 mm to 350 mm (centre to centre). Where significant variations from these norms occurred, data was discarded in order to control the effect of these variables. A more detailed reference to the test is given in Appendix A.

3.3.1 Single axles with conventional single tyres

Apart from the deflection-based method, the other methods depicted in Figure 3-2 do not distinguish between different wheel configurations, viz, the methods do not differentiate between single axles with single wheels and single axles with dual wheels (effects are assumed to be equal). Consequently the LEFs calculated using the AASHO, AASHO related (Power law relationships with n = 4 and n = 4,2) and the HVS-based methods, strictly apply to single axles with conventional dual wheels. Notably higher LEFs are calculated using the deflection based method. Indications are therefore that single axles with conventional single wheels are far more destructive than equally loaded dual wheel axles. Results further indicate that the equivalent axle load on an axle with conventional single wheels, is approximately 60 kN and supports the findings of Scala and Potter (1981)(equivalent single tyre axle load found to be 53 kN). The axle load of 60 kN was obtained based on the findings of a tests conducted on a single pavement. Using a linear elastic multi-layer programme (ELSYM5) and selecting several HVS test sections (see Appendix B for pavement structures) the load of a single wheel that caused the same deflection as under the standard dual wheel load was calculated. The use of static analysis to model HVS test is considered valid due to the slow wheel speed. The corresponding axle loads are given in Table 3-1 and the average calculated axle load was found to be 52 kN (range in values of 41,7 kN to 57,6 kN). As evident from Table 3-1, a fair degree of variation was found in the calculated values. However, the calculation process is highly dependent on the input parameters used, such as the chosen modulus of elasticity, and therefore this mechanistic modelling effort may incorporate inaccuracies. The purpose of the mechanistic analysis was therefore to supplement field results and to highlight that discrepancies could occur.

Table 3-1: Equivalent axle loads for single axles with single wheels and tandem axles with dual wheels obtained from mechanistic analysis based on equal deflection.

Wheel and axle configuration	Axle Group Load (kN)				
	Section 1	Section 2	Section 3	Section 4	Avg.
Single axle with single wheels	58	53	55	42	52
Single axle with dual wheels*	80	80	80	80	80
Tandem axle with dual wheels**	144	130	127	140	135

* Standard axle configuration.

** Inter-axle spacing: 1,40 m.

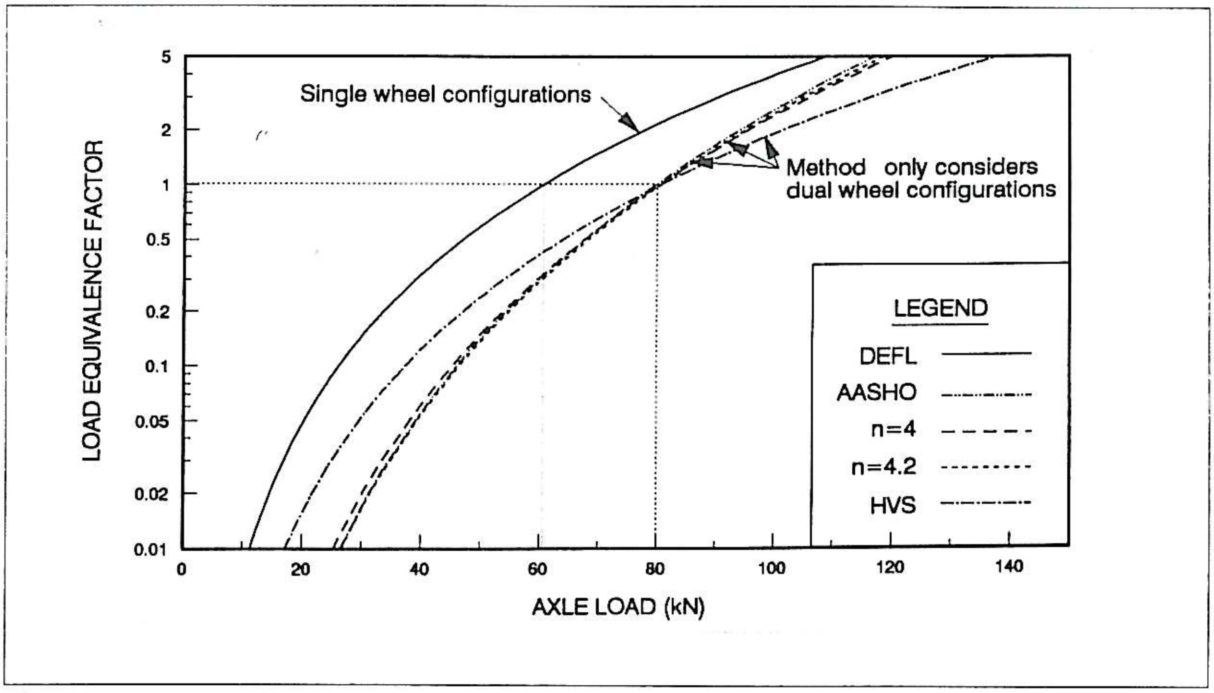


Figure 3-2: Load Equivalency Factors (LEFs) for single axles with single and dual wheels.

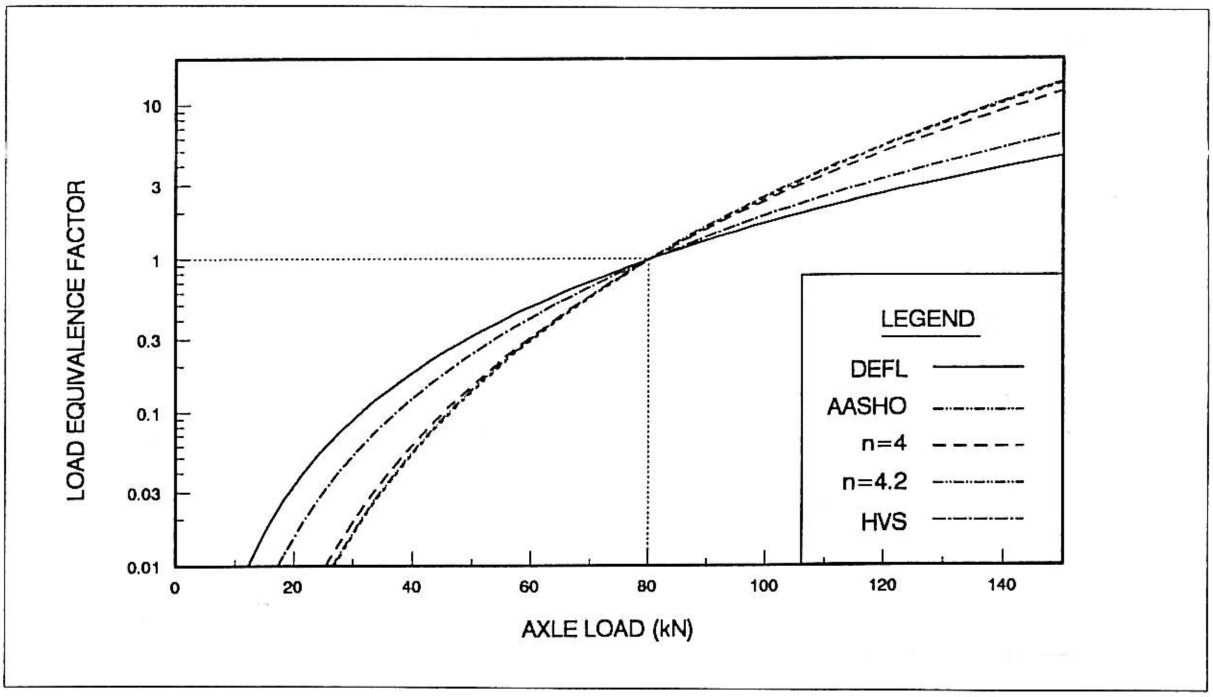


Figure 3-3: Load Equivalency Factors (LEFs) for single axles with dual wheels.

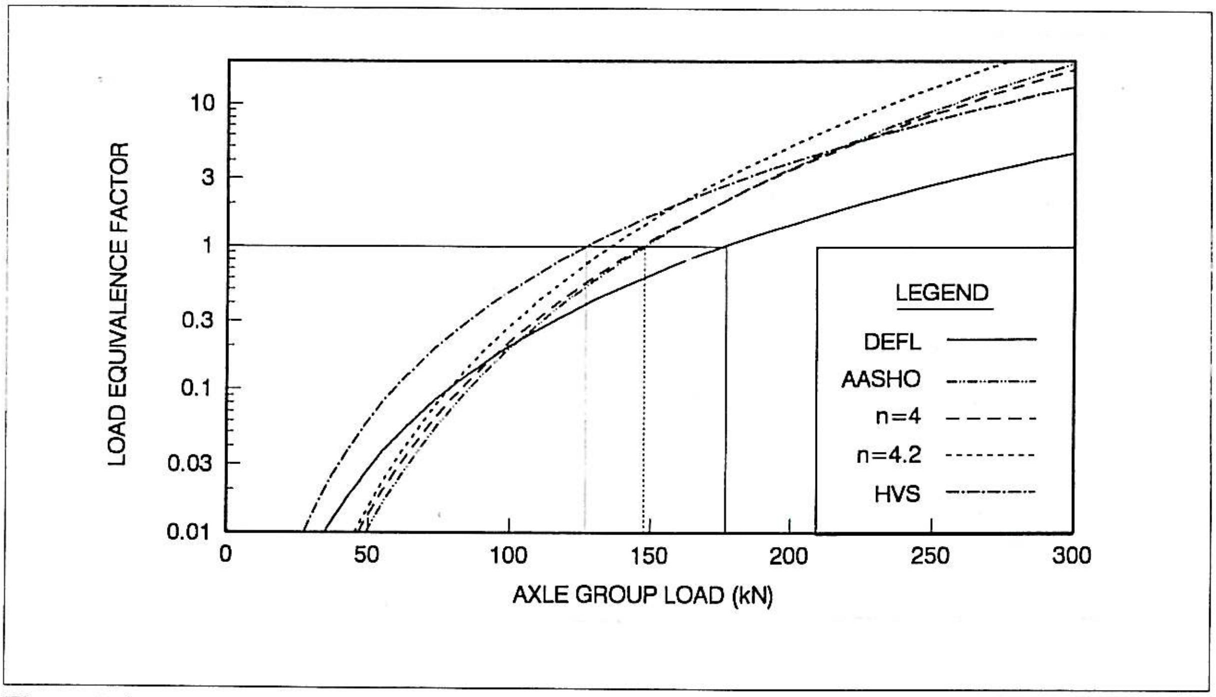


Figure 3-4: Load Equivalency Factors (LEFs) for tandem axles with dual wheels.

3.3.2 Single axles with conventional dual tyres

The AASHO and AASHO-related methods (power law relationships) are all in close agreement, with the deflection-method yielding somewhat higher LEFs for axle loads lower than the standard axle load (80 kN) and lower LEFs for axle loads above this limit. As expected, for axle loads of 80 kN (standard axle load) all methods yield LEF values of 1. LEFs calculated by means of the HVS-based method lie in between values obtained from AASHO-related methods and those from the deflection-based method (Figure 3-3).

3.3.3 Tandem axles with conventional dual tyres

The results of the deflection-based LEFs calculated for tandem axles are shown in Figure 3-4. In general, use of the deflection method resulted in lower calculated LEFs, which indicate higher equivalent axle loads on a tandem axle arrangements (equivalent axle load on a tandem axle configuration of approximately 180 kN) as opposed to other methods, for example AASHO which gives 147 kN. Similarly, as for the case with single axles with single wheels, results of a mechanistic analysis, which was performed to determine the equivalent load on a tandem axle for other pavements and pavement types (see Appendix B), are shown in Table 3-1.

The respective results indicate an average equivalent tandem axle load of 135 kN (range of between 127 kN and 144 kN). This large difference in calculated equivalent loads (180 kN as opposed to 135 kN) can be attributed to differences in the methods used to determine LEFs, viz, the use of a measured vs calculated deflections. Furthermore, the magnitude of the calculated deflections are highly dependent on the mechanistic model used, i.e. the assumption of linear elastic layer theory and the accuracy of the input parameters, such as the modulus of elasticity assumed for the respective layers. Also the loading speed becomes an important parameter here, because deflections alone tend to *mask* the distress parameters, as mentioned earlier.

3.4. SUMMARY OF LEF PREDICTIONS AND COMPARATIVE STUDY

It may be seen that no clear pattern emerges from the comparisons made. For the case of the single axles with conventional dual tyres, results are in close agreement. However, for the other cases some discrepancy exists. This could reasonably attributed to the fact that the methods use different approaches and apart from the deflection based method, do not take cognisance of factors such as number and spacing of the wheels and/or axles and dynamics.

A summary of the deflection-based results are shown in Figure 3-5. From this plot of axle load vs. LEF, the effect of wheel and axle configuration on LEF calculations are evident and therefore clearly illustrates the importance of incorporating these factors into the calculations. Results indicate that the equivalent axle loads, i.e. axle loads which are equivalent to the standard 80 kN dual-tyre single axle load, differ for the cases evaluated in this study. For example, based on the results obtained from the field study presented earlier, the equivalent axle load (LEF = 1) on a single axle with conventional single wheels is 60 kN. Thus, the load of a single axle with single tyres to cause the same damage as the standard 80 kN axle with dual wheels is 25 percent lower. Similarly for tandem axle configurations, the equivalent axle load is 180 kN. This, in turn, implies that a tandem axle of 180 kN (90 kN/axle) is equivalent to two standard axles (80 kN with dual wheels) far apart..

Curves such as shown in Figure 3-5 facilitate the determination of permissible axle loads (maximum load on an axle to result in the same damage caused by the 80 kN dual-wheel single axle) on wheel and axle configurations other than the standard. A summary of the results is presented in Table 3-2. However, it is important to borne in mind that these values are based on the results obtained from a single investigation conducted on a single pavement and therefore need further verification (See Table 4-3 in Section 4.4.5 later).

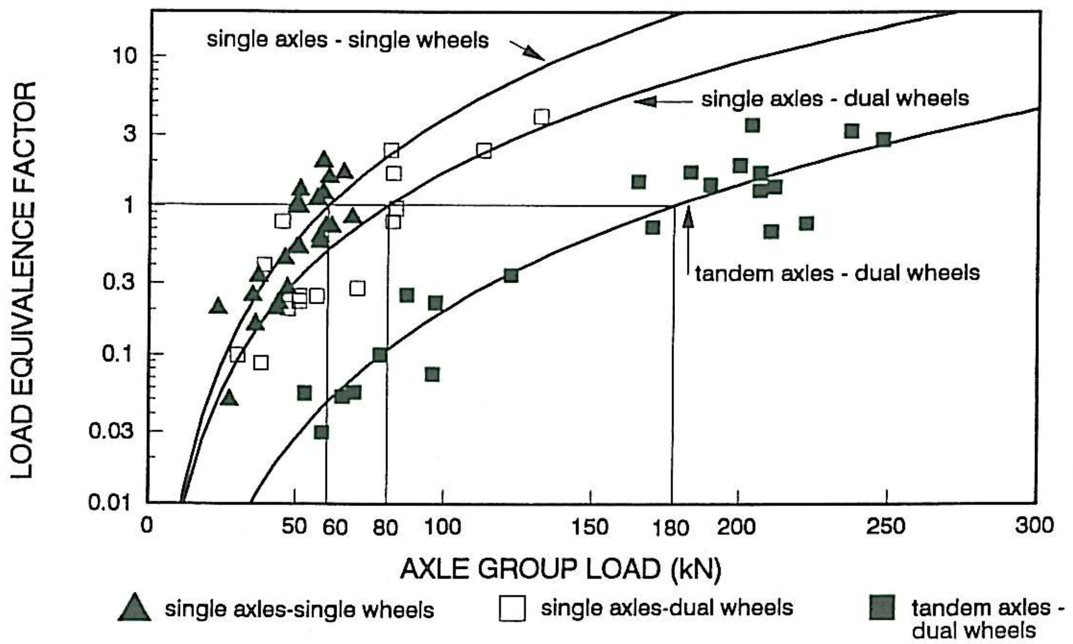


Figure 3-5: Load Equivalency Factors (LEFs) calculated from surface deflections measured on the N4 between Pretoria and Witbank.

Table 3-2: Summary of equivalent axle loads calculated from field tests data using deflection based LEFs (from Figure 3-5)

Wheel and axle configuration	Equivalent axle load (kN)
Single axle with single wheels	60
Single axle with dual wheels*	80
Tandem axle with dual wheels**	180***

* Standard axle configuration.

** Inter-axle spacing: 1.40 m.

*** Total for tandem axle.

Although the suggested deflection-based method (Christison, 1986) overcomes many of the shortcomings of the load-damage-based methods (AASHO and related), the comparison of the field and the mechanistic studies shows some limitations of the suggested method:

(a) The calculated "equivalent load" for the tandem axle is found to be unusually high, however for single axles with single wheels the equivalent load is acceptable.

(b) The procedure is too dependent on the deflection under the standard reference axle load (D_b in Equation 3-1). Small variation in this reference deflection produces a great change in the calculated LEF and therefore in the equivalent load.

(c) Existing load-damage-based methods are not flexible enough to assess the damage of typical wheel load configurations.

At this stage a more comprehensive method which combines the advantages of load-damage and pavement response-based methods in one procedure is proposed. This method is explained in the following sections. The method combines the two main existing techniques of obtaining LEFs (i.e. load-based relationships and response parameters). LEFs are calculated from the pavement stress and strain response and are related to load configuration and magnitude.

4. PROPOSED METHOD FOR THE DETERMINATION OF EQUIVALENT DAMAGE FACTORS BASED ON THE SOUTH AFRICAN MECHANISTIC DESIGN PROCEDURE

4.1 INTRODUCTION TO THE METHOD

A new method to determine equivalency factors for multiple wheel load configurations is herein proposed in an attempt to overcome some of the shortcomings of the existing methods as identified and discussed in the previous sections. The proposed method takes into account different failure criteria through use of the South African Mechanistic Design Method (Freeme, 1983). The approach uses linear elastic multi-layer theory to calculate pavement response under different loading conditions combined with performance curves or "transfer functions" relating stresses and strains to "pavement life" prediction. These transfer functions were obtained from laboratory tests as well as from field accelerated testing with the Heavy Vehicle Simulator (HVS) (NITRR, 1984).

The method is based on the concept of "equal pavement response equal pavement damage". However a different approach to the meaning of equal response than previously used (Scala and Potter, 1981) is now defined. In much of the previous work equivalent response is based on equal maximum surface deflection. The authors consider that this assumption is only valid, or approximately valid, when the performance of the pavement is governed by the behaviour of the lower layers, i.e. selected layers or subgrade. The approach has some shortcomings due to surface deflection not being directly related to some failure parameters. In the new method each pavement is associated with a particular distress mode governed by a specific pavement response (stress or strain). Thus, for example, a pavement that fails after reaching a given strain level at a specific position, is defined as having equal response to different loads only if the same strain level is caused at the same position.

The principal aim of this section is to establish a method of calculating equivalent damage of various traffic load and axle configurations on pavements in different states of behaviour. Once this has been achieved, the necessary tools are to be made available to designers and legislators to convert normally loaded truck axle configurations into the corresponding equivalent number of standard axle loads. In addition, suggestions are to be made in order to extend the applicability of the methodology to assess the equivalency of abnormal axle configurations.

The new method separates the traditional damage factor into three main components: the effect of axle groups, the effect of load and the effect of tyre pressure (see Equation 4-2).

4.1.1 General definitions

A number of terms that are used frequently in the following paragraphs are defined below in order to unified concepts:

Pavement life, is the period during which it is predicted that no major maintenance or rehabilitation works will be required in order to maintain an acceptable level of service of the road pavement. The expressions **life, pavement life, allowable number of repetitions**, or **repetitions to failure** all have the same meaning in this section.

Repetitions, refers to the number of applications of a specific load configuration to the pavement. Thus the life of the pavement may be expressed as the number of applications of an specific load that causes cumulative damage that necessitates major maintenance or rehabilitation work on the pavement.

Critical Life or **Group Life**, refers to the life of the pavement under the application of a unique load configuration - in these cases the critical axle or the group of axles, respectively. In order to standardize criteria the standard load adopted in this work is an 80 kN single axle with dual wheels at 520 kPa tyre pressure. This represents the legal load limit used in South Africa for this axle configuration. This reference load will be referred in this report as the **Standard Load Configuration** (SLC). By using SLC the life is expressed in equivalent 80 kN loads (E80s).

Load configuration, refers to a given loading condition and wheel arrangement. The load conditions or characteristics are load magnitude or force (from now on referred to as load) and load intensity or pressure. The force in kN, corresponds to the load of the axle unless otherwise indicated. The load intensity in kPa represents contact pressure and will be assumed equal to the tyre pressure for the purpose of this study. The load arrangement is referred to as the load distribution on the surface of the pavement. The number of axles per group, inter-axle distance, number of wheels per axle (single or dual), and inter-wheel distances are the characteristics of specific load arrangements.

4.1.2 Specific terms

Standard axle ($STDA_{P/\sigma}$), is herein defined as a single axle with dual wheels which are separated by 350 mm centre to centre. The axle load is **P** evenly divided between four wheels, i.e. axle load = 4 x wheel load; the tyre pressure is designated as σ. Thus, the standard axle of 20 kN per wheel with a tyre pressure of 520 kPa ($STDA_{80/520}$) is taken as the **Standard Load Configuration (SLC)** and used as the reference configuration.

The allowable number of repetitions of the standard axle or standard axle life ($N_{P/\sigma}$) refers to the number of applications of standard axles of load **P** and tyre pressure σ that a certain pavement can carry before failure. Thus, $N_{80/520}$ is the allowable number of repetitions of a standard axle of 80 kN and 520 kPa that a specific pavement can take. Therefore: $N_{80/520}$ is equal to the number of E80s.

Critical Life (N_{cr}) is the number of repetitions of the most critical axle (the most damaging axle) of a group of axles that can be applied before a pavement reaches a failure condition. In this definition the critical axle is analyzed ignoring the contribution of the other axles in the group.

Group Life (N_G): is the allowable number of repetitions of a group of axles that a pavement can withstand. It considers the critical axle *plus* the contribution of all the other axles of the group. N_G is determined by applying a **Weighting Factor** (WF) to the N_{cr} (as explained below).

The deflection ratio (r), is the ratio of the smallest peak deflection under a group of axles to the peak deflection under the standard axle of the same load. It is used to calculate the contribution of non-critical axles to pavement damage. The minor peak deflection under the group in the case of a tandem can be caused by either of the axles which are equally damaging. In the case of a tridem axle, the peak deflection under either the first or third axle is used to obtain the appropriate value of r (Figure 4-1). It should be borne in mind that these deflection are based on static analysis; deflections under multiple axles groups may differ substantially (SARB, 1992). The deflection ratio should correspond to those axles of the group that are not critical in order to assess their contribution to the total damage.

Contribution Factor (CF), is a factor that was developed to take into account the contribution of the second axle of a tandem to the total pavement damage or the contribution of the first and third axles in the case of a tridem configuration.

By definition:

$CF = N_A - r,$

where: N_A : is the number of axles of the group, and

r : deflection ratio.

In the definition of CF, **r** was defined in terms of surface deflection ratio. In the same way, and actually more precisely, it may be defined in terms of a strain ratio or even a stress ratio. Recent research proved that surface deflections tend to mask actual pavement distress (SARB, 1992b). However, the definition of **r** in terms of surface deflection ratio was preferred because under most conditions it is more direct and therefore more realistic to measure deflections than to calculate strains or stresses in the pavement. It is the authors' opinion that a more precise method would consist of first identifying the failure mode (based on either a critical stress or strain) and only then, defining **r** in terms of the appropriate response parameter. This method would be more precise but would also make the determination of **r** too complicated for general practical use.

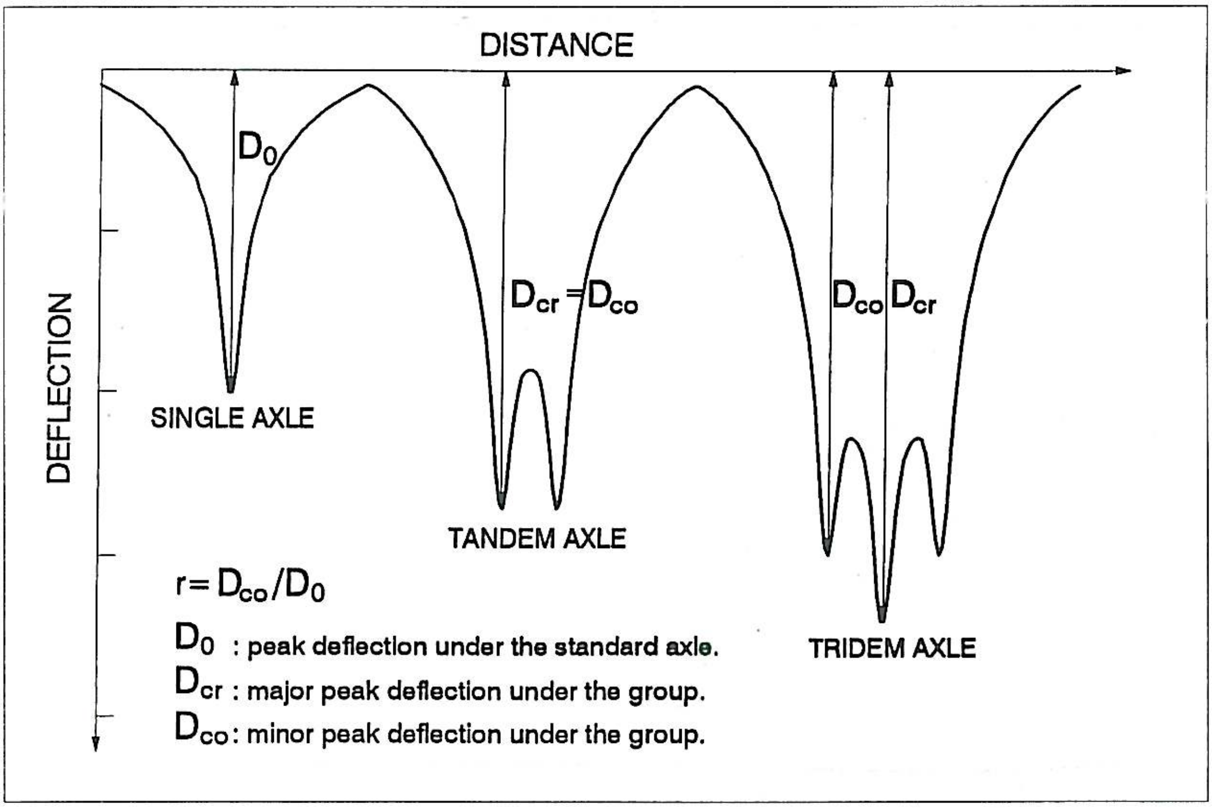


Figure 4-1: Deflection ratio determination based on static analysis (creep speed).

Weighting Factor (WF), is defined as the ratio between the allowable number of repetitions of the most critical axle of a group (N_{cr}) to the allowable number of repetitions of the group (N_G):

WF = Number of Repetitions of the Critical Axle / Number of Repetitions of the Group

i.e. $WF = 1 + CF$

and $N_G = N_{cr} / (1 + CF)$

These two factors (CF and WF) were developed in an attempt to quantify the cumulative damage of multiple axle loads when applied at different distances apart. The case of the tandem axle is now explained and similar reasoning can be followed to define them for a number of axles greater than two.

Consider two equal axles moving at creep speed, i.e. a tandem axle group. When these axles are positioned such that the displacements, strains or stresses produced within the pavement layers by one axle are not affected by the other, the total damage of the axle group can be assumed to be twice the damage of one of the axles. The WF is therefore equal to two, as the allowable number of repetitions of the group is equal to the allowable number of repetitions of either axle (both being critical) divided by two.

Following the definitions:

$CF = N_A - r$, where $N_A = 2$ (tandem axle), and $r = 1$ (the deflection under one axle is not affecting the other). Then:

$CF = 2 - 1 = 1$, i.e. the contribution of the second axle is the same as the first axle, and

$WF = 1 + CF = 2$

As the axles are brought closer to one another, load effects will interact and displacements, stresses and strains will increase. In the extreme case, when the two loads are superimposed deflections will double, as will stresses and strains (if linear elastic theory is used). At this point the life of the group will be the same as the life of the critical axle (which will be a single axle with double the load). The WF will be one, so the group life will be the same as the critical life.

Now following the definitions:

CF = 2 - 2 = 0, as the deflection under the critical axle will be double the deflection under the standard axle of the same load (r = 2). Therefore:

WF = 1 + CF = 1

Based on the linear-elastic approach, for all intermediate cases the weighting factor should be within this range of one to two. The assumption of a linear relationship between deflection ratio and weighting factor has been made for simplicity (see Figure 4-2) and needs to be verified. The weighting factor can also be calculated by applying Equation 2-37 (Christison, 1986) which implies the measurement of the deflection under the various axles of the group. However, any error which the linear relationship assumption may carry into the final equivalency factor is small in comparison to some of the assumptions which are made in some other methods and is therefore thought negligible for the purpose of this study[1].

Group Equivalency Factor (GEF), is defined as the ratio between the allowable number of repetitions of a standard axle ($N_{P/\sigma}$) to the allowable number of repetitions of a group of axles with the same loading conditions (N_G). This factor expresses the number of standard axles of load **P** and tyre pressure σ that would cause the same damage to the pavement as the group of axles with the same loading characteristics per axle, i.e. same load **P** and same tyre pressure **σ**. These factors are developed for specific pavements so they must be applied only to pavements of the same type as the ones they were developed from, i.e. light granular base, heavy granular base, light cemented base, etc. Each pavement type is associated with a specific failure mechanism, therefore variations on the selected layer thicknesses or elastic properties of the materials might cause a change in the failure

[1]At this point the necessity of upgrading the full scale testing facility (HVS) to accommodate tandem or tridem axle configurations is emphasized, as it seems the most direct way of assessing the equivalency of different load arrangements. The Mobile Load Simulator (MLS) (Hugo, 1994) may also contribute in this regard. Nevertheless, laboratory fatigue testing of road materials is suggested at this stage as it is thought that this can provide initial guidelines for analysis of material response to loading cycles.

mode and then the GEF will change accordingly. Per definition the value of GEF for single axles is one (independently of the number of wheels).

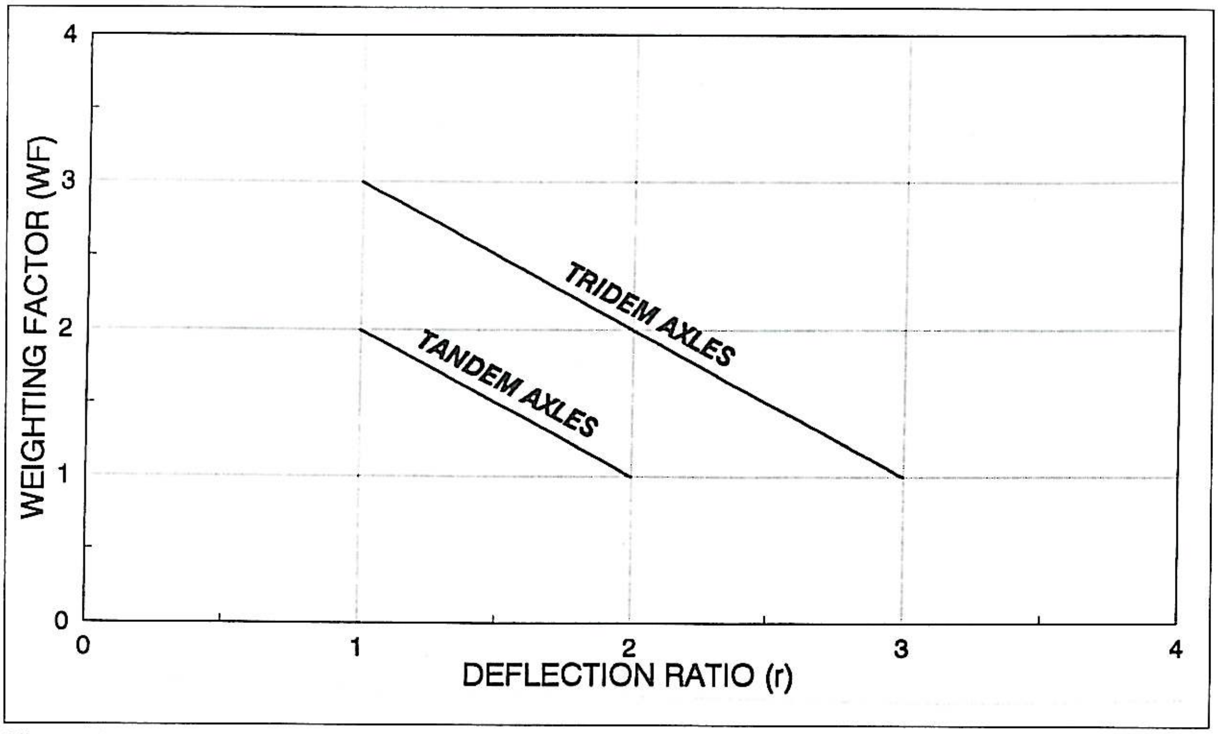


Figure 4-2: Relationship between deflection ratio and weighting factor.

Axle Load Factor (ALF), is defined as the ratio between $N_{80/\sigma}$ and $N_{P/\sigma}$. This factor represents the number of standard axles of 80 kN that would cause the same damage as a standard axle of any load P (different load, same tyre pressure). This factor is pavement dependent.

Load Ratio (P/80), is the ratio between the axle load expressed in kN and 80 which is considered as the standard load magnitude for a single axle with dual wheels. For a standard axle configuration (single axle, dual wheels), P is four times the wheel load in kN.

Load damage coefficient 'α', is defined as the slope of the linear regression calculated from the Axle Load Factor (ALF) versus Load Ratio (P/80) as represented on a log-log scale. Then, ALF = $(P/80)^{\alpha}$ (c.f. "n" in $(P/80)^{n}$ in TRH4 (CSRA, 1989)).

Tyre Pressure Factor (TPF), is the ratio between $N_{80/520}$ to $N_{80/\sigma}$. The equivalent damage effect of tyre pressure variation is assessed with this factor. This factor represents the number of axles of

Standard Load Configuration (80 kN load and 520 kPa tyre inflation pressure) that would cause the same damage as a standard axle of 80 kN and any tyre pressure σ.

Tyre Pressure Ratio (TPR, $\sigma/520$), is the ratio between the actual tyre pressure expressed in kPa to 520 which was adopted as the standard tyre pressure in this study.

Pressure damage coefficient 'ß', represents the slope of the linear regression calculated from the log-log relationship between Tyre Pressure Factor (log TPF) and Tyre Pressure Ratio (log TPR). Thus, TPF = $(\sigma/520)^{\beta}$.

Single Wheel Factor (SWF), is the ratio of the life of the pavement under the SLC or standard axle life to the life of the pavement calculated using a single axle with single wheels of a given load.

Equivalent Single Wheel Axle Load (ESWAL), is the load on an axle with single wheels that is calculated to give the same life as the Standard Load Configuration (SLC). As per definition, this load corresponds to an SWF of one (1).

Equivalent Damage Factor (EDF), is defined as the product of GEF, ALF and TPF. In this way, EDF expresses the number of standard axles of 80 kN and 520 kPa (SLC) which would cause the *same damage* as a group of axles of any load and tyre pressure. Up to date this has been expressed using "E80s" calculated by $(P/80)^n$.

The EDF of a given vehicle, is the number of repetitions of the Standard Load Configuration (SLC) that should be applied to a pavement in order to cause the same amount of damage as one repetition of such a vehicle. Although the meaning of the terms EDF and E80 are similar to the well known and widely used LEF and ESAL respectively (AASHO, 1974), a different notation is proposed in order to distinguish between the different approaches adopted in the AASHO Road Test based methods and the present method. The EDF of the vehicle is the sum of the EDFs of each axle group constituting the vehicle. For individual axles to be part of an axle group (tandem or tridem), the inter-axle spacing is to be less than 3,0 metres[2].

[2]This limit is proposed because it was found that for greater inter axle spacings, the influence of the trailing axle of a tandem group was less that 10 %.

The expression for the determination of the EDF of a given vehicle is:

$$EDF_v = \sum_{i=1}^{n} EDF_i \qquad \text{... Eq. (4-1)}$$

where EDF_v : Equivalent Damage Factor of the vehicle

EDF_i : Equivalent Damage Factor of the axle group I

n : number of axle groups of the vehicle.

$$EDF_i = GEF_i \times ALF_i \times TPF_i \qquad \text{... Eq. (4-2)}$$

For single axles with dual or single wheels, GEF is equal to one; for single axles with single wheels, ALF_i is to be replaced by SWF_i.

It is important to state that the entire procedure is based on the assumption of linear-elastic material characterization. It is appreciated that the assumptions are probably only valid for the theoretical study. Nevertheless since all the partial factors defined above are relative comparisons, it is the authors' opinion that this characterization is more than adequate for the short term. Further research is suggested in order to assess the applicability of this working method to actual material behaviour, and with a much broader scope, use of non-linear material models in the mechanistic design technique.

4.2 THE SOUTH AFRICAN MECHANISTIC DESIGN PROCEDURE

The SA Mechanistic Design procedure has principally been developed over the past two decades and includes both flexible and rigid pavement types. A summary of the method is given by Maree and Freeme (1981). A comprehensive use of the technique takes into account factors relating to design strategy including road category, traffic volumes and structural design period, and considers material types, environment, drainage, compaction and cost analysis. A less involved approach is that of the catalogue of designs which is typically used as a preliminary assessment of the pavement type required. Good descriptions of some of the development of the SA mechanistic approach is given by Walker et at (1977) and Paterson and Maree (1978). The basic approach of the method has not altered to any great extent since the publication of mentioned references but better quantification of existing failure criteria and recognition of new ones have occurred (De Beer, 1992). A description of the failure criteria used in the analysis of pavement sections in the study is given in Table 4-1 and discussed below.

Table 4-1: Failure criteria used in the mechanistic analysis

Material Type and layer	Failure Criteria	Inputs Required for Analysis	Critical Position in Layer*
Granular Base	Shear Failure (Factor of Safety)	σ_1, σ_3	Middle
Granular Subbase	Rutting	ϵ_z	Top
Cemented Base	Crushing (N_c) Effective Fatigue (N_{ef}) Shear Failure (in equivalent granular phase)	σ_z ϵ_h σ_1, σ_3	Top Bottom Middle
Cemented Subbase	Crushing (N_c) Effective Fatigue (N_{ef}) Rutting (in equivalent granular phase)	σ_z ϵ_h ϵ_z	Top Bottom Top
Asphalt Surfacing (20-75mm thick)	Flexural Fatigue Cracking	ϵ_h	Bottom
Asphalt Base (>75mm)	Flexural Fatigue Cracking	ϵ_h	Bottom
Subgrade	Rutting	ϵ_z	Top

* The suggested critical position in a layer should be checked to ensure that it actually is the most critical for any of the parameters given (Freeme, 1983; Jordaan, 1988).

4.2.1 Failure criteria

4.2.1.1 Thin Asphalt Surfacing (<75mm layer thickness)

The failure mode for these materials has been identified as fatigue failure which is caused by excessive tensile strains in the material - usually on the bottom surface. Much of the work in this field was carried out by Freeme (1971) and Freeme and Marais (1973) who show the correlation of laboratory and field test results.

An important difference between thin surfacing (20-75 mm) and relatively thick bituminous bases (>75 mm) is that the failure of thin surfacing is more dependent on tyre pressure than wheel load, hence the correlation between induced tensile strain and repetitions of the particular load under investigation is used in the criteria, and not standard 80 kN axle loading (Freeme, 1983).

4.2.1.2 Bituminous bases (>75 mm)

Bituminous bases typically fail through flexural fatigue-induced cracking although permanent deformation can also be a problem in relatively hot conditions. The possibility of deformation (rut) failure is normally minimised by mix design, and thus for purposes of structural analysis, only layer thicknesses, material stiffness and applied loads are used to calculate the maximum tensile strains for fatigue analysis.

As the layers considered are relatively thick, some allowance has to be made for the propagation of cracks (usually initiated in the bottom of the layer) through the layer. Shift factors (depending on the road category) are therefore multiplied by the number of repetitions calculated for crack initiation.

4.2.1.3 Granular bases

Granular materials are to a greater or lesser degree stress-dependent and normally show distress through permanent deformation (densification due to micro-shear) or inadequate stability (macro-shear), both having been shown to be related to material shear strength (Maree, 1978). Crushing of particles also contributes to the total distress but its contribution its negligible compared with the shear mechanisms.

The safety factor approach (Maree, 1978; Freeme, 1983) has been developed for base layers of these materials and has the objective of limiting shear stresses and thus safeguard the layer from excessive shear deformation and shear failure. The allowable shear stress in the layer can be calculated from the maximum single load shear strength (expressed by the Mohr-Coulomb strength parameters). In simple terms the reasoning for this is that if stresses are kept within the elastic limit, then only deformation due to densification occurs until an equilibrium state is reached for that stress level. Thereafter, if that stress (within elastic limits) increases, the material will deform to a new equilibrium state. Finally, when stresses exceed the elastic limit, deformation occurs at different rates according to the stress level and material type. Recommendations have been made regarding permissible values for the Factor of Safety for different road categories (Maree, 1978).

4.2.1.4 Cemented layers

Much of the initial work carried out on cemented materials was by Otte (1978) who developed formulae and criteria predicting crack initiation, largely through laboratory work. After a number of years de Beer used HVS generated field test data and observations to improve the existing and develop new criteria for lightly cemented (C3/C4) materials (De Beer, 1990). For example, the number of traffic repetitions required to develop crack initiation in cemented layers (N_f) is not considered a final failure mode, whereas cracks propagating through the layer is. The number of load repetitions required for this to occur is termed the Effective Fatigue life (N_{ef}). Jordaan (1988) developed separete models for strongly cemented materials (C1/C2).

For both shallow and deep (De Beer et al, 1988) cemented-base pavement structures, N_{ef} is defined as where surface deflection is between 0.5 - 0.75 mm with an associated permanent deformation of 2 mm. Once this stage has been reached and cracks have propagated through the layer, the increasing vertical deflection incurred as the cemented material breaks into progressively smaller blocks and changes into an equivalent granular layer. Therefore pavements with cemented layers fail in successive phases. A pavement may go through as many intermediate phases as cemented layers it has. For example a pavement with a cemented base and subbase might fail in the following phases: firstly, effective fatigue of the cemented subbase which will change into an equivalent granular layer (Phase I); then, effective fatigue of the cemented base and transformation into an equivalent granular (Phase II), and finally, failure of either the surface, the granular base, the granular subbase or the subgrade (Phase III, pavement failure).

An additional failure mechanism of fatigue failure has been recognised and defined by de Beer (1992) as being that of crushing failure at the top of lightly cementitious pavement layers. N_c is defined as the number of load repetitions required for 1% "permanent deformation" in the cemented layer. For thinly surfaced pavements with cemented bases the permanent deformation required for crushing failure causes cracking in the seal almost immediately, leading to water ingress, pumping and disintegration of the pavement and potholes.

Tentative relationships for 5 and 10 mm crushing deformation are also available, as are expressions for use in predicting rutting in granular (or equivalent granular) materials below the base (De Beer, 1989a).

4.2.1.5 Subgrade

Elastic vertical subgrade strain-traffic repetition relationships used in the South African Mechanistic Design procedure are modifications of the correlations proposed by Dorman and Metcalf (1965); these modifications are based on investigations carried out by the U. S. Corps of Engineers (Brabston et al, 1975) and local research (Marree, 1983). The present day curves correspond to a surface rut of 20 mm, but the three different standards allow different length of road to exceed that limit according to the category (A, B or C) (CSRA, 1985). Limited work by De Beer (1989a) indicated that this criteria may still be used until new research proves otherwise.

4.2.2 Transfer functions

To predict pavement life from stresses and strains (as indicated in Table 4-1), functions linking life to these responses are required, and in this report are referred as "Transfer Functions". Details of the functions used are given below (taken from Freeme, 1983 and De Beer, 1992):

4.2.2.1 Thin Asphalt Surfacing

Gap-graded asphalt with approximated stiffness 2400 MPa:

$$N = 10^{\left(\frac{3{,}4288 - \log \epsilon_h}{0{,}1454}\right)} \qquad \text{... Eq. (4-3)}$$

where: N = number of repetitions before initiation of cracking,

ϵ_h = maximum horizontal tensile strain calculated at the bottom of the layer, in microstarins.

Continuously-graded asphalt with approximated stiffness 2000 MPa:

$$N = 10^{\left(\frac{3{,}5765 - \log \epsilon_h}{0{,}1858}\right)} \qquad \text{... Eq. (4-4)}$$

where: N and ϵ_h represent the same properties as for Equation 4-3.

4.2.2.2 Bituminous Bases with approximated stiffness 1800 MPa:

$$N = 2 \times 10^{\left(\frac{2{,}7657 - \log \epsilon_h}{0{,}2301} + 4\right)} \qquad \text{... Eq. (4-5)}$$

where: N and ϵ_h represent the same properties as for Equation 4-3.

4.2.2.3 Granular Bases

Road Category A:

$$N = 10^{\left(\frac{FOS\ +\ 0,7076}{0,2983}\right)} \qquad \text{... Eq. (4-6)}$$

Road Category C:

$$N = 10^{\left(\frac{FOS\ +\ 1,4721}{0,3716}\right)} \qquad \text{... Eq. (4-7)}$$

where *FOS* denotes "Factor of Safety" (Freeme, 1983) as calculated from:

$$FOS = \frac{\sigma_3 \times \phi\ term\ +\ c\ term}{(\sigma_1\ -\ \sigma_3)} \qquad \text{... Eq. (4-8)}$$

Where c term and ϕ term represent terms accounting for material cohesion and shear resistance. Table 4-2 gives values for various materials.

Table 4-2: c term and ϕ term for calculation of Factor of Safety (Freeme, 1983)

Material, Code	Moisture State	c term (kPa)	ϕ term
High Density Crushed Stone, G1	Dry Wet	392 171	8.61 5.44
Moderate Density Crushed Stone, G2	Dry Wet	303 139	7.06 4.46
Crushed Stone and Soil Binder, G3	Dry Wet	261 115	6.22 3.93
Base Quality Gravel, G4	Dry Wet	223 109	5.50 3.47
Subbase Quality Gravel, G5	Moderate Wet	147 83	3.43 3.17
Low Quality Subbase Gravel, G6	Moderate Wet	103 64	2.88 1.76

4.2.2.4 Cemented layers

Crushing Failure (N_c):

$$N_c = 10^{8,21 \times \left(1 - \frac{\sigma_z}{1,2\ UCS}\right)} \qquad \text{... Eq. (4-9)}$$

where: σz = vertical stress

UCS = 7 day unconfined compressive strength at 100% Mod. AASHTO density for new pavements, and Dynamic Cone Penetrometer (DCP) determined UCS of the top 50 mm of the layer for existing pavements.

Effective Fatigue Failure (N_{ef}, De Beer, 1989b):

$$N = 10^{7,19 \times \left(1 - \frac{d \times \epsilon_h}{8\ \epsilon_b}\right)} \qquad \text{... Eq. (4-10)}$$

where: ϵ_h = maximum horizontal strain calculated at the bottom of the layer,

ϵ_b = tensile breaking strain of the respective class of material,

d = dimensionless factor to compensate for shrinkage cracking.

Rutting for cemented layers (in the post-cracked granular phase) below the base (De Beer, 1989b):

For 15 mm deformation:

$$N = 10^{\left(\frac{3,7164 - \log \epsilon_z}{0,1569}\right)} \qquad \text{... Eq. (4-11)}$$

and for 10 mm deformation:

$$N = 10^{\left(\frac{3,9310 - \log \epsilon_z}{0,2103}\right)} \qquad \text{... Eq. (4-12)}$$

Where ϵ_z (microstrains) is maximum vertical strain at the top of the layer (in both cases).

4.2.2.5 Subgrade

For Road Category C:

$$N = 10^{\left(\frac{3,6410 - \log \epsilon_z}{0,1081}\right)} \qquad \text{... Eq. (4-13)}$$

For Road Category A:

$$N = 10^{\left(\frac{3,1068 - \log \epsilon_z}{0,0718}\right)} \qquad \text{... Eq. (4-14)}$$

where: ϵ_z = maximum vertical strain at the top of the layer (in microstrains).

4.3 DESCRIPTION OF THE PROPOSED METHOD FOR THE DETERMINATION OF EQUIVALENT DAMAGE FACTORS (EDFs)

4.3.1 Pavement types and material properties

Eight different pavement types were investigated in order to assess the possible influence of pavement variables in Equivalent Damage Factors (EDF). The pavement structures are shown in Figure 4-3 and described below:

Pavement A: a heavy pavement with a granular base under dry conditions, Road category A and design traffic class E4. Structure: 50 mm asphalt surfacing, 150 mm G1 granular base, and two 150 mm C3 cemented subbases.

Pavement B: a heavy pavement with a granular base under wet conditions, Road category A and design traffic class E4. Structure: the same as above but with different material properties due to the wet condition.

Pavement C: a light pavement with a granular base under dry conditions, Road category C and design traffic class E1. Structure: 15 mm surface treatment or seal, 100 mm G4 granular base, 125 mm C4 subbase.

Pavement D: a light pavement with a granular base under wet conditions, Road category C and design traffic class E1. Structure: the same as Pavement C but with different material properties due to the wet conditions.

Pavement A

Layer	Poisson's Ratio	Elastic Moduli (MPa) Phase I	Phase II	Phase III
50 AG*	0.44	2000	2000	1500
150 G1*	0.35	450	450	350
150 C3*	0.35	2000	2000	250
150 C3	0.35	1500	250	250
SUBGRADE	0.35	180	180	180

Pavement B

Layer	Poisson's Ratio	Elastic Moduli (MPa) Phase I	Phase II	Phase III
50 AG*	0.44	2000	1800	1500
150 G1*	0.35	250	250	240
150 C3*	0.35	2000	1700	160
150 C3	0.35	1500	120	110
SUBGRADE	0.35	90	90	90

Pavement C

Layer	Poisson's Ratio	Elastic Moduli (MPa) Phase I	Phase II
S*	0.44	1000	1000
100 G4*	0.35	300	225
125 C4*	0.35	1000	200
SUBGRADE	0.35	140	140
	-	-	-

Pavement D

Layer	Poisson's Ratio	Elastic Moduli (MPa) Phase I	Phase II
S*	0.44	1000	1000
100 G4*	0.35	200	180
125 C4*	0.35	1000	120
SUBGRADE	0.35	70	70
	-	-	-

Pavement E

Layer	Poisson's Ratio	Elastic Moduli (MPa) Phase I	Phase II	Phase III
40 AG*	0.44	2400	2000	1600
120 BC*	0.44	2000	1800	1500
150 C3*	0.35	2000	1000	300
150 C4*	0.35	1000	300	200
SUBGRADE	0.35	140	140	140

Pavement F

Layer	Poisson's Ratio	Elastic Moduli (MPa) Phase I	Phase II
S*	0.44	2000	1600
80 BC*	0.44	2000	1600
150 C4*	0.35	1000	300
SUBGRADE	0.35	140	140
	-	-	-

Pavement G

Layer	Poisson's Ratio	Elastic Moduli (MPa) Phase I
30 AG*	0.44	2400
150 C3*	0.35	2000
300 C4*	0.35	1000
SUBGRADE	0.35	180
	-	-

Pavement H

Layer	Poisson's Ratio	Elastic Moduli (MPa) Phase I
S1*	0.44	2000
100 C4*	0.35	2000
100 C4*	0.35	1000
SUBGRADE	0.35	140
	-	-

*** Classification according with TRH14 (CSRA, 1985)**

FIGURE 4-3: Pavement structures and material properties used for the mechanistic analysis.

Pavement E: a heavy pavement with a bituminous base, Road category A and design traffic class E4. Structure: 40 mm asphalt surfacing, 120 mm asphalt base, 150 mm C3 cemented subbase, another 159 mm C4 subbase.

Pavement F: a light pavement with a bituminous base, Road category C and design traffic class E2. Structure: 15 mm surface treatment or seal, 90 mm asphalt base, 150 mm cemented subbase.

Pavement G: a heavy pavement with a cemented base, Road category B and design traffic class E3. Structure: 30 mm asphalt surfacing, 150 mm C3 cemented base, 300 mm C4 cemented subbase.

Pavement H: a light pavement with a cemented base, Road category C and design traffic class E0. Structure: 15 mm surface treatment or seal, 100 mm C4 cemented base, 100 mm C4 cemented subbase.

All pavement structures are founded on selected layers or subgrade with assumed material properties according with road category and traffic class. Road category and design traffic class are defined in TRH4 (CSRA, 1989).

The particular structures chosen are considered a fair representation of many of the pavements found in South Africa and should allow a pavement designer to correlate many typical cases to one of the pavements analyzed and thereafter apply the findings in terms of equivalent loads.

Material properties used in the analysis of the eight selected pavement structures were assumed according to the guidelines from document RP/19/83 (Freeme, 1983), recent Heavy Vehicle Simulator (HVS) (NITRR, 1984) test results and TRH14 (CSRA, 1985). Values of elastic moduli and Poisson's ratios used to carry out ELSYM5 (Ahlborn, 1963) runs on the computer are given in Figure 4-3.

4.3.2 Load configurations

The damaging effect of single (single and dual wheeled), tandem and tridem axles of different load conditions (magnitude and intensity) were evaluated and then converted into a equivalent number of standard load configurations (E80) by means of Equivalent Damage Factors (EDF). The different group configurations considered in the theoretical phase of the study to obtain the pavement response parameters are given in Figure 4-4.

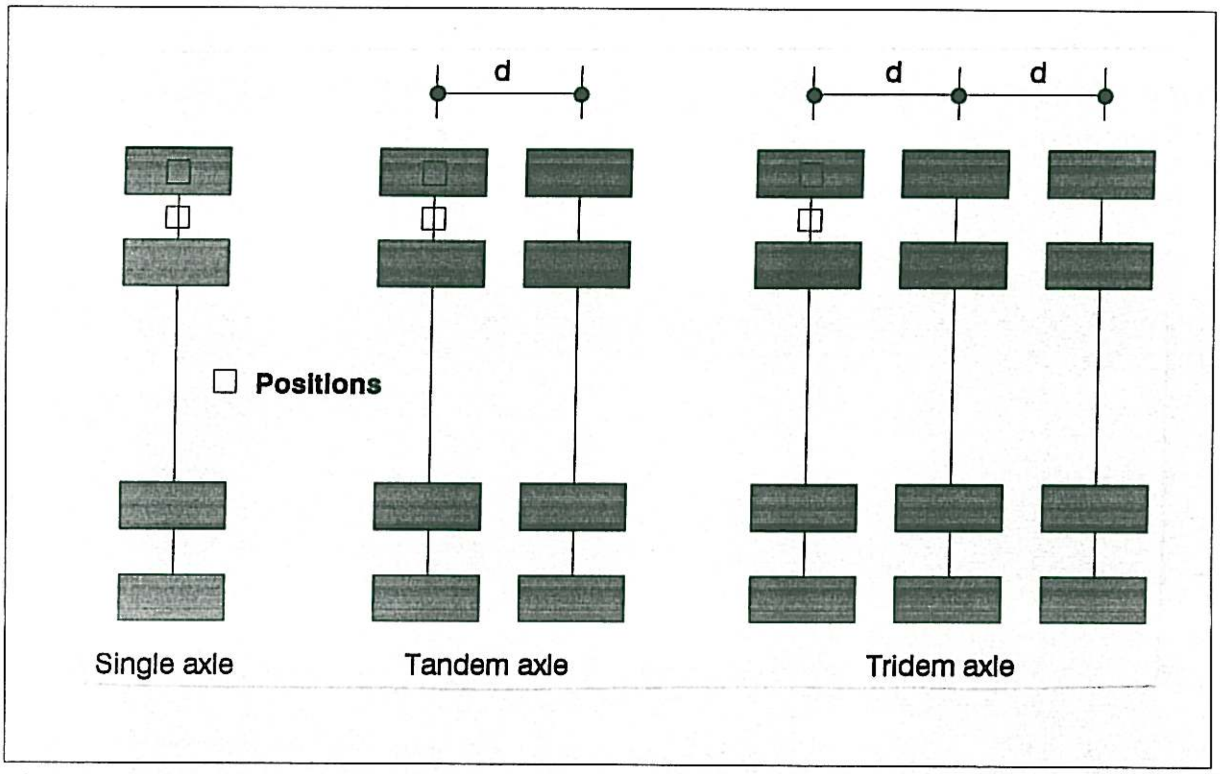


Figure 4-4: Positions investigated under different axle configurations.

Single axle with dual wheels: a single axle with dual wheels on each side was considered as the standard axle (STDA). For simplicity only one half of the axle was used in calculation. The wheels were separated by 350 mm (centre to centre) and had loads varying from 10 to 40 kN representing axle loads from 40 to 160 kN. The tyre pressure, which for the mechanistic analysis was considered equal to the contact pressure, varied from 400 kPa to 1000 kPa. However, this rage was only considered for those pavement types in which the failure mode was crushing of the cemented base. It must be borne in mind that the variation of contact pressure primarily affects the performance of the upper layers, i.e. surfacing and base.

Single axle with single wheels: a single axle with single wheels was also investigated to assess the equivalency of typical heavy vehicles. In this case the wheel load used ranged from 10 to 40 kN, representing axles from 20 to 80 kN. The tyre pressure was 520 kPa in all cases.

Tandem axle: a tandem axle is a group of two single standard axles separated by an inter-axle spacing which for the purpose of this study was varied from 0 to 5000 mm. The distance between the dual wheels was 350 mm, and the same considerations as in the case of single axle were made in respect to the tyre pressure. Even though actual values of inter-axle distances vary in a much narrower range it was decided to extend the range to 5000 mm to investigate the effects (if any). Actual values of inter-axles distance vary around 1365 mm, with typical limits ranging between 1220 and 1850 mm (Duncan, 1988). During the field study conducted on the N4-1 the measured inter-axle distances varied from 1,30 to 1,40 m (see Appendix A, Table A-3).

Tridem axle: a tridem axle is a group of three single standard axles with equal inter-axle separation between 0 and 2500 mm. The maximum inter-axle distance was 2500 mm due to limitations imposed by the computer program.

4.3.3 Calculation of Equivalent Damage Factor (EDF)

The Equivalent Damage Factor (EDF) has been defined in Section 4.1.2. (Equations 4-1 and 4-2). The procedure for calculating EDFs was divided in three steps which differed depending on the pavement type and the corresponding distress mode. The Single Wheel Factor was calculated for the first six pavements analyzed (A to F), while the Tyre Pressure Factor only needed to be considered for the last two structures (Pavements G and H).

The first step was to compare the damaging effect of a group of axles of equal load P per axle and a specific tyre pressure σ, with the damaging effect of a standard axle with the same load characteristics P and σ. This allowed the calculation of a Group Equivalency Factor (GEF). In this way the effect of inter-axle spacing on life was taken into account.

The second step dealt with the Axle Load Factor (ALF) which specifically took into account the effect of axle load. The damage produced by a standard axle of any load with a given tyre pressure was compared with a standard axle of 80 kN with the same tyre pressure. Here the damaging effect of the increase of axle load magnitude was assessed independently of other aspects.

The third step (for Pavements A to F) was to determine the Single Wheel Factor (SWF) to compare axles with single wheels to those with dual wheel configurations. At this stage, the concept of Equivalent Single Wheel Axle Load (ESWAL) was utilized.

The third step (for pavements G and H) was the assessment of the damaging effect of variations in tyre pressure and the introduction of the Tyre Pressure Factor (TPF) concept.

Finally, in order to assess the damaging effect of a normal heavy vehicle in terms of the standard load configuration the respective Equivalent Damage Factor (EDF) was calculated. The EDF of a given vehicle was calculated as the sum of the EDFs of the respective axle groups by means of the Equations 4-1 and 4-2.

It must be noted that Equations 4-1 and 4-2 change to accommodate the effect of single axles with single wheels, GEF becomes one (as for any single axle) and ALF is replaced by SWF.

4.3.4 Determination of pavement response parameters using linear-elastic theory

The selected the road sections, material properties and load configurations were the input data for a multi-layer linear elastic program (ELSYM5, (Ahlborn, 1963)) to obtain displacements, stresses and strains at selected positions within the pavement structure. The selected points for the purpose of this study were directly under the wheel and between the two wheels (Figure 4-4). These points were generally found to be the *most critical* after having used a mesh of fifteen different points to investigate different possibilities. The depths were selected according to the number and thicknesses of the layers and their constituent materials. Table 4-1 gives the suggested positions to check according to the layer and the expected failure mode.

ELSYM5 (Ahlborn, 1963) was used to investigate tandem axle configurations with inter-axle spacings varying from 0 to 5000 mm, and tridem axle configurations with inter-axle spacing from 0 to 2500 mm. To calculate pavement life, transfer functions were applied to the computed critical displacements, stresses and strains. Pavement lives were calculated for each layer and each of the inter-axle distances. The *critical layer* was selected as the one with the *minimum life*. Critical lives were then divided by the Weighting Factor (WF) to obtain the group life (N_G). After calculating group life, life for the standard axle was determined using the same loading and failure criterion ($N_{P/\alpha}$). This quasi-static procedure is valid for static and slow moving traffic loads which in-service pavements show to be the critical. For high speed traffic load a dynamic analysis is recommended (SARB, 1992a, 1992b, 1995).

The Group Equivalence Factor (GEF) was obtained by dividing the standard axle life by the respective minimum life for a group of axles at a given inter-axle distance (both the group life and the standard life are calculated at the same position using the same failure mode).

To calculate the Axle Load Factor (ALF) the programme was run again for a single axle configuration but with different axle loads ranging from 10 to 40 kN per wheel (representing a dual-wheel single axle load of 40 to 160 kN). Initially only one tyre pressure was used: 520 kPa. The critical failure mode was determined and then the allowable number of repetitions for the 80 kN standard axle was divided by the allowable number of repetitions of the standard axle of a given load. The ALF was plotted against load ratio (P/80) with a log-log scale and a linear regression was carried out. The slope of this line was defined as the damage coefficient α. For pavements with thin surfacing and cemented bases (i.e. pavements G and H) the procedure was repeated using different tyre pressures

In cases where tyre pressure variation was assumed to have a significant effect on pavement performance, the problem was assessed as follows. An 80 kN standard axle at different tyre pressures and an 80 kN standard axle with 520 kPa were compared. The ratio of the corresponding lives were defined as the Tyre Pressure Factor (TPF). Also, as in the case of ALF a coefficient (β) could be calculated by regression as the slope of the curve log of tyre pressure ratio (TPR) versus log of tyre pressure factor (TPF).

To assess the equivalency of single axles with single wheels the Single Wheel Factor (SWF) was obtained. The program was run for single-wheeled axles with different loads (20 to 80 kN) and the corresponding lives were calculated and compared with the respective standard axle life; this ratio was termed SWF. This procedure permitted the determination of Equivalent Single Wheel Axle Load (ESWAL) which represents the load of a single axle with single wheels which would cause the same damage as the standard axle.

4.4 RESULTS

The results from the mechanistic study are presented graphically for all eight pavement types (A to H) in Appendix C.

4.4.1 Granular bases

One heavy granular pavement and one light granular pavement were analyzed both under wet and dry conditions. The inclusion of the moisture condition in the analysis is due to its significant influence on granular layer performance. The effect of moisture was taken into account by changing the c term and ϕ term of the materials according to Table 4-2. The structures and the selected material properties were given in Section 4.3.1 and Figure 4-3.

Heavy pavements

Structures A and B represent the heavy pavement in dry and wet conditions respectively. The failure was predicted to occur in three successive phases: first, fatigue cracking of the lower cemented subbase; then, fatigue failure of the upper cemented subbase, and finally rutting of the last one.

The calculated life in the case of tandem axles decreases as the inter-axle spacing increases (Figures C1 and C9). This is due to the confining effect of the second axle which produces a reduction in the tensile strains. The respective Group Equivalency Factors (GEF), in turn, increase with inter-axle spacing from about 1,50 to about 1,90 for spacings of 1,0 to 5,0 metres (Figures C2 and C10). This increase indicates the benefit of using tandem axles instead of two single axles to carry the same load.

As is shown in Figures C3 and C11, for tridem axles the pavement life remains almost constant for the range of inter-axle spacings investigated (note that in this case the maximum spacing is only 2,50 metres). The GEFs do not show any specific tendency, the minimum are 2,60 for 1,00 m distance and dry conditions, and 2,25 at 2,00 m for wet conditions (Figures C4 and C12).

From the analysis of the above mentioned graph results indicate that a granular pavement designed for wet conditions has an expected life of about 50 percent less than the same pavement under dry conditions.

Figures C5 and C13 show the reduction on pavement life as the axle load increases; therefore the calculated Axle Load Factors (ALFs, Figures C6 and C14) increase with load, showing similar trend for both dry and wet conditions. The regression curves, given in Figures C7 and C15 give the obtained load damage factors "α", 1,89 and 1,79 respectively.

Light Pavements

Light granular pavements are represented by structures C and D. Failure of these pavements is predicted to begin in the cemented subbase through fatigue cracking (Phase I). Then, rutting due to densification (mainly micro-shear) occurs in the same layer if in the dry state; under wet conditions however, the granular base is predicted to fail due to its inadequate stability or low factor of safety (potential to shear failure).

There is a significant reduction in life as the axles are moved apart, both for tandem and tridem in dry or wet conditions (Figures C17, C19, C25 and C27). This is due to the initial mode of failure being effective fatigue in the cemented subbase. The close proximity of the axles reduces the horizontal tensile strains developed in the cemented material and therefore apparently improves pavement performance. The calculated GEFs, represented in Figures C18, C20, C26 and C28, indicate the advantage of using multiple rather than single axles, mainly for inter-axle spacings lesser than 3,0 metres. For a typical spacing of 1,40 m suggested values of GEF for tandem axles are 1,70 (in dry state) and 1,50 (in wet state), for tridem axles 2,15 and 2,20 (dry and wet states respectively).

As axle load increases, life decrease rapidly (Figures C21 and C29); this reduction (much more pronounced for wet conditions) is associated with an increase in Axle load Factors (ALF, Figures C22 and C30). The calculated coefficients are 3,33 and 4,94 for dry and wet conditions respectively (Figures C23 and C31); this difference reflects how granular pavements are more sensitive to overloading in the wet state.

4.4.2 Bituminous bases

Pavements chosen in this category (E and F) represent heavy and light structures with bituminous bases. The total pavement life has been defined as comprising three phases in the case of the heavy asphalt pavement and two in the case of the light structure - see Figures C33, C35, C41 and C43. For the heavy pavement, effective fatigue failure is predicted as beginning in the lower cemented subbase, then progressing through the upper cemented subbase, finally the asphalt base fails in fatigue. The cemented subbase of the light pavement is also the first layer to fail in effective fatigue and is followed by the failure of the asphalt base.

It is appreciated that subgrade moduli, as used in this study, tend to be quite high (deep structures, De Beer et el, 1988) but were taken as being fairly typical of some available test results (derived from linear elastic multi-layer theory analysis using a semi-infinite subgrade depth). However, the succession of test modes does not alter if a lower modulus is used for the subgrade, merely the magnitude of calculated life.

Heavy pavement

The calculated GEFs for a typical inter-axle distance of 1,40 m are 1,55 and 2,30 for tandem and tridem axles respectively (Figures C34 and C36). As the axle load increases four times (from 40 to 160 kN) the life decreases twelve times (Figure C37), the Axle Load Factor, in turn, increases from 0,25 to 3,50 (Figure C38). The regression analysis of these data indicates a load damage coefficient α of 1,72 (Figure C39).

Light pavement

Most of the reduction in pavement life occurs as the inter-axle spacing changes from 1,0 to 3,0 metres, thereafter on the life remains almost constant (Figure C41). The corresponding GEFs vary from 1,40 to 1,90 (Figure C42). For tridem axles the findings are similar, with GEFs varying from 1,40 to 2,40 (Figure C44). This type of pavement is very sensitive to overloading as can be seen from Figures C45 and C46, and from the value of the load damage coefficient (Figure C47).

As with other heavy and light pavements, the large difference in load damage coefficients indicates how much more sensitive light structures are to the effects of load.

4.4.3 Cemented bases

Pavements G and H represent cemented-base structures with relatively thin surfacing: a heavy cemented-base pavement (Category B) and a light cemented-base pavement (Category C), respectively. The pavement sections and the materials characterization are briefly described in Section 4.3.1 and Figure 4-3.

These examples are of particular interest because a different failure criteria was found applicable in this case, namely compression (crushing) failure (N_c). The relatively high subgrade modulus, i.e. deep structure (De Beer et al, 1988) was selected to ensure the failure mechanism was that of crushing. Lower subgrade moduli and therefore shallow structure (De Beer et al, 1988) would have resulted in fatigue failure of the subbase and would therefore have behaved similarly to some of the other pavements investigated. The latest developments in the behaviour of lightly cementitious materials (De Beer, 1990) ascribe this failure mode to cemented pavements with thin surfacing (Example G) or seal treatments (Example H). Although tyre pressure, asphalt thickness and cemented material strength play a major role in the performance of these pavements, only the effect of tyre pressure changes was investigated here.

The life of this type of pavement is extremely dependent on the tyre pressure and to a less extend on inter-axle distance (Figure C49, C59, C51 and C61). As in all the previous pavements the Group Equivalency Factors (GEF) are only slightly influenced by the axle load and the tyre pressure, as seen in Figures C50 and C60. Even though there is an appreciable variation in GEF with inter-axle spacing (Figures C50, C52, C60 and C62), values of 1,75 and 2,50 are suggested for tandem and tridem axles respectively since typical inter-axle spacings are approximately constant (about 1,40 m).

As pavement life is not only dependent on axle load but also on tyre pressure (Figure C53 and C63), Axle Load Factors (ALF) for these pavements alter with different tyre pressures (Figure C54 and C64). An equation is therefore given for the load damage coefficient α rather than a fixed value (See Figure C55 and C65). The coefficients are low because under compression failure, the magnitude of the applied load is less significant than contact pressure.

Special attention however must be paid to avoid misinterpretation. It must be understood that even though load increase does not affect this distress mode significantly, it may affect these pavements performance by changing the critical failure mode.

In order to use the Tyre Pressure Factor (TPF) in a similar approach as the ALF, a so-called pressure damage coefficient was developed: ß. This coefficient was only determined for cemented-base pavements with thin surfacing because the contact pressure was found to have a minor influence on the other modes of distress studied. The reduction in pavement life with tyre pressure is shown in Figures C56 and C66. The variation of the respective TPF with tyre pressure is given in Figures C57 and C67. After regression analysis of these curves the coefficients were calculated and are given in Figures C58 and C68.

These pavements that fail in the crushing mode (which appear to be a special case) have been incorporated in this study because they are commonly found in South Africa, especially in the Transvaal. This type of distress is expected to occur in cemented-base pavements with thin surfacing courses or in cemented subbases under bituminous bases when certain factors are present, i.e. high tyre pressures, thin asphalt layers and low cemented material strengths (i.e. Unconfined Compressive Strength (De Beer, 1992)). If the above mentioned conditions are not present, fatigue failure may possibly be the dominant mode of distress.

4.4.4 Equivalent Single Wheel Axle Load (ESWAL)

To use guidelines in various regulations and documents it is sometimes necessary to use the concept of Equivalent Single Wheel Load (ESWL) which has been the traditional way of representing a group of wheel loads as one equivalent wheel load for calculation purposes. There are a number of existing techniques and formulae that can be used to calculate values of ESWL (Yoder and Witczak, 1975; Wolff, 1992) but do not necessarily agree well with the SA Mechanistic Design approach. The ESWL concept was therefore not used in this investigation since the interaction between pavement structure and load configuration may be greatly altered if a group of wheels is substituted by a unique wheel, and therefore be misleading. However, in an attempt to quantify the damage of single axles with single wheels, a similar concept was developed: Equivalent Single Axle Load (ESWAL). The main difference with previous approaches is that the equivalency is considered in terms of pavement performance or life and therefore more suitable for the purpose of this study.

Linked to the ESWAL concept, the Single Wheel Factor (SWF) was developed. Each pavement has an associated ESWAL, for axle loads above than ESWAL the respective SWFs are greater than one, for axle loads less than ESWAL, corresponding SWFs are smaller than one.

The figures representing pavement life against single wheel axle load (C8, C16, C24, C32, C40 and C48) provide the basis for determination of ESWALs. The single wheel axle load corresponding to the same life as the determined for the standard load configuration is defined as ESWAL.

In Table 4-3 the calculated values for the different pavements are given. It should be noted that with this new approach the same load configuration has different Equivalent Single Wheel Axle Loads (ESWAL) depending on the type of pavement. Note also that light pavements show smaller values due to their sensitivity to load.

4.4.5 Determination of equivalent tandem axle load

In order to verify some of the permissible axle group loads determined in Section 3.3 with the deflection-based method (Tables 3-1 and 3-2), the present method was applied to determine equivalent tandem axle load. This tandem axle load is the load on a tandem axle which causes the same damage to the pavement that a single standard axle, i.e. a tandem axle with a Group Equivalency Factor of one (GEF = 1). The calculated values are the following:

Table 4-3: Equivalent axle loads (kN) for single axle with single wheels and tandem axle with dual wheels for Pavements A to F as calculated by mechanistic analysis.

Configuration	Pavement type						
	A	B	C	D	E	F	Avg. (Std. Dev.)
ESWAL	62	73	58	60	65	48	61 (8,3)
Tandem	124	129	132	148	126	143	134 (9,7)

For single axles with single wheels, the determined equivalent loads are similar to those calculated by using the deflection-based method (See Table 3-2). In the case of tandem axles with dual wheels, much more realistic values were determined than previously.

5. GUIDELINES FOR USING THE PROPOSED METHOD FOR THE DETERMINATION OF EQUIVALENT DAMAGE FACTORS (EDFs)

5.1 USER'S TABLES AND GRAPHS

In this section a summary of the findings from the present study are presented in the form of graphs and tables. These graphs and tables are the tools that the Engineer needs to calculate Equivalent Damage Factors (EDFs) of single axles, group of axles and/or complete heavy vehicles, in order to convert them into the equivalent number of standard 80 kN axles (E80s). These two concepts are similar to the well known Load Equivalency Factors (LEFs) and Equivalent Single Axle Load applications (ESALs) respectively (AASHTO, 1974); the difference in the notation is proposed so as to distinguish between the two different approaches and criteria used for the determination.

Figures 5-1 and 5-2 are a summary of calculated Group Equivalency Factors (GEF) for tandem and tridem axles respectively. Even though typical inter-axle spacings within a group vary in a narrow range (around 1365 mm), the spacing was extended in order to analyze the influence of axles not forming the specific group. For a given axle group (tandem or tridem), the inter-axle spacing is measured and the respective GEF is read from the curve corresponding to the type of pavement being analyzed in Figures 5-1 and 5-2 respectively. It must be borne in mind that for single axles, the corresponding GEF is one (1).

The determination of Axle Load Factors (ALF) is based on the regression analysis of the curves given in Figure 5-3, however the original curves are provided to the reader. Once the load of the individual axles of a given group is determined, the respective ALF is obtained from the curve corresponding to the type of pavement being studied. The load of the individual axles is obtained by dividing the total group load by the number of axles forming the group. This procedure which is not conservative is suggested at this stage until further research is conducted.

In the case of single axles with single wheels the approach is slightly different. The ALF is replaced by the Single Wheel Factor (SWF) and the corresponding GEF is one (1). Figure 5-4 is given to obtain the respective SWF once the axle load and the type of pavement have been determined.

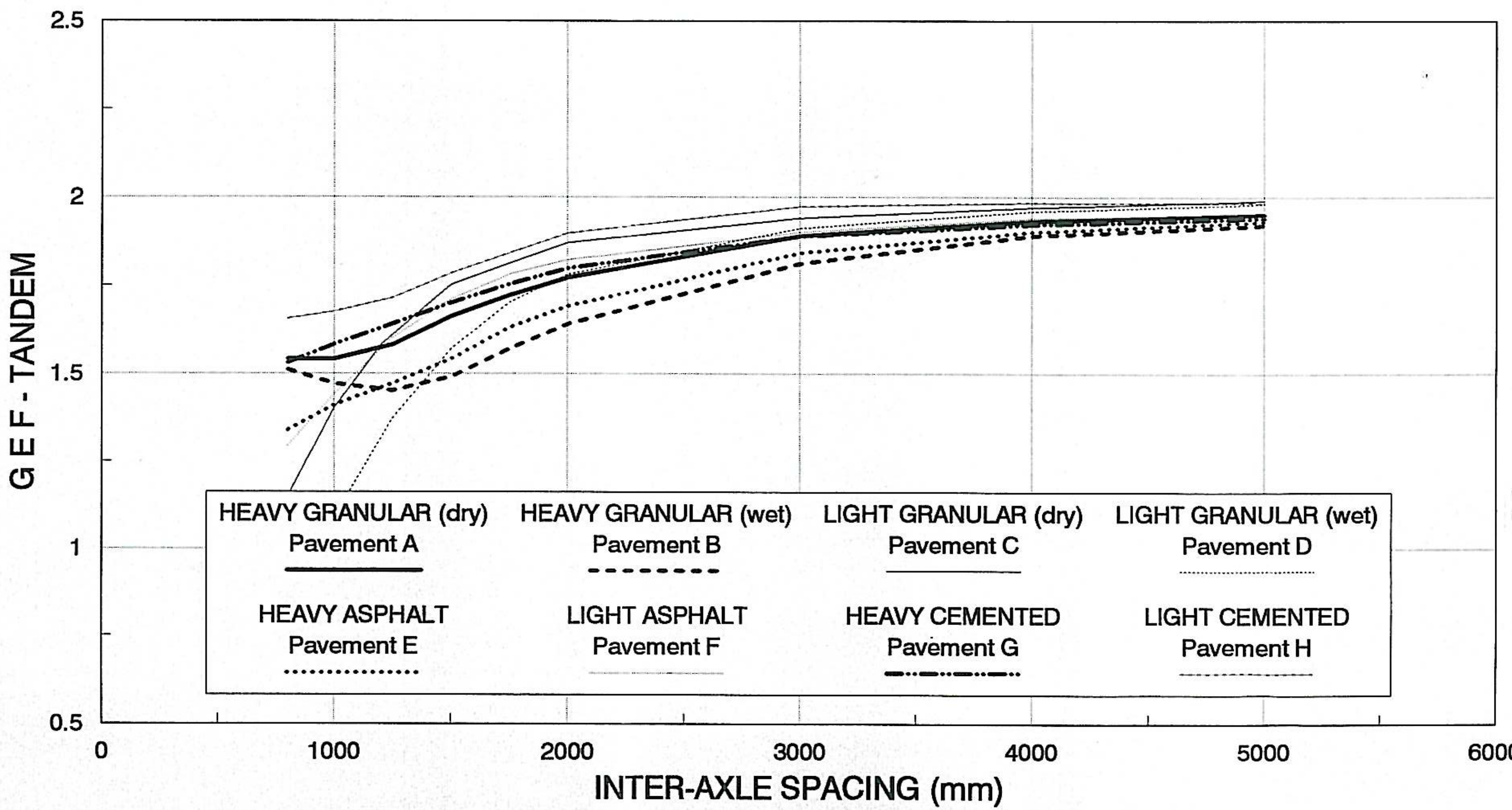


FIGURE 5-1: Comparison of Group Equivalency Factors (GEF) for different pavement types for tandem axle of different inter-axle spacing

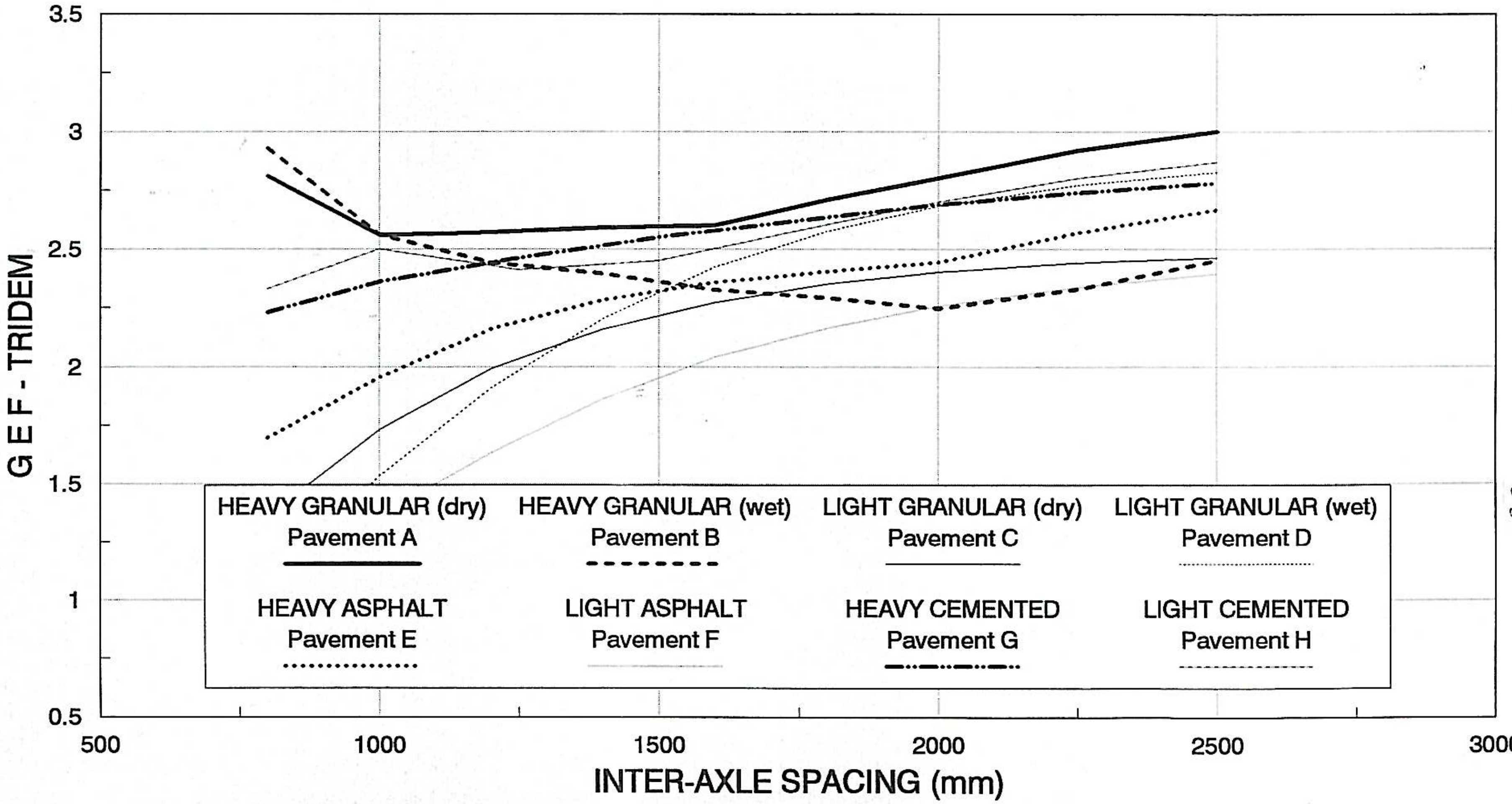


FIGURE 5-2: Comparison of Group Equivalency Factors (GEF) for different pavement types for tridem axles of different inter-axle spacing

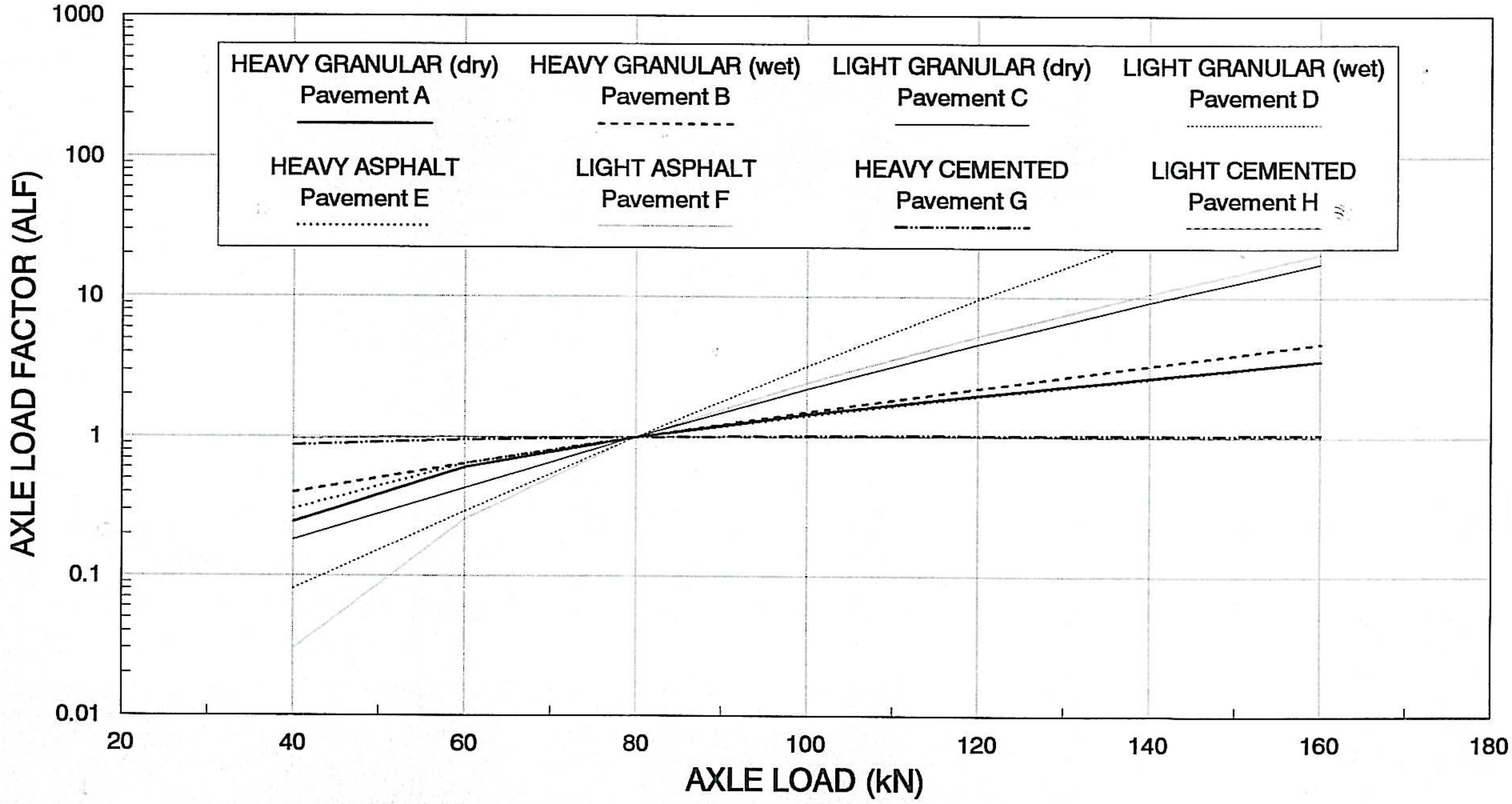


FIGURE 5-3: Curves for the determination of Axle Load Factors (ALFs) for the different pavement types analyzed

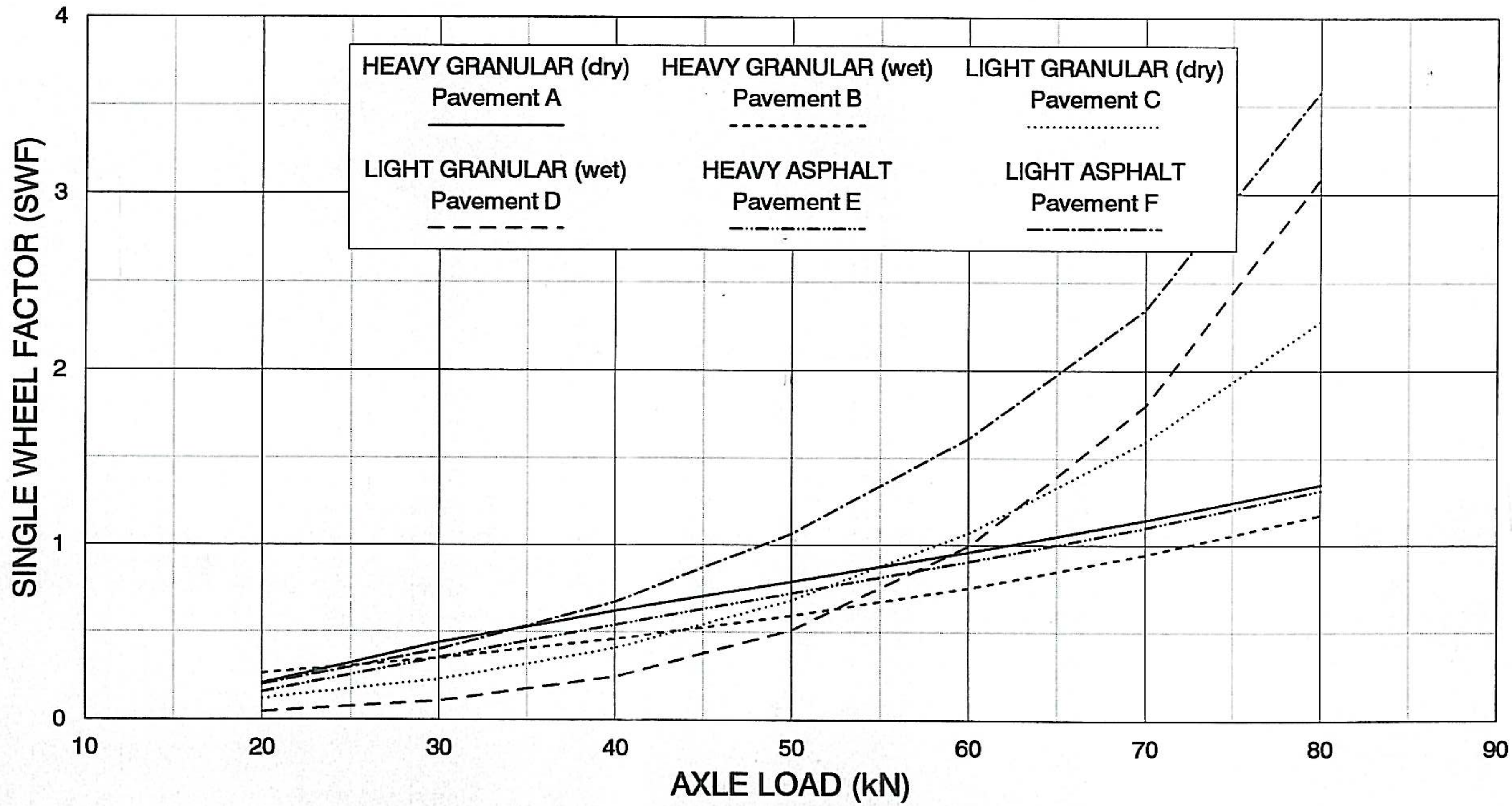


FIGURE 5-4: Curves for the determination of Single Wheel Factors (SWFs) for single axles with single wheels

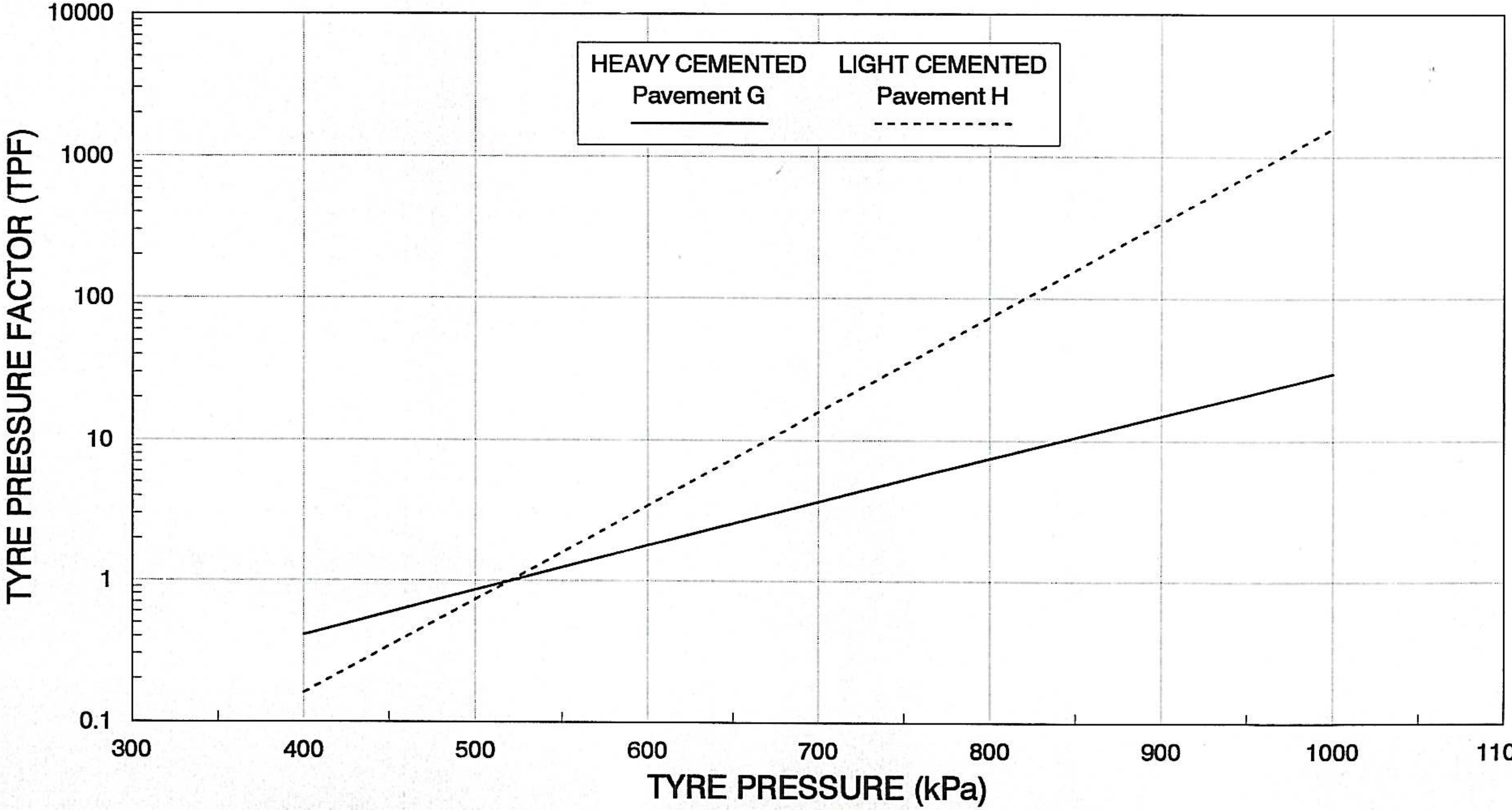


FIGURE 5-5: Curves for the determination of Tyre Pressure Factors (TPFs) for heavy and light cemented-treated base pavements

The Tyre Pressure Factor (TPF) concept was developed in order to evaluate the influence of tyre pressure in pavement life. At this stage it was only determined for cemented base pavements with thin surfacing where the pressure is known to have significant influence in pavement performance (life). Figure 5-5 shows the curves obtained for the heavy and light structures analyzed. Once the tyre pressure and the pavement type have been determined the respective TPF can be obtained from the mentioned figure.

Table 5-1 is provided as an alternative tool to the above described figures. This table contains a summary of suggested GEF for tandem and tridem axles, as well as the load and pressure damage coefficients calculated by regression analysis. These two coefficient facilitate the determination of Axle Load Factors (ALF) and Tyre Pressure Factors (TPF) respectively, just by applying the formulae provided at the bottom of the table.

Table 5-1: Summary of partial factors and damage coefficients

PAVEMENT TYPE	ROAD CATEGORY (*)	TRAFFIC CLASS (*)	GEF (**)		$\alpha^{(1)}$	$\beta^{(2)}$
			Tandem	Tridem		
Heavy granular (dry)	A	E4	1,65	2,65	1,89	-
Heavy granular (wet)	A	E4	1,50	2,40	1,79	-
Light granular (dry)	C	E1	1,70	2,15	3,33	-
Light granular (wet)	C	E1	1,50	2,20	4,94	-
Heavy bituminous	A	E4	1,55	2,30	1,72	-
Light bituminous	C	E2	1,70	1,90	4,63	-
Heavy cemented	B	E3	1,70	2,50	0,12+0,47 (TPR-1)$^{(3)}$	4,8
Light cemented	C	E0	1,75	2,45	0,04+0,18 (TPR-1)$^{(3)}$	10,0

* Road Category and Design Traffic Class classification according with TRH4 (CSRA, 1989).

** Values for typical inter-axle spacing (1,40 m). For other spacings consult Figures 5-1 and 5-2.

(1) $ALF = (P/80)^{\alpha}$, with P: axle load in kN.

(2) $TPF = (\sigma/520)^{\beta}$, with σ: tyre pressure in kPa.

(3) TPR = Tyre Pressure Ratio = $\sigma/520$, with σ: tyre pressure in kPa.

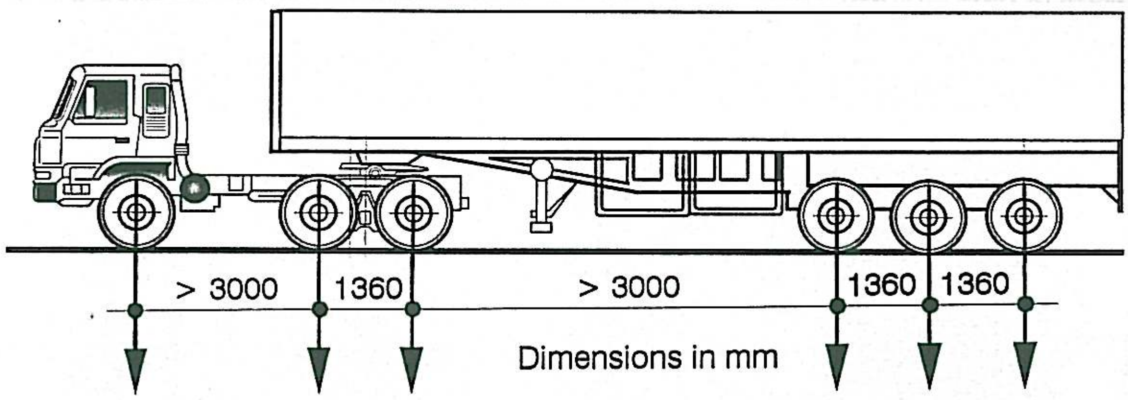
5.2 WORKED EXAMPLE

To illustrate the proposed method for the determination of Equivalent Damage Factors (EDFs) and the use of the figures and tables provided in this section, an example of a heavy vehicle is given here. The selected vehicle axle configuration represents the mode from vehicle weighbridge statistics obtained during 1992 (CSIR, 1992). Three different load configuration are analyzed so as to represent:

(a) the permissible mass per axle, i.e. legal axle load limits currently used in South Africa for the respective group type (single axle with single wheels, tandem axle with dual wheels and tridem axle with dual wheels,

(b) the average overloaded sample as from the above mentioned statistics, and

(c) the maximum overloaded sample from the same source.



Axle Group 1 (Single Axle)	Axle Group 2 (Tandem Axle)	Load Configuration (kg)	Axle Group 3 (Tridem Axle)
6000	16400	Permissible load	21000
6473	18000	Average overload	22880
6473	28347	Maximum overload	47880

Figure 5-6: Vehicle axle and load configuration as used for the worked example.

Figure 5-6 shows the axle configuration of such vehicle and the three selected load configurations. The load configuration analyzed are also given in Table 5-2.

Table 5-2: Axle mass (kg) of the different load configurations

Load configuration	Single Axle	Tandem Axle	Tridem Axle
(a) Permissible load	6000	16400	21000
(b) Average overload	6473	18000	22880
(c) Maximum overload	6473	28347	47880

The following four pavement types are considered in this worked example:

(i) Pavement A: heavy granular in dry conditions,

(ii) Pavement C: light granular in dry conditions,

(iii) Pavement G: heavy cemented, and

(iv) Pavement H: light cemented.

As no data was available on the tyre pressure of the vehicles, two different tyre pressures were used: 520 kPa for load configuration (a) and 700 kPa for load configurations (b) and (c). The Equivalent Damage Factor for each load configuration was determined by using the following equations:

$$EDF_v = \sum_{1}^{i=3} EDF_i = \sum_{1}^{i=3} GEF_i \times ALF_i \times TPF_i \qquad \text{... Eq. (5-1)}$$

where $i=3=n$ represent the number of axle groups.

Axle Group 1 is the front single axle with single wheels, Axle Group 2 is the middle tandem axle with dual wheels and Axle Group 3 is the rear tridem axle with dual wheels. As for single axles with single wheels GEF becomes one (1) and ALF is replaced by SWF, then:

$$EDF_v = (SWF_1 \times TPF_1) + (GEF_2 \times ALF_2 \times TPF_2) + (GEF_3 \times ALF_3 \times TPF_3) \qquad \text{... Eq. (5-2)}$$

5.2.1 Single Wheel Factors (SWF)

The corresponding axle masses for the three load configurations are converted into axle load (kN) and the respective SWFs are obtained from Figure 5-4. For examples G and H as the load magnitude has minor influence on the final EDF, no SWF were calculated, and a value of one (1) is suggested until more detailed study is done.

5.2.2 Tyre Pressure Factors (TPF)

TPFs are only considered for pavements G and H, for other types a value of one (1) must be adopted. The tyre pressure is assumed to be the same for all the axle groups of each load configuration, therefore: $TPF_1 = TPF_2 = TPF_3$. For load configuration (a) as the tyre pressure is the standard (520 kPa) therefore TPF is one (Figure 5-5). From the same figure, and for a tyre pressure of 700 kPa the TPF for the two pavements are determined from the respective curves.

5.2.3 Group Equivalency Factors (GEF)

As the inter-axle spacing is 1365 mm, the respective GEFs for the tandem and tridem axles can be taken from Table 5-1 (for other spacings Figures 5-1 and 5-2 must be used).

5.2.4 Axle Load Factors (ALF)

The load of the individual axles forming the groups is determined by dividing the group load by two or three for tandem and tridem axles respectively. Then, by using Figure 5-3, the ALFs are obtained. ALFs can also be obtained by applying the load damage coefficients given in Table 5-1 to the formulae given in the same table. Notice that for the tandem axle with the permissible load (load configuration (a)) the ALF is one as this was the utilized reference load.

In Table 5-2 a summary of the calculated Equivalent Damage Factors (EDFs) as well as all the partial factors (SWF, TPF, GEF, ALF) determined for all the cases are shown.

Table 5-3: Summary of calculated factors for the worked example.

Pavement type	SWF_1	TPF_2	GEF_2	ALF_2	TPF_2	GEF_3	ALF_3	TPF_3	EDF_v
PERMISSIBLE LOAD									
A	0.93	1.00	1.65	1.00	1.00	2.65	0.75	1.00	4.57
C	1.03	1.00	1.70	1.00	1.00	2.15	0.60	1.00	4.02
G	1.00	1.00	1.70	1.00	1.00	2.50	0.98	1.00	5.15
H	1.00	1.00	1.75	1.00	1.00	2.45	0.99	1.00	5.18
AVERAGE OVERLOAD									
A	1.03	1.00	1.65	1.20	1.00	2.65	0.88	1.00	5.34
C	1.25	1.00	1.70	1.39	1.00	2.15	0.80	1.00	5.33
G	1.00	3.80	1.70	1.03	3.80	2.50	0.98	3.80	19.8
H	1.00	18.0	1.75	1.01	18.0	2.45	0.99	18.0	93.4
MAXIMUM OVERLOAD									
A	1.03	1.00	1.65	2.84	1.00	2.65	3.56	1.00	15.2
C	1.25	1.00	1.70	6.30	1.00	2.15	9.36	1.00	32.1
G	1.00	3.80	1.70	1.17	3.80	2.50	1.21	3.80	22.9
H	1.00	18.0	1.75	1.06	18.0	2.45	1.07	18.0	98.6

The calculated EDF_v for a given vehicle and a given pavement structure, when multiplied by the number of vehicles of the same type, results in the number of 80 kN single axle load applications (E80s) which will have an equivalent effect on the performance of that pavement.

6. CONCLUSIONS

In Sections 2 and 3 an overview of the existing methods to derive Load Equivalency Factors (LEFs) is presented together with the application of a pavement response-based method. Due to the identified limitations of the investigated methods and, as no suitable method for local conditions was found, a new approach for the determination of LEFs was developed by using mechanistic analysis. Some of the identified limitations are assessed with this investigation. However, calculations do not take the dynamic of the pavement response into account and therefore, the new approach is applicable to static and slow moving wheel loads.

The study was confined to eight pavement types which are considered to be quite representative of many South African pavements. Three typical different types of axle groups with dual wheels were used in the analysis: single, tandem and tridem. In addition the effect of single axles with single wheels was investigated, as well as the effect of load and tyre pressure. During the study new terms were developed to describe different aspects of pavement damage, namely:

Group Equivalency Factor	GEF	:	which takes into account the effects of inter-axle spacing;
Axle Load Factor	ALF	:	describing the effect of axle load magnitude;
Tyre Pressure Factor	TPF	:	which is used to assess the tyre pressure effect; and
Single Wheel Factor	SWF	:	used to compare the effects of single and dual wheels.

The above terms, known in this study as "partial factors", are combined together to obtain the **Equivalent Damage Factor** (EDF) by using the following expression:

$$EDF = GEF \times ALF \times TPF$$

For single axles with single wheels, as GEF is one and ALF is replaced by SWF, the expression changes to:

$$EDF = SWF \times TPF$$

By applying these two basic equations, the EDF of a given vehicle is worked out as the sum of the EDFs of all its axle groups. Once the EDF of a vehicle within a load group is determined, the number of equivalent 80 kN single-axle loads (E80s) can be obtained by multiplying the number of vehicles in the same load group by the relevant EDF.

6.1 TANDEM AXLES

For inter-axle spacing greater than 3,0 m each axle may be assumed to affect pavement performance without influence from the other axle, within a tolerance of 10% (i.e. GEF $\geq$ 1,8). For a tolerance of 5%, the distance is approximately 4,0 m.

The interaction on stresses and strains between axles is always greater for light structures.

For a typical inter-axle spacing of 1365 mm the GEF varies from 1,50 to 1,75. Therefore there is a beneficial effect if a tandem axle is used rather than two single axles with the same load. Probably for other failure modes than the adopted in the present study, this statement does not hold true.

6.2 TRIDEM AXLES

The variation in GEF for tridem axles is less well-defined than for tandem axles; this is due to differences in pavement-load configuration interaction, as presently defined in the South African Mechanistic Design Procedure.

The axle interaction is greater than in the case of tandem axles, and due to limitations on the software it is difficult to establish the limit between group and independent axles. Therefore a limit can only be assumed to be somehow greater than for tandem, i.e. 4,0 m.

For typical inter-axle spacings used in practice GEF for tridem axle groups varies between 1,90 and 2,65 for the different pavement types investigated.

6.3 AXLE LOAD FACTOR (ALF)

Values of ALF indicate that light pavements are more sensitive to changes in wheel load than heavy pavements. ALF is determined using the formula (*P*: axle load in kN):

$$ALF = \left(\frac{P}{80}\right)^{\alpha}$$

Values of α range from 1,72 for a heavy asphalt pavement to 4,94 for a light granular pavement.

For cemented pavements, mechanistic analysis indicated the crushing mode of failure to be critical, therefore the coefficient α is extremely low (0,12 and 0,04) because wheel load has an insignificant influence on performance as opposed to tyre pressure. However, is must be borne in mind that an important load increase may affect this coefficient by changing the selected distress mode.

6.4 TYRE PRESSURE FACTOR (TPF)

As in the case of load factors, the lighter the pavement structure the more sensitive to tyre pressure variation. This is reflected in the higher coefficient ß found for the light cemented pavement:

$$TPF = \left(\frac{\sigma}{520}\right)^{\beta}$$

with β = 4,8 and 10 for heavy and light pavements respectively (σ: tyre pressure in kPa).

6.5 SINGLE AXLES WITH SINGLE WHEELS

The concepts of Equivalent Single Wheel Axle Load (ESWAL) and Single Wheel Factor (SWF) have been developed in order to quantify the damage of single axles with single wheels. Notice that there is a difference with the traditional Equivalent Single Wheel Load (ESWL) which represents the isolated single-wheel load which produces an equivalent effect to that produced by all the wheels in a group. In ESWAL, the "equivalency" is based upon the pavement performance via the transfer function and therefore considered as a more advanced approach.

7. RECOMMENDATIONS FOR FURTHER RESEARCH

(a) The Equivalent Damage Factor (EDF) was defined as a multiple factor comprising three partial factors, namely, GEF, ALF and TPF. In this way other aspects influencing EDF, such as those from Table 2-5, may be assessed in further research and incorporated into the expression by adding new factors. Aspects such as wheel spacing within an axle and the definitions of distress modes and terminal levels are recognized of primary importance.

(b) It is of fundamental importance to upgrade the existing full scale testing facility (Heavy Vehicle Simulator (HVS)) so as to accommodate tandem and tridem axles; specially now since the renewed world wide interested in full scale accelerated testing is increasing.

(c) Laboratory research into dynamic testing should be carried out to investigate the performance of road materials under loading cycles such as those produced by multiple axle arrangements.

(d) The proposed method of calculating GEF in this study is based on maximum calculated surface deflections as suggested after the literature survey (Section 1). However it is well known that peak surface deflections do not always represent the adequate response in terms of pavement life. The suggestion is made to investigate failure criteria individually and define contribution factors in terms of the specific pavement response, i.e. stress or strains.

(e) The updating of the South African Mechanistic Design Method is by now a necessity. The main identified deficiency is in terms of "transfer functions". The incorporation of the latest work carried in this regard and the use of the existing knowledge in the DRTT HVS data base to develop new functions is suggested.

(f) Investigations should be carried out to relate in-situ deflection bowl parameters to multi-axle load configuration effects.

(g) The nonlinear dynamic analysis of road pavements should be incorporated into the South African Mechanistic Design method as soon as local material characterization is available.

8. REFERENCES

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO), AASHTO interim guide for design of pavement structures 1972, Washington, D.C., 1974.

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO), AASHTO guide for design of pavements structures 1986, Washington, D.C., 1986.

AHLBERG, H. L. and BARENBERG, E. J., The University of Illinois Pavement Test Track -A tool for evaluating highway pavements, Highway Research Record Number 13, Flexible Pavement Design, Highway Research Board, National Academy of Sciences - National Research Council, Publication 1110, Washington, D.C., 1963.

AHLBORN, G., ELSYM5, computer program for determining stresses and strains in a multiple-layer asphalt pavement system, Internal Report (unpublished), CHEVRON Research Corporation, Richmond, California, 1963.

BARKER, W. R., BRABSTON, W. N. and CHOU, Y. T., A general system for the structural design of flexible pavements, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, Session III, pp. 188-206.

BARTELSMEYER, R. R. and FINNEY, E. A., Use of AASHO Road Test findings by the AASHO Committee on Highway Transport, Special Report 73, The AASHO Road Test, proceedings of a conference held May 16-18, 1962, St. Louis, Mo., pp. 415-438, Highway Research Board, National Academy of Sciences - National Research Council (NAS-NRC), publication No 1012, Washington, D.C., 1962.

BENKELMAN, A. C. and CAREY, Jnr. W. N., Prediction of flexible pavement performance from deflection measurements, Special Report 73, The AASHO Road Test, proceedings of a conference held May 16-18, 1962, St. Louis, Mo., pp. 224-226, Highway Research Board, National Academy of Sciences - National Research Council (NAS-NRC), publication No 1012, Washington, D.C., 1962.

BENKELMAN, A. C., KINGHAM, R. I. and FANG, H. Y., Special deflection studies on flexible pavements, Special Report 73, The AASHO Road Test, proceedings of a conference held May 16-18, 1962, St. Louis, Mo., pp. 102-125, Highway Research Board, National Academy of Sciences - National Research Council (NAS-NRC), publication No 1012, Washington, D.C., 1962.

BONAQUIST, R., SURDAHL, R. and MOGAWER, W., Effect of tire pressure on flexible pavement response and performance, Transportation Research Record 1227, Rigid and Flexible Pavement Design and Analysis, Transportation Research Board, National Research Council, Washington, D.C. 1989, pp. 97-106.

BOSMAN, J. and VILJOEN, A. W., Load equivalency factors for heavy vehicles. Research Report RR455, reprint from the proceedings of the Annual Transport Convention (ATC 1986), Vol 2B, Paper 2B/VII, Pretoria, August 1986.

BOUSSINESQ, V. J., Application des potentiels à l'étude de l'équilibre, et du mouvement des solides élastiques avec des notes etendues sur divers points de physique mathématique et d'analyse. Gauthier-Villars, Paris, France, 1885. (Discussion on theory adopted from: Soil Mechanics for Road Engineers, pp. 413 - 416, Transport and Road Research Laboratory (TRRL), London, UK, 1974).

BRABSTON, W. N., BARKER, W. R. and HARVEY, G. G., Development of a structural design procedure for all-bituminous concrete pavements for military roads, Technical Report S-75-10, Soils and Pavement Laboratory, U. S. Army Engineer Waterways Experimental Station, Vicksburg, 1975.

BROWN, D. N., and AHLVIN, R. G., Revised method of thickness design for flexible pavements at military installations, U.S. Army Engineers, Waterways Experiment Station Technical Report 3-582, 1961.

BROWN, S. F. and BELL, C. A., The validity of design procedures for the permanent deformation of asphalt pavements, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, pp. 467-508.

BROWN, S. F., PELL, P. M., and STOCK, A. F., The application of simplified fundamental design procedures for flexible pavements, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, pp. 327-341.

CAREY JNR., W. N. and IRICK, P. E., The pavement serviceability-performance concept, Pavement Performance Concepts, Highway Research Board (HRB) Bull. 250, proceedings of the 39th meeting of the HRB, 1960.

CARMICHAEL, R. F., EIKELBOOM, J., ELSENAAR, P. M. W. and HUDSON, W. R., Selection of pavement models in a rational pavement system to be used by the Dutch State Road Laboratory, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, pp. 188-206.

CÉLARD, B., Esso road design technology, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, August 1977, pp. 249-268.

CHRISTISON, J. T., Volume 9: Pavements response to heavy vehicle test program: Part 2 - Load Equivalency Factors, Technical Steering Committee of the Vehicle Weights and Dimensions Study, Canroad Transportation Research Corporation, Ottawa, Canada, 1986.

CLAESSEN, A. I. M., EDWARDS, J. M., SOMMER, P. and UGE, P., Asphalt pavement design - the Shell Method, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, pp. 39-74.

COMMITTEE OF STATE ROAD AUTHORITIES (CSRA), TRH 4: 1985, Structural design of interurban and rural road pavements, Pretoria, Department of Transport, 1989.

COMMITTEE OF STATE ROAD AUTHORITIES (CSRA), Draft TRH 12: 1991. Flexible pavement rehabilitation investigation and design, Pretoria, Department of Transport, 1991.

COMMITTEE OF STATE ROAD AUTHORITIES (CSRA), TRH 14: 1985. guidelines for roads construction materials, Pretoria, Department of Transport, 1985.

COUNCIL FOR SCIENTIFIC AND INDUSTRIAL RESEARCH (CSIR), Annual Statistics 1992: Vehicle Overloading Management System, Division of Roads and Transport Technology (DRTT), CSIR, Pretoria, 1992.

CURTAYNE, P. C. and WALKER, R. N., A fresh look at the interpretation of Pavement Serviceability and an experiment to measure riding quality of roads in South Africa, reprint from the 3rd International Conference on the Design of Asphalt Pavements, London, 1972, Research Report RR144, NITRR, CSIR, Pretoria, 1972.

DEACON, J. A., Load equivalency in flexible pavements, proceedings of the Association of Asphalt Paving Technologists (AAPT), Volume 38, Technical Sessions held (February 10-12) at Los Angeles, California, 1969, pp. 465-494.

DE BEER, M. (1989a), Resilient response of pavements with lightly cementitious layers. Research Report DPVT 38, DRTT, CSIR, Pretoria, April 1989.

DE BEER, M. (1989b), Aspects of the design and behaviour of road structures incorporating lightly cementitious layers, Ph.D. Thesis, University of Pretoria, Pretoria, 1989.

DE BEER, M., Developments in the failure criteria of the South African mechanistic design procedure for asphalt pavements, paper presented at the 7th International Conference on Structural Design of Asphalt Pavements, Royal Centre, Nottingham, United Kingdom, 17-20 August 1992.

DE BEER, M., KLEYN, E. G. and SAVAGE, P. F., Towards a classification system for the strength-balance of thinly surfaced flexible pavements, proceedings of the Annual Transportation Convention, Volume 3D, Paper 3D-4, Pretoria, July 1988.

DORMAN, G. M. and METCALF, C. T., Design curves for flexible pavements based on layered system theory, Highway Research Record 71, Highway Research Board, Division of Engineering and Industrial Research National Academy of Sciences, National Research Council, Washington, D. C., 1965.

DUNCAN, M. A. G., A procedure for the selection of critical effects and spans to be used for the assessment of abnormal load vehicles on road bridges, Research Report DPVT/20, DRTT, CSIR, Pretoria, September 1988.

ELLIOT, R. P. and LOURDESNATHAN, D., Improved characterization model for granular bases, Transportation Research Record 1227, Rigid and Flexible Pavement Design and Analysis, Transportation Research Board, National Research Council, Washington, D.C., 1989, pp. 128-133.

FINN, F., SARAF, C., KULKARNI, R., SMITH, W. and ABDULLAH, A., The use of distress prediction subsystems for the design of pavement structures, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, pp. 3-38.

FREEME, C. R., The behaviour of bituminous surfacing in asphalt pavements, Ph.D. Thesis, University of Natal, Durban, 1971.

FREEME, C. R. and MARAIS, C. P., Thin bituminous surfaces: their fatigue behaviour and prediction, Special Report 140, Highway Research Board, Division of Engineering and Industrial Research, National Academy of Sciences, National Research Council, Washington, D.C., 1973.

FREEME, C. R., Evaluation of pavement behaviour for major rehabilitation of roads. Research Report RP/19/83, DRTT, CSIR, Pretoria, September 1983.

FREEME, C. R. and DE BEER, M., Notes for R & I course: Pavement behaviour, Technical Note TN/14/87, NITRR, CSIR, Pretoria, June 1987.

FREEME, C. R. and SERVAS, V. P., Advances in pavement design and rehabilitation, proceedings of the Annual Transport Convention, S.350, Session: Transport Infrastructure; Accelerated Testing of Pavements, CSIR, Pretoria, 29 July - 2 August 1985.

GEIPOT, Research on the interrelationships between costs of highway construction, maintenance and utilization (PICR), Final Report, 12 Volumes, Empresa Brasileira de Planejamento de Transportes (GEIPOT), Ministry of Transport, Brasilia, 1982.

GOVIND, S. and WALTON, C. M., Fatigue model to assess pavement damage, Transportation Research Record 1227, Rigid and Flexible Pavement Design and Analysis, Transportation Research Board, National Research Council, Washington, D.C., 1989, pp. 88 - 96.

GRIVAS, D. A., Risk analysis of Accelerated Testing, proceedings of the Annual Transport Convention, S.350, Session: Transport Infrastructure; Accelerated Testing of Pavements, CSIR, Pretoria, 29 July - 2 August 1985.

HAJEK, J. J. and AGARWAL, A. C., Influence of axle group spacing on pavement damage, PAV-89-06, Research and Development Branch, Ontario Ministry of Transportation, Canada, 1989.

HIGHWAY RESEARCH BOARD (HRB, 1962a), The AASHO Road Test, Report 5, Pavement Research, Highway Research Board, Special Report 61E, Publication 954, HRB, NAS-NRC, Washington, D.C., 1962.

HIGHWAY RESEARCH BOARD (HRB, 1962b), The AASHO Road Test, Report 6, Pavement Research, Highway Research Board, Special Report 61F, Publication 955, HRB, NAS-NRC, Washington, D.C., 1962.

HIGHWAY RESEARCH BOARD (HRB, 1962c), The AASHO Road Test, Highway Research Board, Special Report No 61A to 61F and Publications 950 - 955, HRB, NAS-NRC, Washington, D.C., 1962.

HUDSON, W. R. and IRICK, P., The place of full scale road tests in Accelerated Pavement Testing, proceedings of the Annual Transport Convention, S.350, Session: Transport Infrastructure; Accelerated Testing of Pavements, CSIR, Pretoria, 29 July - 2 August 1985.

HUGO, F., Some factors affecting the design and use of the Texas Mobile Load Simulator, Vehicle-Road Interaction, ASTM STP 1225, B. T. Kulakowski, Ed., American Society for Testing and Materials, Philadelphia, 1994, pp. 67-88.

HVEEM, F. N. and SHERMAN, G. B., Thickness of flexible pavements by the California formula compared to AASHO Road Test data, Highway Research Record Number 13, Flexible Pavement Design, Highway Research Board, National Academy of Sciences - National Research Council, Publication 1110, Washington, D.C., 1963, pp. 142-167.

IRICK, P. E. and ARE INC-ENGINEERING CONSULTANTS, Characteristics of load equivalence relationships associated with pavement distress and performance, Final Report Trucking Research Institute, American Trucking Association, Austin, Texas, October 1989.

IRICK, P. E. and HUDSON, W. R., Guidelines for satellite studies of pavement performance, NCHRP Report 2A, Highway Research Board, National Academy of Sciences - National Research Council, 1964.

JORDAAN, G. J., Analysis and development of some pavement rehabilitation design methods, Ph.D. Thesis, University of Pretoria, 1988.

JUNG, F. W. and PHANG, W. A., Elastic layer analysis related to performance in flexible pavement design, RR191, Engineering Research and Development Branch, Ministry of Transportation and Communications, Ontario, Canada, 1974.

KEKWICK, S. V. (1984a), Accuracy of relative damage values from HVS testing. Technical Note TP/163/84, NITRR, Pretoria, December 1984.

KEKWICK, S. V. (1984b), The relative damage power coefficient (n) of a concrete pavement, Technical Note TP/169/84, NITRR, Pretoria, December 1984.

KEKWICK, S. V., Derivation of pavement relative damage coefficients from Heavy Vehicle Simulator testing: methods and accuracy, Research Report RP/17, NITRR, Pretoria, 1985.

KENNEDY C. K. and LISTER N. W., Prediction of pavement performance and the design of overlays, TRRL Laboratory Report 833, TRRL, Crowthorne, 1978.

KENNIS, W. J., A design method for flexible pavements using the VESYS structural subsystem, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, Session II, pp. 101-130.

KIRWIN, R. W., SNAITH, M. S. and GLYNN, T. E., A computer based subsystem for the prediction of pavement deformation, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, pp. 509-518.

KLEYN, E. G., FREEME, C. R. and TERBLANCHE, L. J., The impact of Heavy Vehicle Simulator testing in the Transvaal, proceedings of the Annual Transport Convention, S.350, Session: Transport Infrastructure; Accelerated Testing of Pavements, CSIR, Pretoria, 29 July - 2 August 1985.

KLEYN, E. G. and SAVAGE, P. F., The application of the pavement DCP to determine bearing properties and performance of road pavements, International Symposium on Bearing Capacity of Roads and Airfields, Trondheim, 1982.

LANGSNER, G., HUFF, T. S. and LIDDLE, W. J., Use of road test findings by AASHO Design Committee, Special Report 73, The AASHO Road Test, Proceedings of a Conference held May 16-18, 1962, St. Louis, Mo., pp. 399-414, Highway Research Board, National Academy of Sciences - National Research Council, publication No 1012, Washington, D.C., 1962.

LISTER, N. W. AND KENNEDY, C. K., A system for the prediction of pavement life and design of pavement strengthening, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, Session VI, pp. 629-648.

MAREE, J. H., Ontwerpparameters vir klipslag in plaveisels, M.Sc. Thesis, University of Pretoria, Pretoria, 1978.

MAREE, J. H., Aspekte van die ontwerp en gedrag van padplaveisels met korrelmateriaal-kroonlae, Ph.D. Thesis, University of Pretoria, Pretoria, 1983.

MITCHELL, J. K. and MONISMITH, C. L., A thickness design procedure for pavements with cement stabilized bases and thin asphalt surfacing, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, Session VI, pp. 409-416.

NATIONAL INSTITUTE FOR TRANSPORT AND ROAD RESEARCH (NITRR), Symposium on recent findings of Heavy Vehicle Simulator Testing, Annual Transportation Convention, Council for Scientific and Industrial Research (CSIR), Pretoria, 1984.

NIJBOER, L. W. and VAN DER POEL, C., A study of vibration phenomena in asphaltic road constructions, proceedings of the Association of Asphalt Paving Technologists (AAPT), Volume 22, 1953.

OECD FULL-SCALE PAVEMENT TEST, Organisation for Economic Co-operation and Development (OECD), Road Transport Research, Report prepared by an OECD scientific expert group, Paris, France, 1991.

OTTE, E., The applicability of the AASHO Road Test results to pavements in South Africa, Report RP/2/72, DRTT, CSIR, Pretoria, 1972.

OTTE, E., A structural design procedure for cemented-treated layers in pavements, D.Sc. (Eng) Thesis, University of Pretoria, Pretoria, 1978.

PATERSON, W. D. O., and MAREE, J. H., An interim mechanistic design method, NITRR Technical Report RP/5/78, CSIR, Pretoria, April 1978.

PATERSON, W. D. O., Road deterioration and maintenance effects : models for planning and management, The Highway Design and Maintenance Standards Series, Volume III, The International Bank for Reconstruction and Development, The World Bank, Chapter IV and IX, Washington, D.C., 1987.

PRAKASH, A. and AGARWAL, A., tri-axle study: proposed methodology to study effect on pavements, Draft report, 1988.

QUEIROZ, C., CARAPETIS, S., GRACE, H. and PATERSON, W., Observed behaviour of bituminous-surfaced low-volume laterite pavements, proceedings of the Fifth International Conference on Low-Volume Roads, May 19-23, 1991, Raleigh, North Carolina, Transportation Research Record 1291, Volume II, Transportation Research Board, National Research Council, Washington, D.C., 1991, pp. 126-136.

QUEIROZ, C., VISSER, A. T. and MOSER, B. K., Load, temperature and season effects on pavement deflections, Technical Publication No. 691, Brazilian Road Research Institute (IPR), Rio de Janeiro, Brazil, 1982.

RILETT, L. R. and HUTCHINSON, B. G., LEF estimation from Canroad pavement load-deflection data, Transportation Research Record 1196, Pavement Evaluation and Rehabilitation, Transportation Research Board, National Research Council, Washington, D.C., 1988.

ROAD RESEARCH LABORATORY, A guide to the structural design of pavements for new roads, Road Note 29, Third Edition, Department of the Environment, H. M. Stationery Office, London, 1970.

SCALA, A. J., Comparison of the response of pavements to single and tandem axle loads. proceedings of the 5th ARRB Conference 5(4), 1970, pp. 231-252.

SCALA, A. J. and POTTER D. W., A method to predict load equivalence factors for specialised vehicles, Research Report, ARRB No. 114, Australian Road Research Board, Vermont South Victoria, Australia, 1981.

SEBAALY, P. and TABATABAEE, N., Effect of Tire Pressure and Type on Response of Flexible Pavement, Transportation Research Record 1227, Rigid and Flexible Pavement Design and Analysis, Transportation Research Board, National Research Council, Washington, D.C., 1989, pp. 115-127.

SEBAALY, P., TABATABAEE, N., BONAQUIST, R. and ANDERSON, D., Evaluating structural damage of flexible pavements using cracking and Falling Weight Deflectometer data, Transportation Research Record 1227, Rigid and Flexible Pavement Design and Analysis, Transportation Research Board, National Research Council, Washington, D.C., 1989, pp. 44-52.

SHACKLETON, M. C., Permanent deformation in pavements with granular bases and subbases, Draft Contract Report DPVT/C 164.1, DRTT, CSIR, Pretoria, 1991.

SMALL, K. A., WINSTON, C. and EVANS, C. A., Road work: a new highway pricing and investment policy, ISBN 0-8157-9470-3, The Brooking Institution, Washington, D.C., 1989.

SOUTH AFRICAN ROADS BOARD (SARB), Dynamic loading effects of heavy vehicles on road pavements, Project Report PR 91/280, Pretoria, Department of Transport, Chief Directorate National Roads, March 1992.

SOUTH AFRICAN ROADS BOARD (SARB), Nonlinear dynamic analysis and design of road pavements, Research Report RR 90/030, Pretoria, Department of Transport, Chief Directorate National Roads, March 1992.

SOUTH AFRICAN ROADS BOARD (SARB), Backfilling of bridge abutments and wing walls, Project Report PR 93/478, Pretoria, Department of Transport, Chief Directorate National Roads, March 1994.

SOUTH AFRICAN ROADS BOARD (SARB), Surfacing, base and tyre interaction for low cost pavement design, Project Report PR 93/559, Pretoria, Department of Transport, Chief Directorate National Roads, January 1995.

THE ASPHALT INSTITUTE, Asphalt overlays and pavement rehabilitation, Asphalt Institute Manual (MS - 17), USA, 1969.

THOMPSON, M. R., Implications of the stress dependant properties of granular materials in flexible pavement behaviour, ASTM Symposium, Atlanta, Georgia, December 1974.

TREYBIG, H. J., Equivalency factor development for multiple axle configurations, Transportation Research Record (TRR) 949, 62nd Annual Meeting, Washington, D.C., 1983, pp. 32-44.

ULLIDTZ, P., Overlay and stage by stage design, proceedings of the 4th International Conference on Structural Design of Asphalt Pavements, Volume 1, University of Michigan, Ann Arbor, USA, 22-26 August 1977, pp. 722-735.

VAN DE LOO, P. J., A practical approach to the prediction of rutting in asphalt pavements - The Shell Method, TRB, Washington, D.C., 1976.

VAN VUUREN, D. J., Die toelaatbare wiel- en asmassa van swaar voertuie, D.Sc. Thesis, Faculty of Engineering, University of Pretoria, Pretoria, May 1972.

VAN ZYL, N. J. W. and FREEME, C. R., Determination of relative damage done to roads by heavy vehicles, Research Report RR622, reprint from the proceedings of the Annual Transport Convention (ATC 1984), S.343, Vol C-1, Paper C-1-4, Pretoria, August 1984.

VILJOEN, A. W., The equivalency coefficient and HVS trafficking, Technical Note TP/32/84, NITRR, CSIR, Pretoria, 1984.

WALKER, R. N., PATERSON, W. D. O., FREEME, C. R. and MARAIS, C. P., The South African Mechanistic Pavement Design Procedure, proceedings of the 4th International Conference on the Design of Asphalt Pavements, Ann Arbor, Michigan, August 1977.

WOLFF, H., The elasto-plastic behaviour of granular pavement layers in South Africa, Ph.D. Thesis, University of Pretoria, Pretoria, November 1992.

YODER, E. J. and WITCZAK, K. M. W., Principles of pavement design, Second Edition, New York, Wiley-Interscience Publications, 1975.

Appendix A - Detail of the field study conducted on the N4-1

1. INTRODUCTION

A vehicle axle weight and deflection study was conducted during November 1991 on the N4-1 between Pretoria and Witbank. The purpose of this study was to investigate the response (resilient deflection) of a pavement under multiple wheel and axle configuration loads and to gather data from which load equivalency factors (LEFs) for different wheel and axle arrangements could be calculated. The methods used for calculating LEFs include the deflection-based method (method proposed in this study) and other widely used and accepted methods such as the method derived at the AASHO Road Test, simplifications thereof, such as the load based "Fourth Power Law" and the method used by the California Division of Highways, and the HVS-based method used in South Africa. All these methods were presented and discussed earlier. The following paragraphs briefly outlines the procedures followed during the study in gathering the data and calculating the respective LEFs.

2. LOCATION AND LAYOUT OF TEST SECTION

The test section is located on the eastbound lane at km 20,2 on the N4-1 between Pretoria and Witbank, as shown in Figure A-1. The pavement structure, shown in Figure A-2, consisted of a 50 mm asphalt surface, a 120 mm cement treated base layer, a 100 mm cemented crusher-run subbase, two selected layers of 115 mm and 165 mm respectively, and an in situ layer consisting of a sandy clay. The N4 route is a major highway, which link the Pretoria/-Witwatersrand area to the eastern Transvaal. The N4 can typically be classified as an A-category road, designed for heavy trafficking (typical E3/E4 traffic class; road and traffic classification according to TRH4 (CSRA, 1989)).

3. FIELD TEST PROCEDURE AND RESULTS

The test section constituted approximately a 100 m section of the slow lane, which was closed to normal traffic during the test operation. Approaching heavy vehicles were diverted onto this test section and the test procedure involved the measurement of the respective axle loads, the recording of vehicle wheel and axle dimensions and lastly, the measurement of the vertical deflections using the multi-depth deflectometer (MDD).

3.1 MEASURING OF AXLES (Static axle weight)

The individual axles were weighed (units of 1000 kg/tons) by means of a Vehicle Load Monitor (VLM). This transportable device was developed by the Division of Roads and Transport Technology and facilitated the measurement of each axle individually, while the vehicle slowly drove over a weigh pad. The device was calibrated on site prior to the start of the investigation, by means of a special test vehicle (vehicle generally used in conjunction with the Road Surface Deflectometer (RSD), known as the RSD truck, as it has a pre-loaded rear axle which conforms to the standard wheel and axle configuration load of 80 kN (8200 kg))

A total of 25 heavy vehicles were evaluated. The observed information allowed for axles to be grouped into three categories, i.e. single axles with single wheels, single axles with dual wheels and tandem axles with dual wheels. The loads on the axles ranged from 2,4 to 7 tons (23,5 to 68,6 kN) for single axles with single wheels; 3,1 to 13,5 tons (30,4 to 132,3 kN) for single axles with dual wheels and from 5,4 to 25,3 tons (52,9 to 247,9 kN) for the tandem configurations.

3.2 RECORDING OF VEHICLE INFORMATION

The vehicle information which were recorded included:

a) The number of axles
b) Spacing of axles (centre to centre)
c) Wheel configuration, i.e. single or dual
d) Spacing of wheels (centre to centre)
e) Tyre type and size
f) Tyre pressure
g) Vehicle speed

The variation in wheel and axle spacings within an axle group, i.e. single or tandem axle arrangements, were found to be minimal for this test. Axle spacings within a tandem arrangement (sample size of 22) varied between 1,3 m to 1,4 m (centre to centre) and the wheel spacings between 350 and 360 mm (centre to centre). For single axles with dual tyres (sample size of 16), the range in wheel spacings was somewhat wider, i.e. 280 mm to 350 mm (centre to centre). Tyre type, tyre size and tyre pressure were recorded for the sake of completeness, as these variables have little or no effect on deflections. The evaluation of the effect of vehicle speed on deflection did not form part of this study. However, the influence thereof may be substantial when speeds reach excessive high values (speeds $\geq$ 30 km/h). The aim was therefore to control the influence of this variable to some extent, viz, to control vehicle speeds to approximately creep speed conditions (i.e. vehicle speeds $\leq$ 15 km/h). Vehicle speeds were automatically calculated and recorded by a computer, via the trigger sensors (piezo electrical strip), used during the measurement of the deflections (a detailed reference to the method of measurement and the specifications on the equipment used, is given by De Beer (1991)). Vehicle speeds ranged between 3,0 and 15,2 km/h (average of 6,6 km/h).

3.3 DEFLECTION SURVEY

Deflections were measured with Multi-depth Deflectometers (MDDs), which were installed at depths of 0 mm (surface), 160 mm (bottom of cemented base), 310 mm (top of selected layer 1) and 540 mm (bottom of selected layer 2). The measurement of the complete deflection bowl for each of the in-depth sensors were recorded simultaneously by means of a computer. The data for each vehicle was stored separately (separate files on disk), which facilitated the processing of the data thereafter. The test vehicle (RSD truck) was used to measure the standard deflection profile under a standard axle configuration load. The results of the recorded deflections for each of the vehicles are shown in Figures A-3 to A-28.

4. LEF CALCULATIONS

LEFs for the respective axle and load configurations were calculated using several methods such as the AASHO method, the Fourth Power Law (n = 4), the method used by the California Division of Highways (CDH) (n = 4,2), the HVS based method and the proposed deflection-based method. Axles were grouped into three categories, i.e. single axles with single wheels, single axles with dual wheels and tandem axles with dual wheels. The LEFs calculated for each axle groups are given in Tables A-1 to A-3. The following paragraphs outlines the methodology used in calculating LEFs from each of these methods.

4.1 THE AASHO METHOD

LEFs for the measured axle loads were calculated using the tabulated AASHO LEFs values (AASHTO, 1974). A terminal serviceability level (p_t) of 2 was chosen for the pavement section. The corresponding pavement structural number (SN) was calculated using both the material coefficients derived at the AASHO Road Test, and the revised values proposed by Otté (1972) for materials in South Africa. However, the results were in close agreement (SN = 4,06 and SN = 4,02 using the AASHO and the revised material coefficients respectively). Thus, for a chosen terminal serviceability (p_t) of 2, a calculated pavement structural number (SN) of 4, the LEFs for the respective axle loads were determined. An important point to take cognisance of is that the AASHO LEFs were developed for axles with dual wheel configurations. The method therefore does not distinguish between axles with single or dual wheels and thus, assumes that the effects are equal.

4.2.2 Fourth Power Law

LEFs were calculated by applying the two basic equations, i.e.:

$$LEF = \left(\frac{L}{80}\right)^4 \qquad \textit{for dual-tyred single axles}$$

and

$$LEF = \left(\frac{L}{147}\right)^4 \qquad \textit{for dual-tyred tandem axles}$$

in which:	LEF	=	the load equivalence factor
	L	=	measured axle load
	80	=	reference dual-tyre single axle load in kN
	147	=	reference dual-tyre tandem axle load in kN used at the AASHO Road Test

However, an important point to take cognisance of is that the Fourth Power Law is a simplification of the AASHO method, which only considers dual wheel configurations. The method therefore does not differentiate between axles with single or dual wheels and thus the effects of the two wheel configurations are assumed to be equal.

4.2.3 California Division of Highways (CDH)

LEFs were calculated by applying the basic design equation, i.e.:

$$LEF = \left(\frac{L}{80}\right)^{4,2} \qquad \textit{for dual-tyred single axles}$$

in which:	LEF	=	the load equivalence factor
	L	=	measured axle load
	80	=	reference dual-tyre single axle load in kN

The above equation only considers single axles with dual wheels. The method therefore assumes that the effects of single wheels and dual wheels are equal and that the effect of a tandem axle is equal to the combined effect of the individual axles, viz that the LEF for a tandem axle is equal to the sum of the LEFs calculated for each of the individual axles in the tandem group.

4.2.4 HVS-based method

The general equation for calculating LEFs using the HVS-based method is:

$$LEF = \left(\frac{L}{80}\right)^{d}$$

in which:	LEF	=	the load equivalence factor
	L	=	measured axle load
	80	=	reference dual-tyre single axle load in kN
	d	=	relative damage coefficient

Factors to be considered when applying the above equation is the type and state of pavement, as these factors influence the value of the damage coefficient to be used. For the pavement section evaluated during the field test, a value of d = 3 was chosen (pavement with a cement stabilised base in a post-cracked state).

The HVS-based method, and subsequently the above equation, only applies to single axles with dual wheels. Therefore the method assumes that the effects of single wheels and dual wheels are equal. A further assumption of the method is that the effect of a tandem axle is equal to the combined effect of the individual axles, viz that the LEF of a tandem axle is the sum of the LEFs calculated for each of the individual axles in the tandem group.

4.2.5 Deflection-based method

The deflection-based LEFs were calculated using the measured surface deflections and the following equations:

Single axles:

$$LEF = \left(\frac{D_l}{D_b}\right)^{3,8}$$

Tandem axles:

$$LEF = \left(\frac{D_l}{D_b}\right)^{3,8} + \left(\frac{\Delta_1}{D_b}\right)^{3,8}$$

in which:

in which:	LEF	=	load equivalency factor
	D_b	=	standard deflection (average of the deflections measured under the 80 kN dual-tyred single axle — Figures A-3, A-4 & A-28)
	D_l	=	deflection under various single axle loads, or deflection under leading axle in the case of tandem axles
	Δ_1	=	difference between maximum deflection under the second axle and the intermediate deflection between axles

The method facilitates the calculation of LEFs for any wheel and axle configuration, as the method uses a response ratio (deflection) as the basis for its LEF calculations, as opposed to a load ratio used in previous methods. Thus, LEFs for the three cases evaluated during this study, i.e. single axles with single wheels, single axles with dual wheels and tandem axles with dual wheels, were calculated using the deflection responses measured under each of these configurations.

5. REFERENCES (APPENDIX A)

1. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORT OFFICIALS (AASHTO) (1974). AASHTO Interim Guide for Design of Pavement Structures 1972, Washington, D.C.

2. DE BEER M (1991). Pavement response measuring system. Paper intended for the 2nd International Symposium on Pavement Response Monitoring Systems for Roads and Airfields, September 6 - 9, 1991, Hanover, New Hampshire, USA.

3. COMMITTEE OF STATE ROAD AUTHORITIES (CSRA)(1989). TRH 4: 1985, Structural design of interurban and rural road pavements. Department of Transport, Pretoria, South Africa.

4. OTTÉ E (1972). The applicability of the AASHO Road Test results to pavements in South Africa, Report RP/2/72, NITRR (later DRTT), CSIR, Pretoria, South Africa.

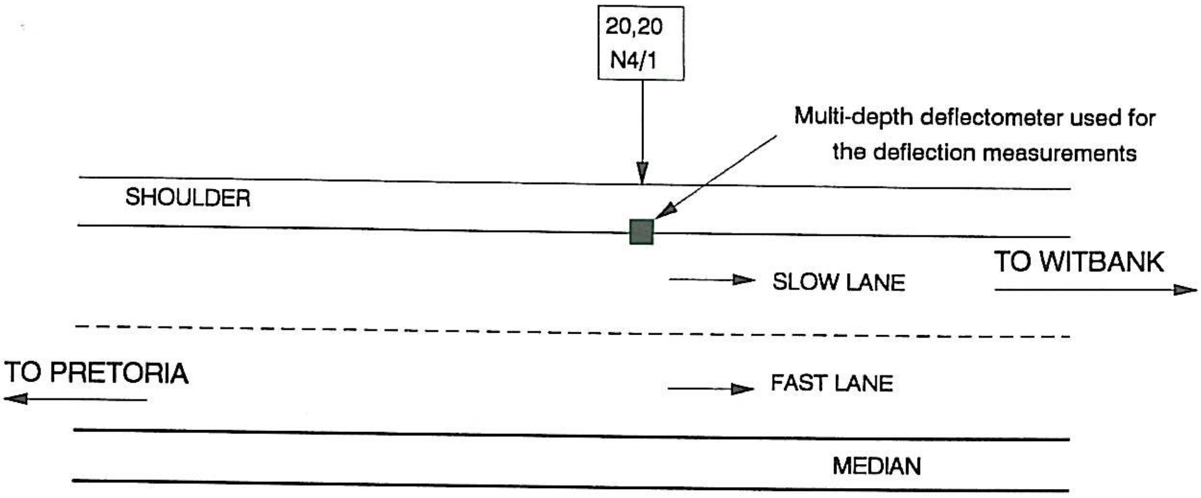


FIGURE A-1

LOCATION OF TEST SECTION ON THE N4-1
BETWEEN PRETORIA AND WITBANK

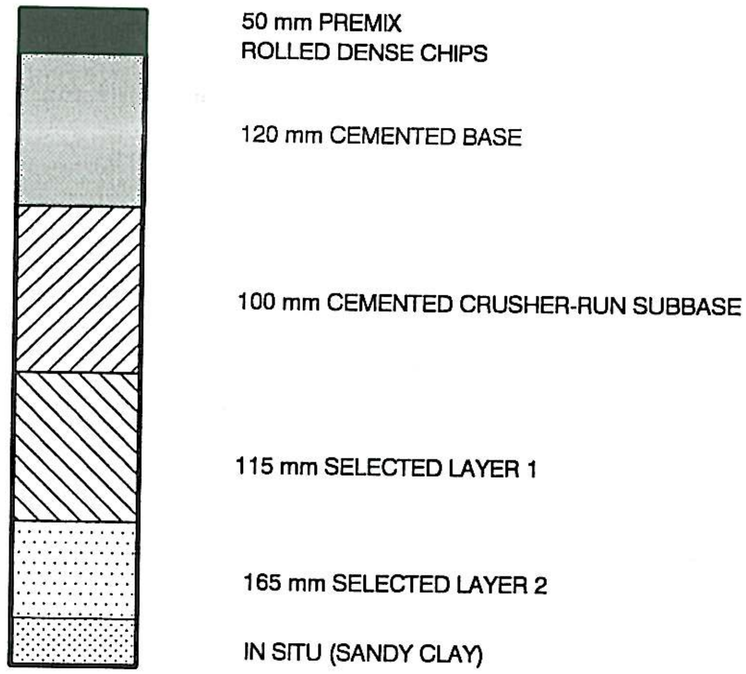
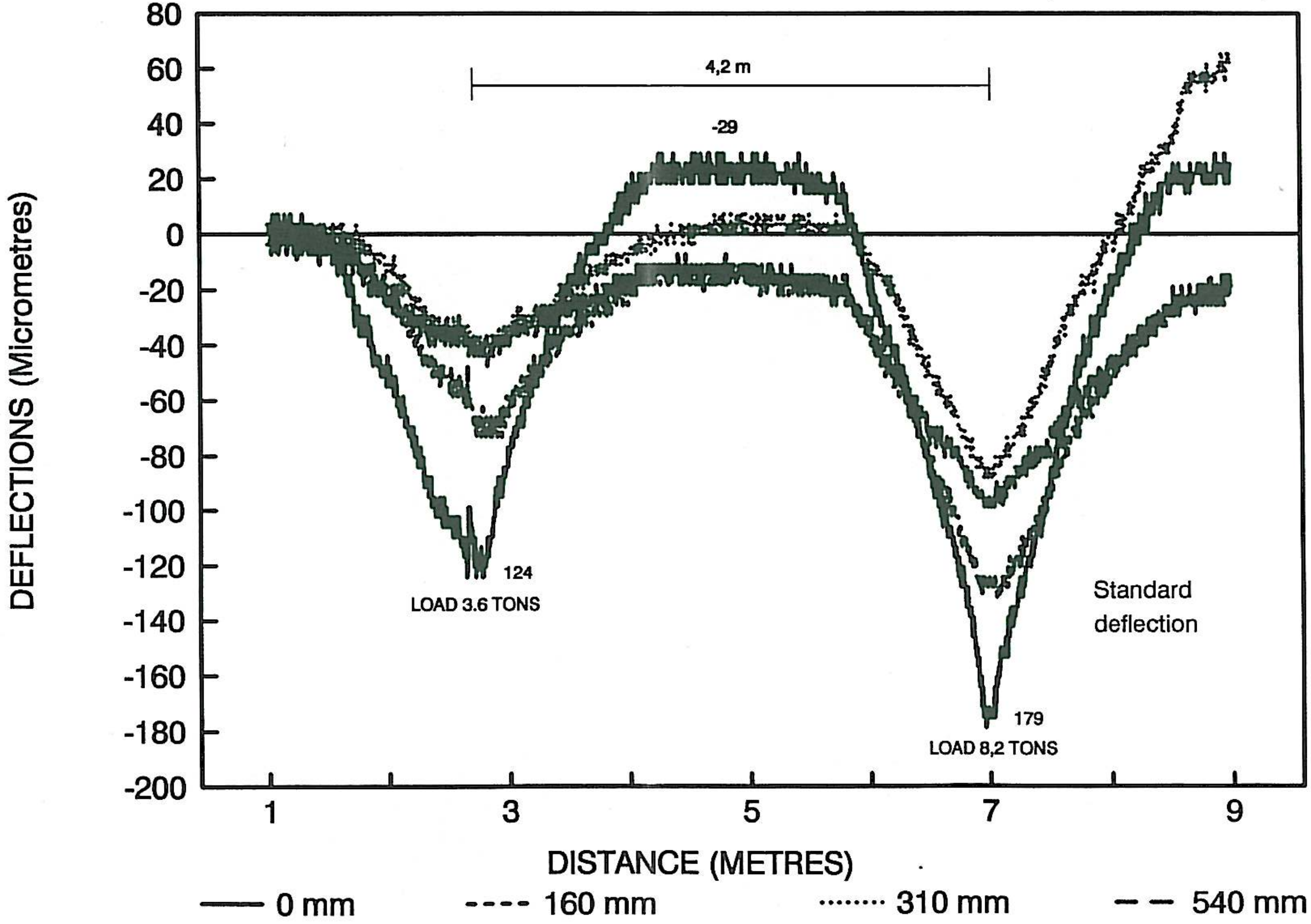


FIGURE A-2

LOCATION AND CONSTRUCTION DETAIL OF TEST SECTION
ON THE N4-1 BETWEEN PRETORIA AND WITBANK

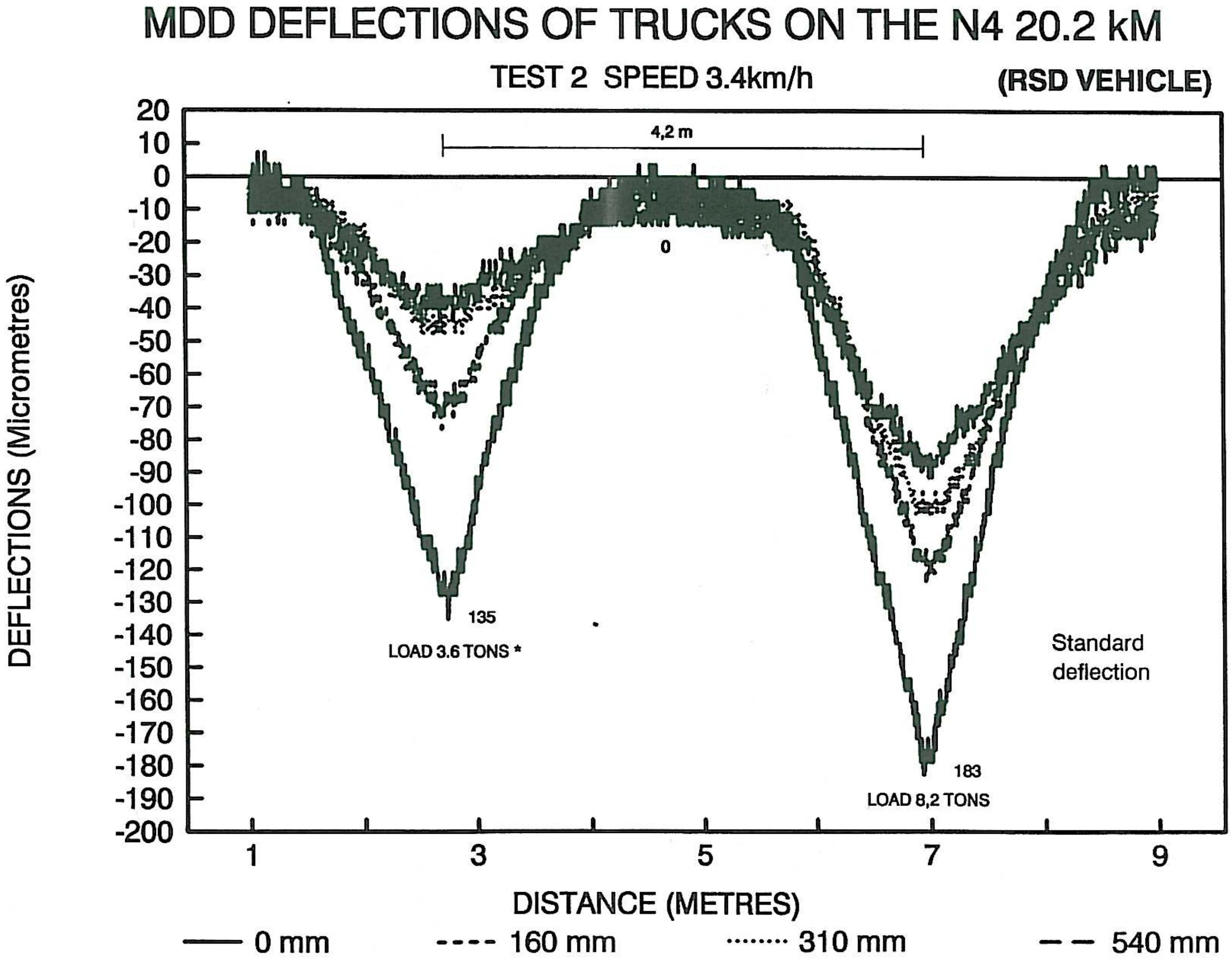
MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 1 SPEED 3.1km/h (RSD VEHICLE)



NR1.DRW

FIGURE A-3



NR2.DRW

FIGURE A-4

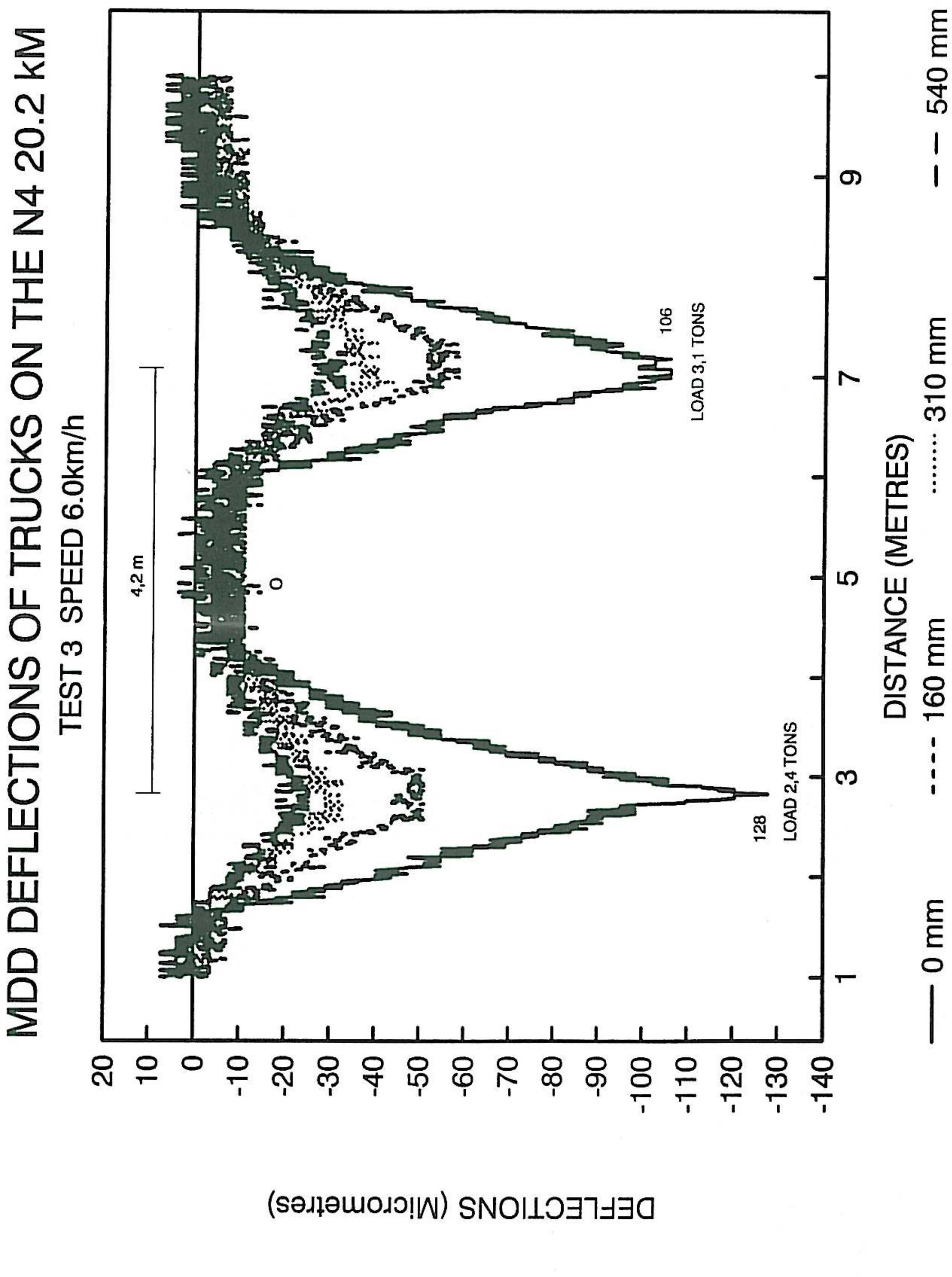
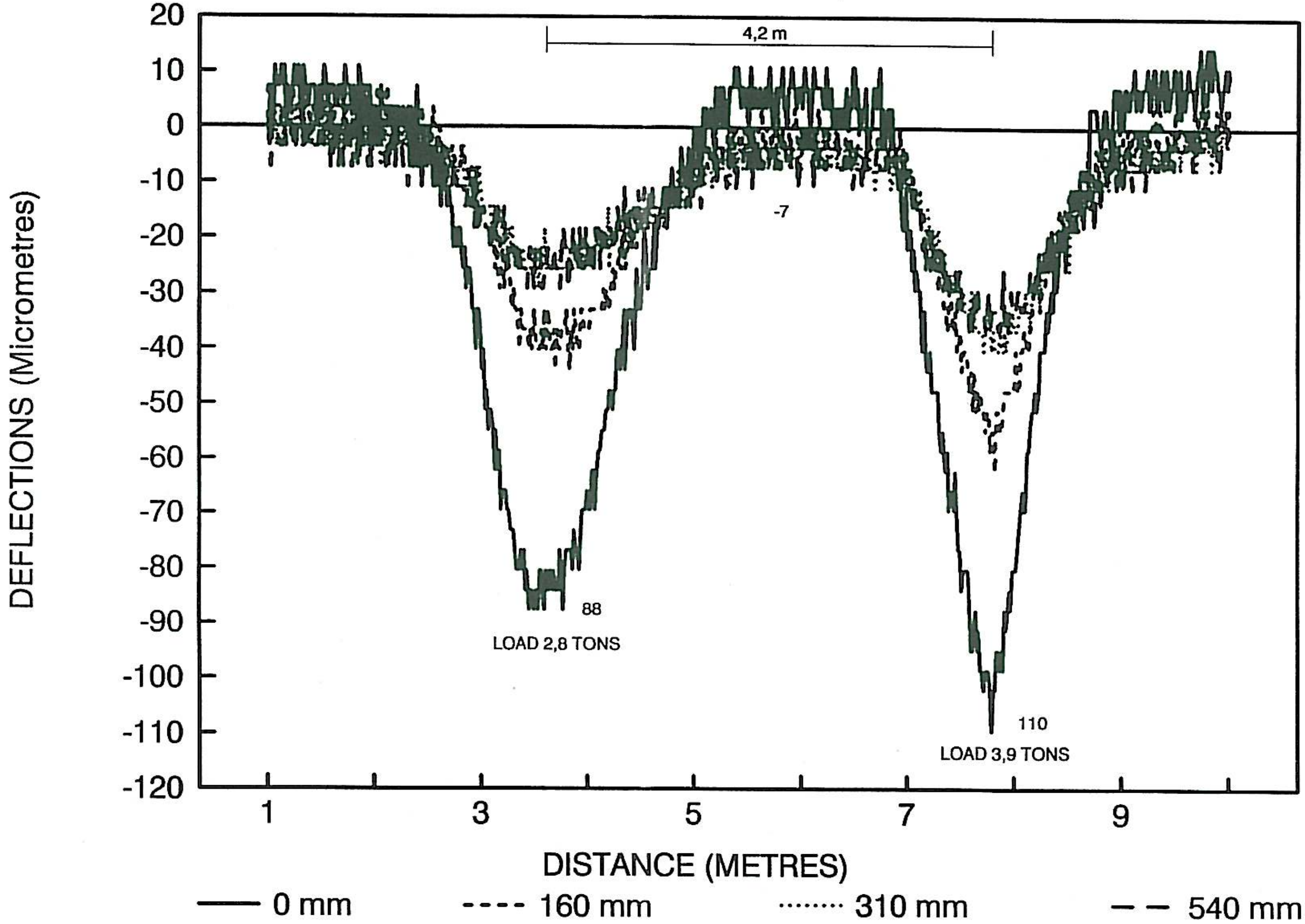


FIGURE A-5

NR3.DRW

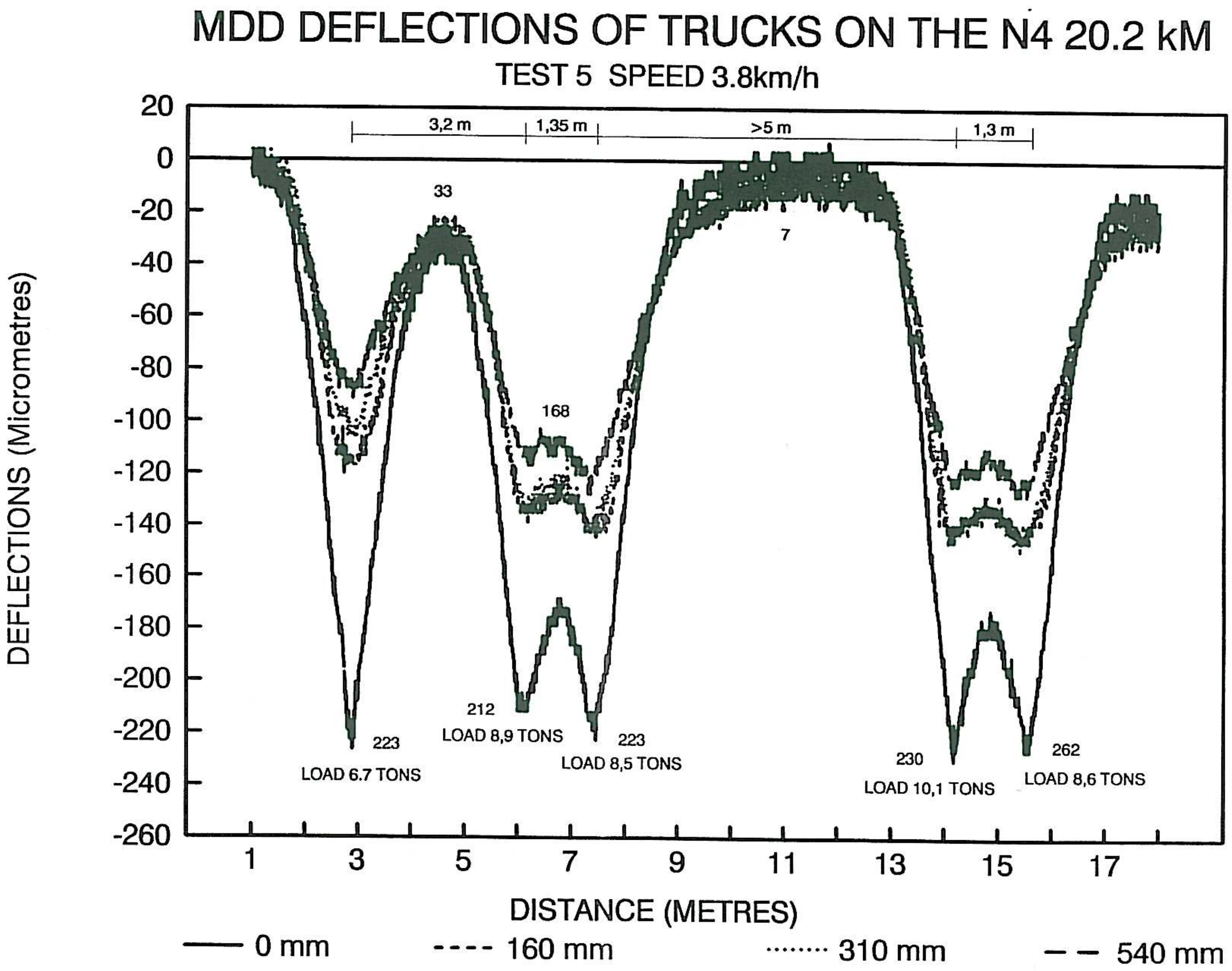
MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 4 SPEED 8.0km/h



NR4.DRW

FIGURE A-6



NR5.DRW

FIGURE A-7

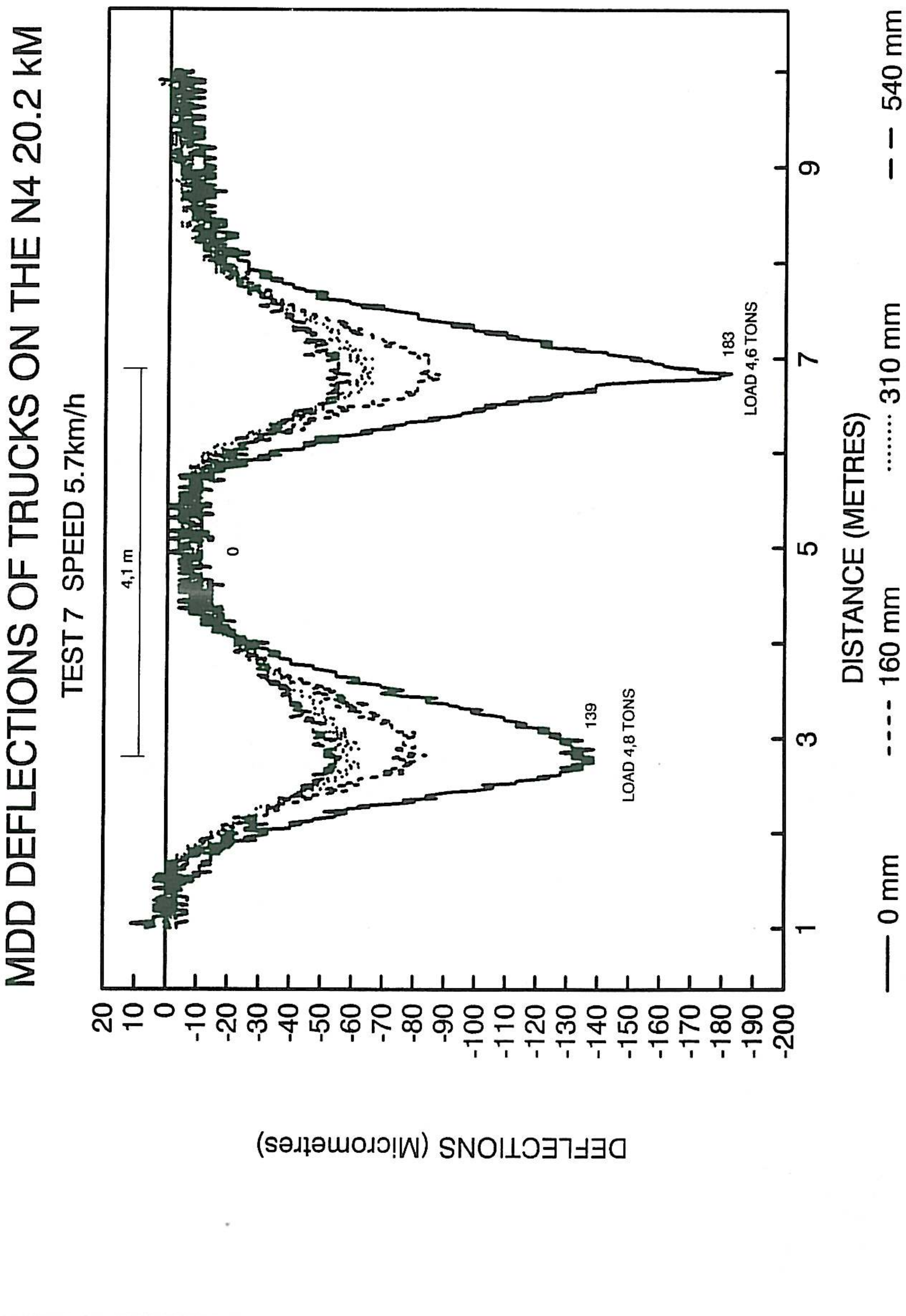
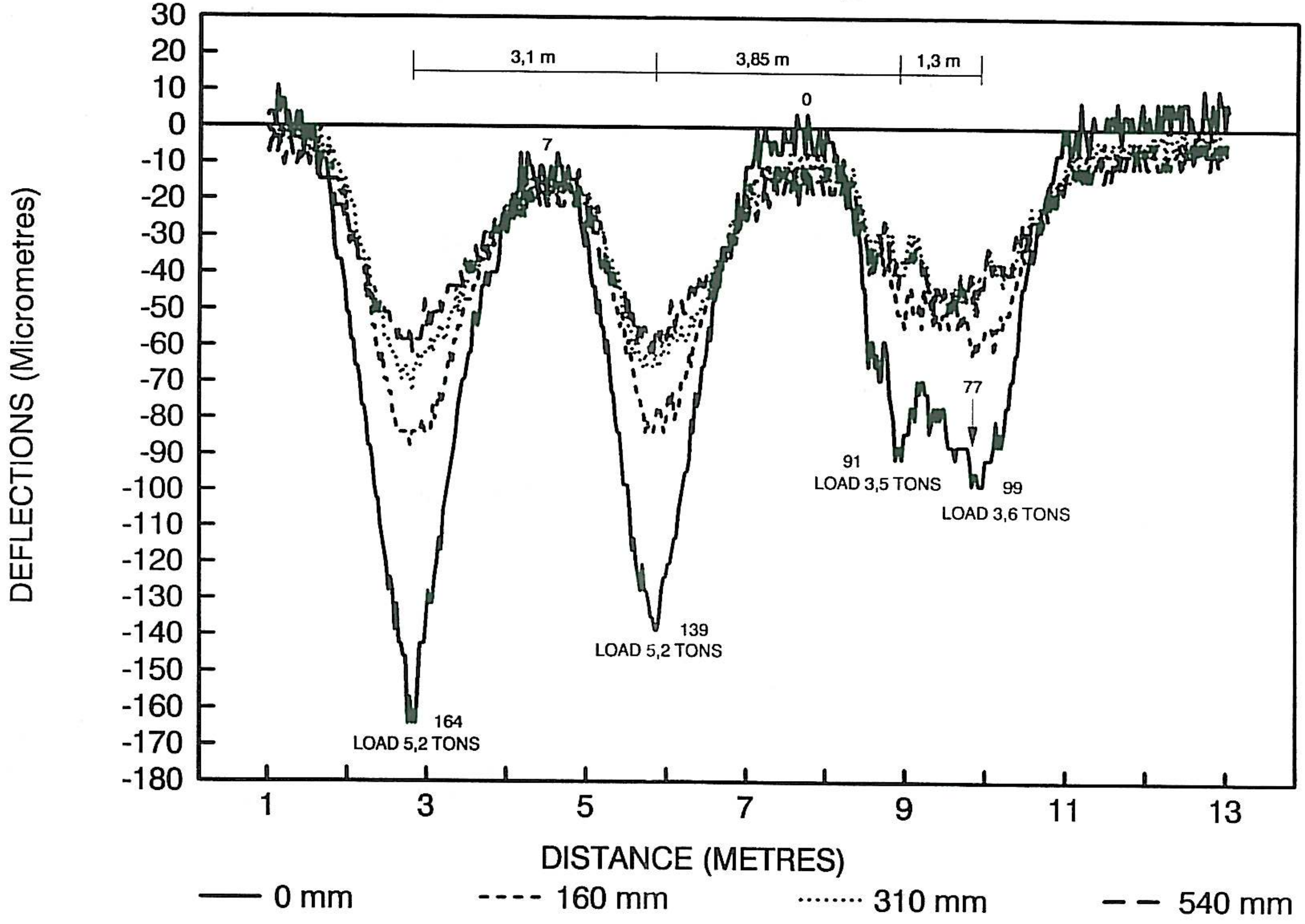


FIGURE A-8

NR7.DRW

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 8 SPEED 9.5km/h



NR8.DRW

FIGURE A-9

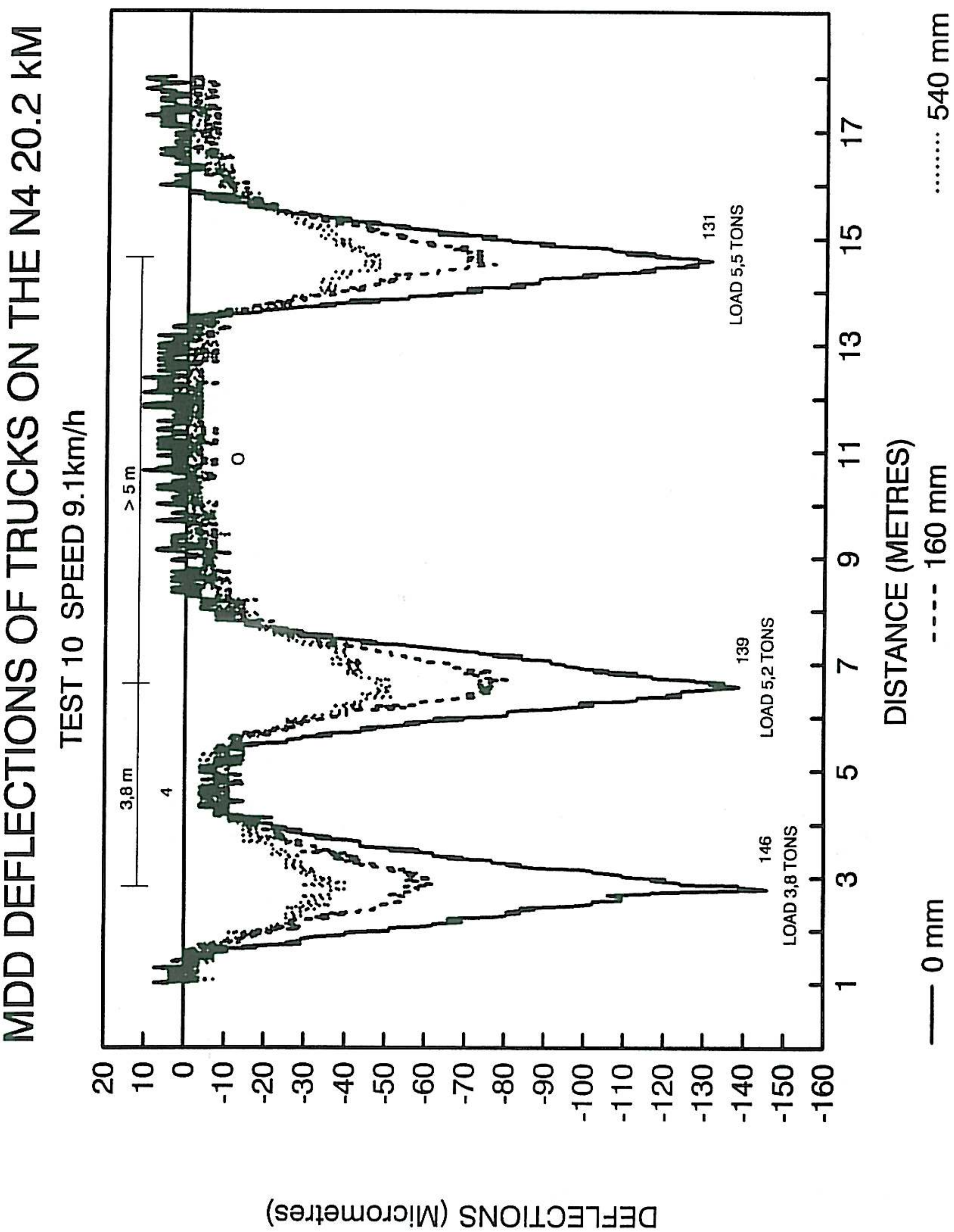
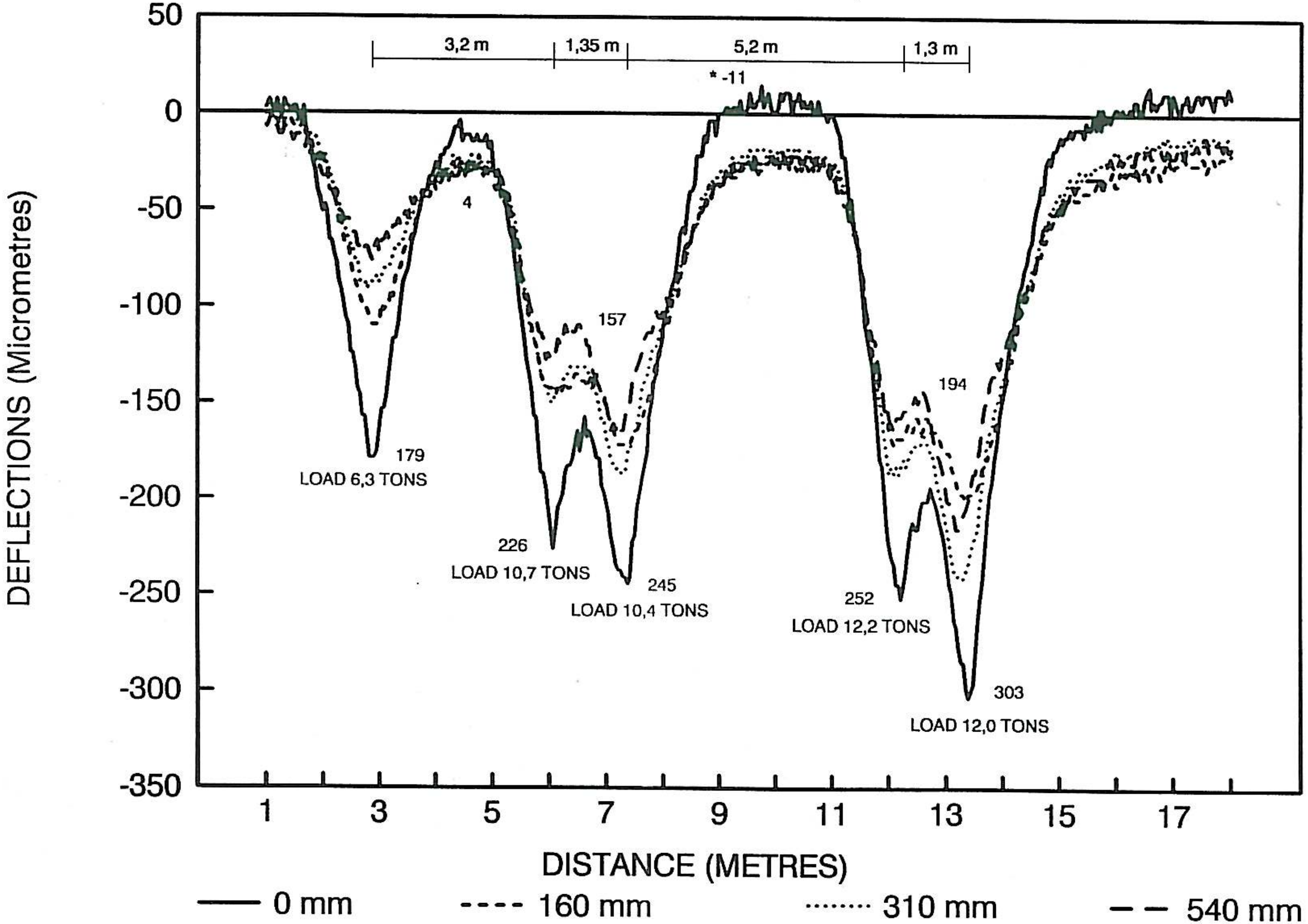


FIGURE A-10

NR10.DRW

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 11 SPEED 15.2km/h



NR11.DRW

FIGURE A-11

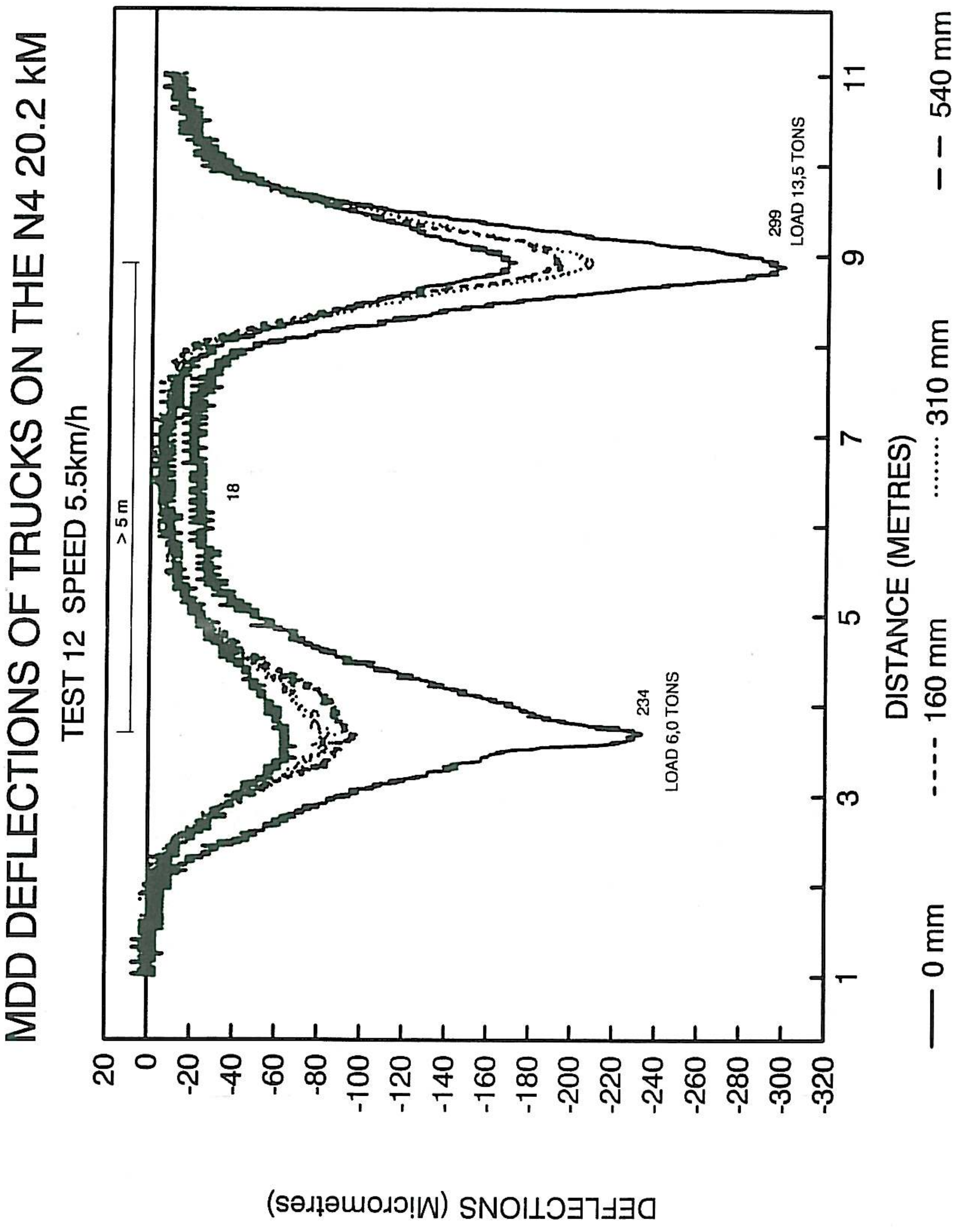
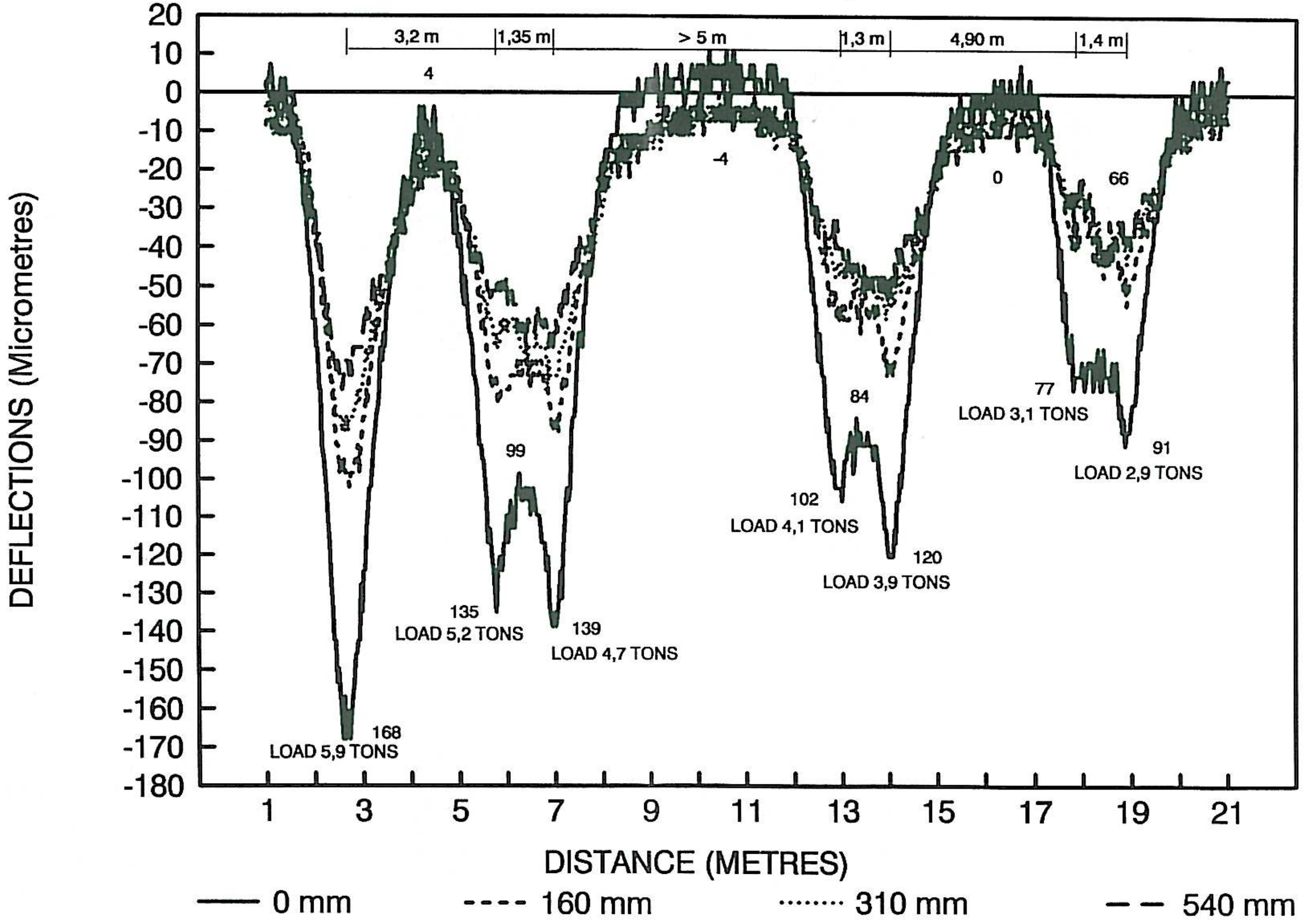


FIGURE A-12

NR12.DRW

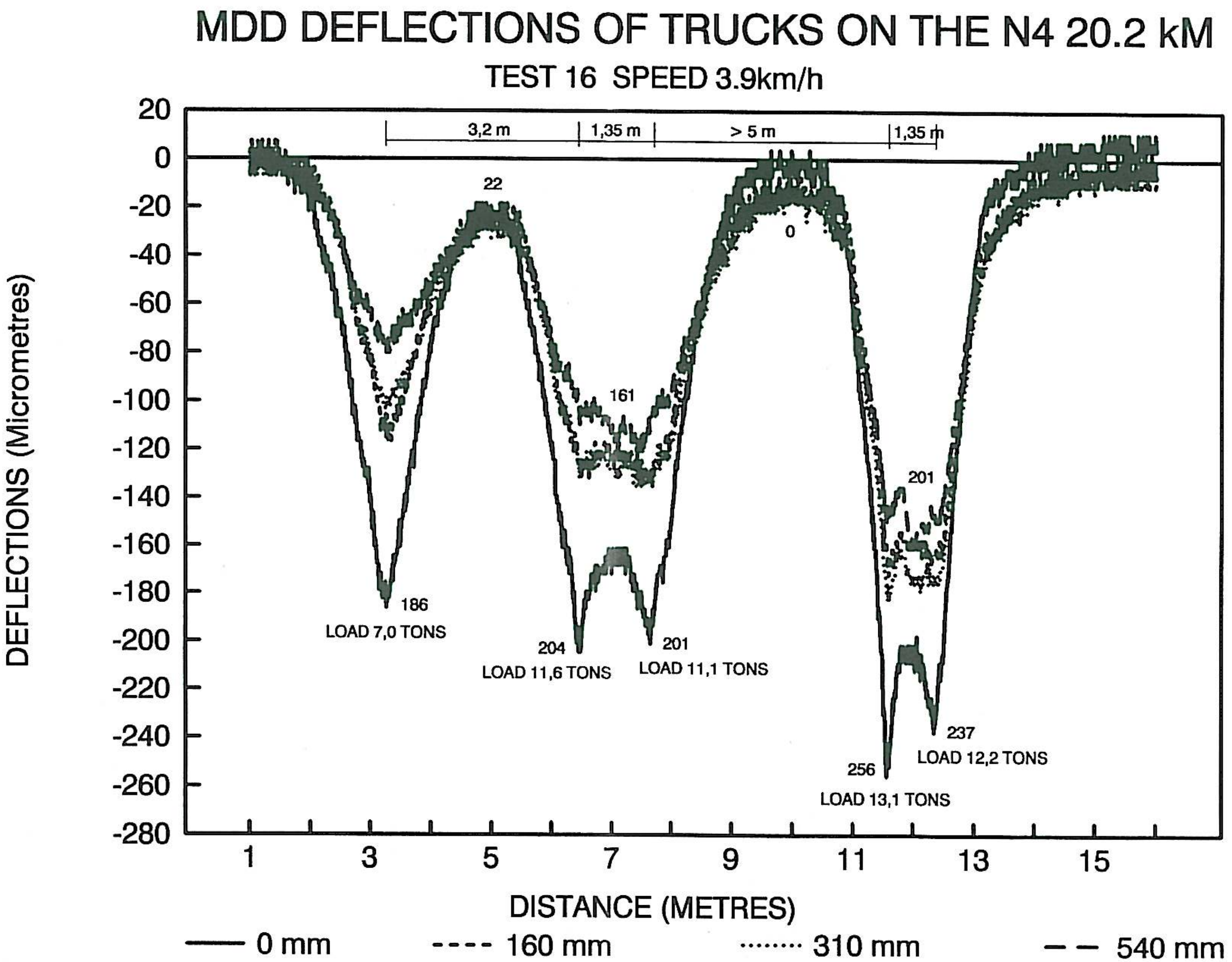
MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 14 SPEED 9.6km/h



NR14.DRW

FIGURE A-13

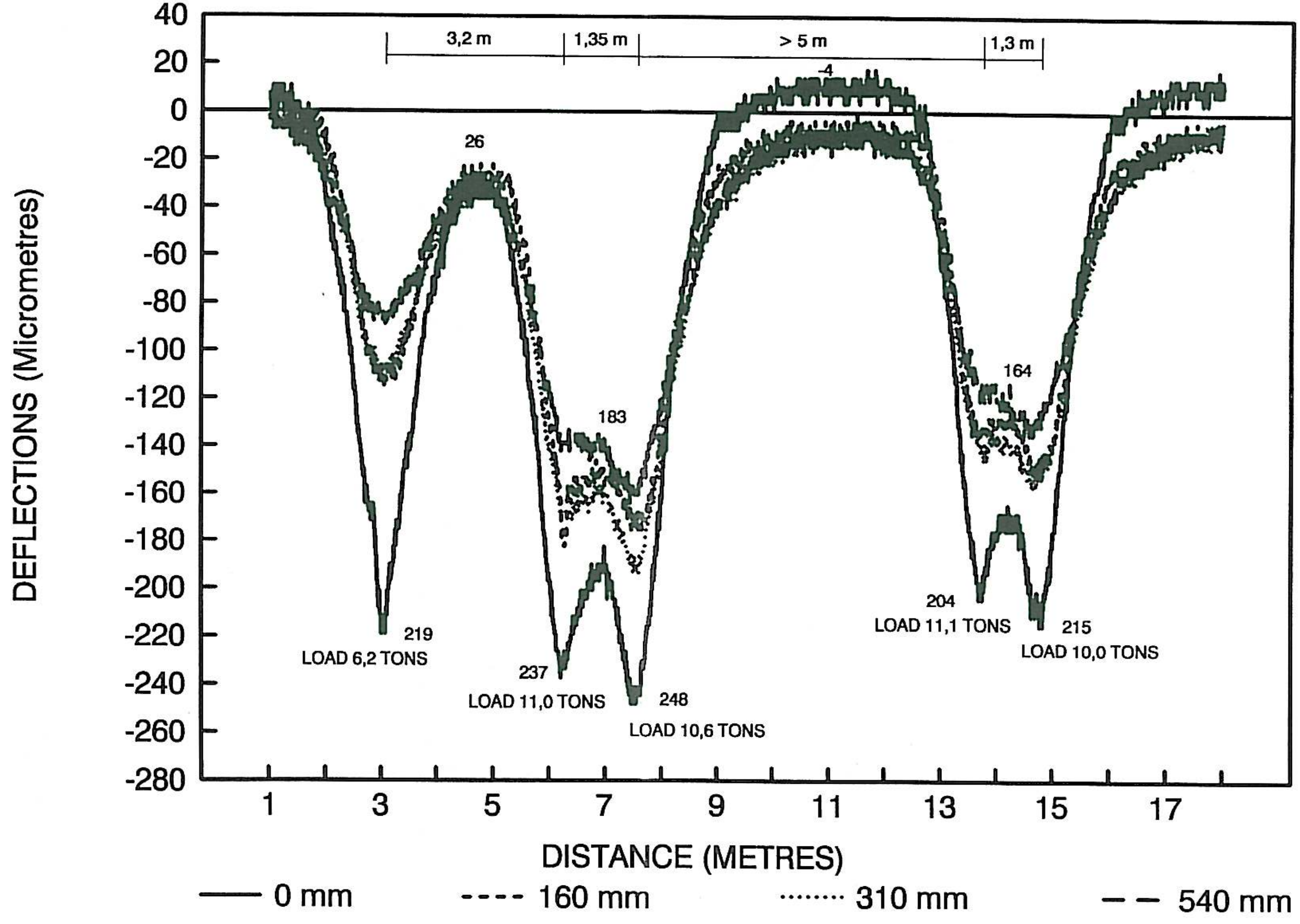


NR16.DRW

FIGURE A-14

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 17 SPEED 3.7km/h

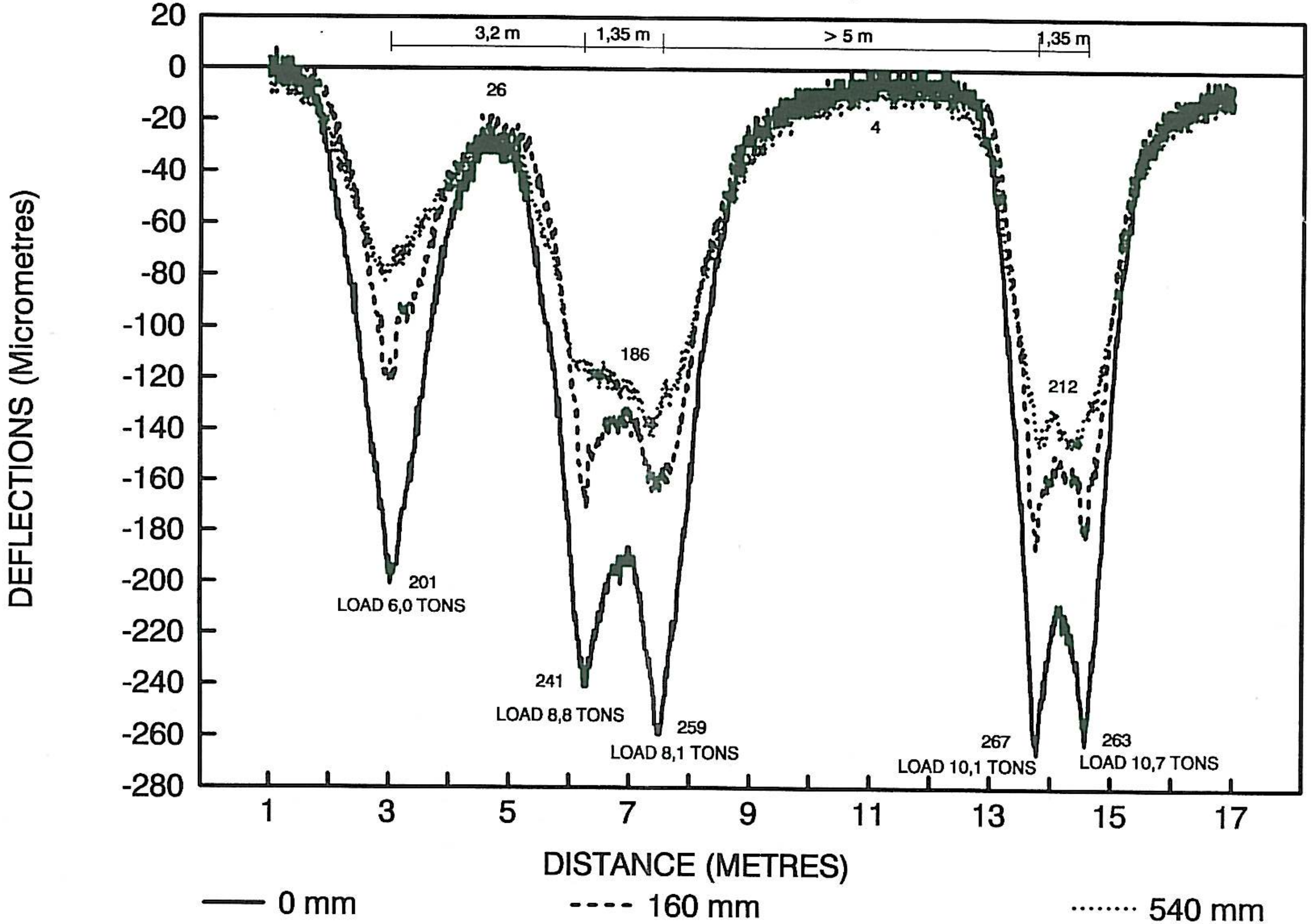


NR17.DRW

FIGURE A-15

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 18 SPEED 5.0km/h

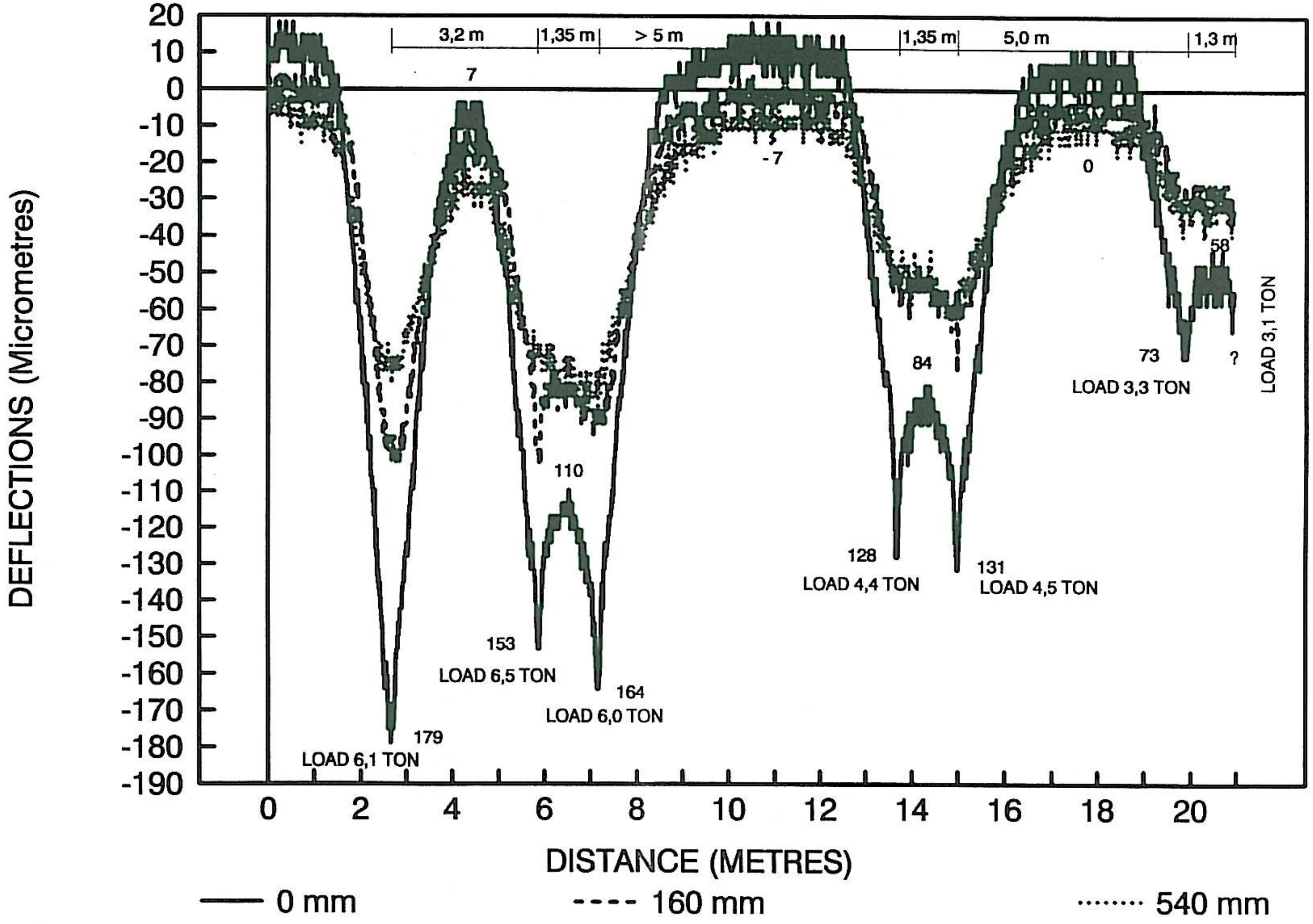


NR18.DRW

FIGURE A-16

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 km

TEST 19 SPEED 3.8 km/h



NR19.DRW

FIGURE A-17

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 22 SPEED 7.0km/h

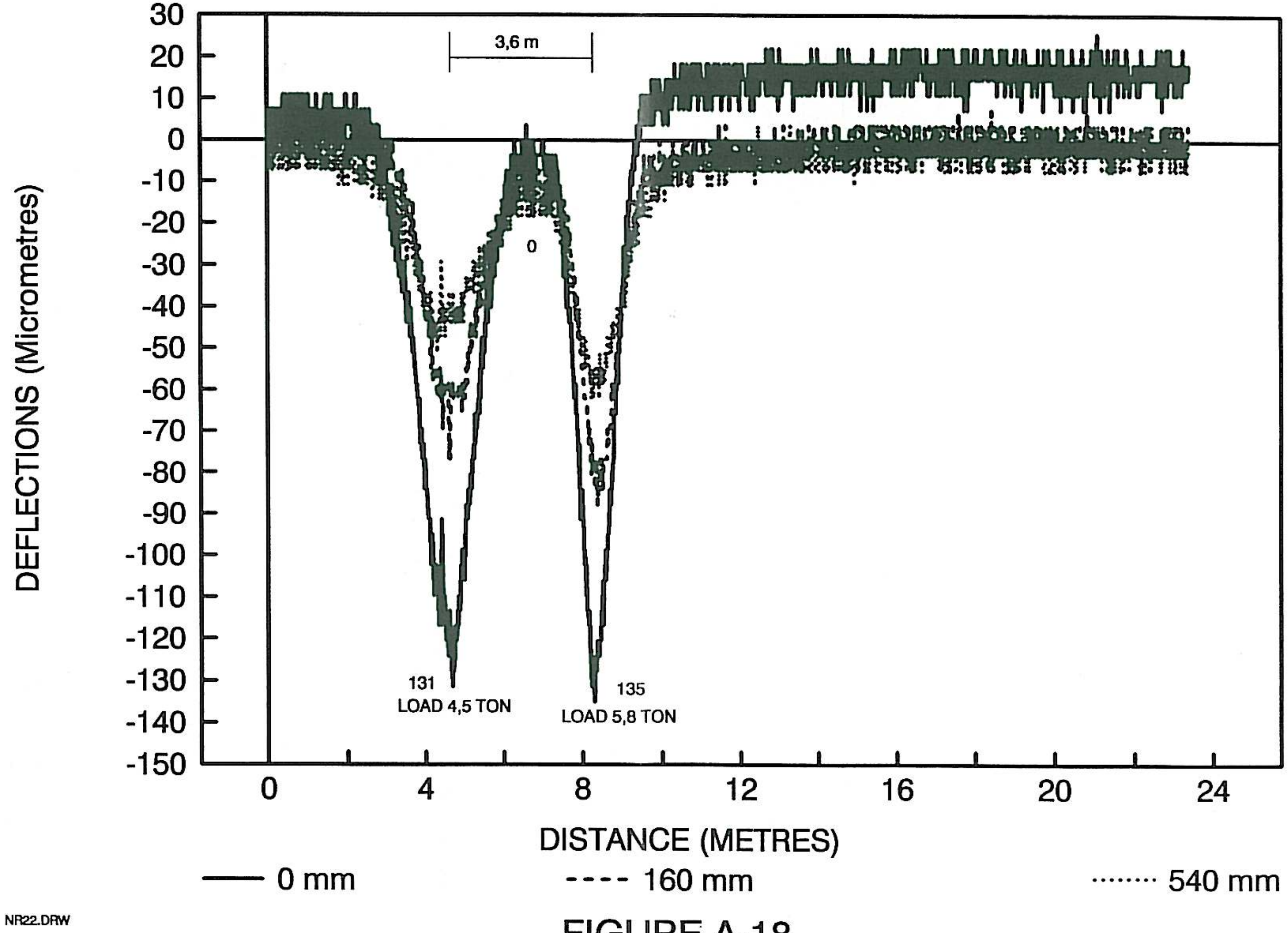
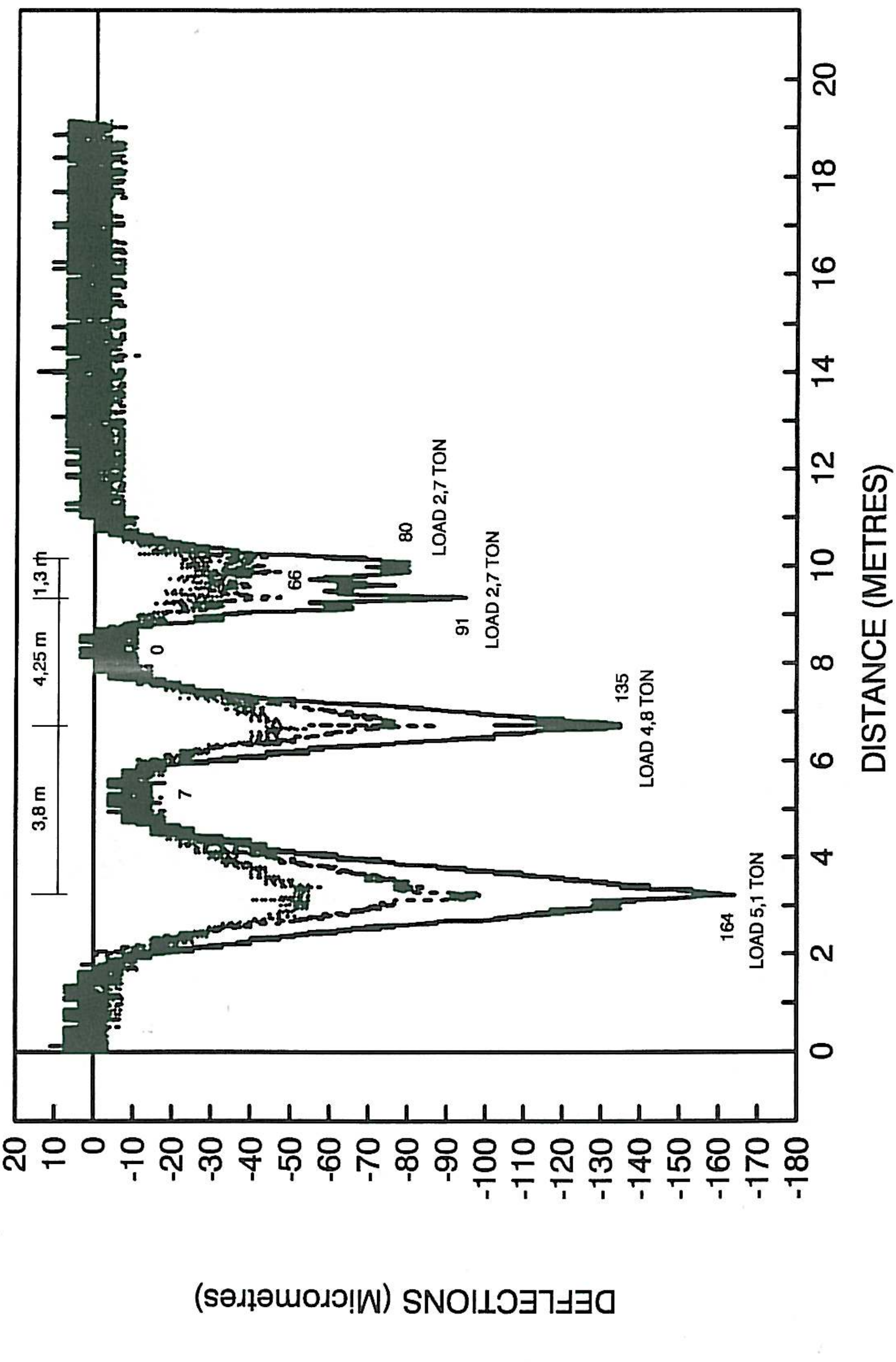


FIGURE A-18

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 23 SPEED 3.8km/h



—— 0 mm - - - - 160 mm .... 540 mm

FIGURE A-19

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

TEST 24 SPEED 6.6km/h

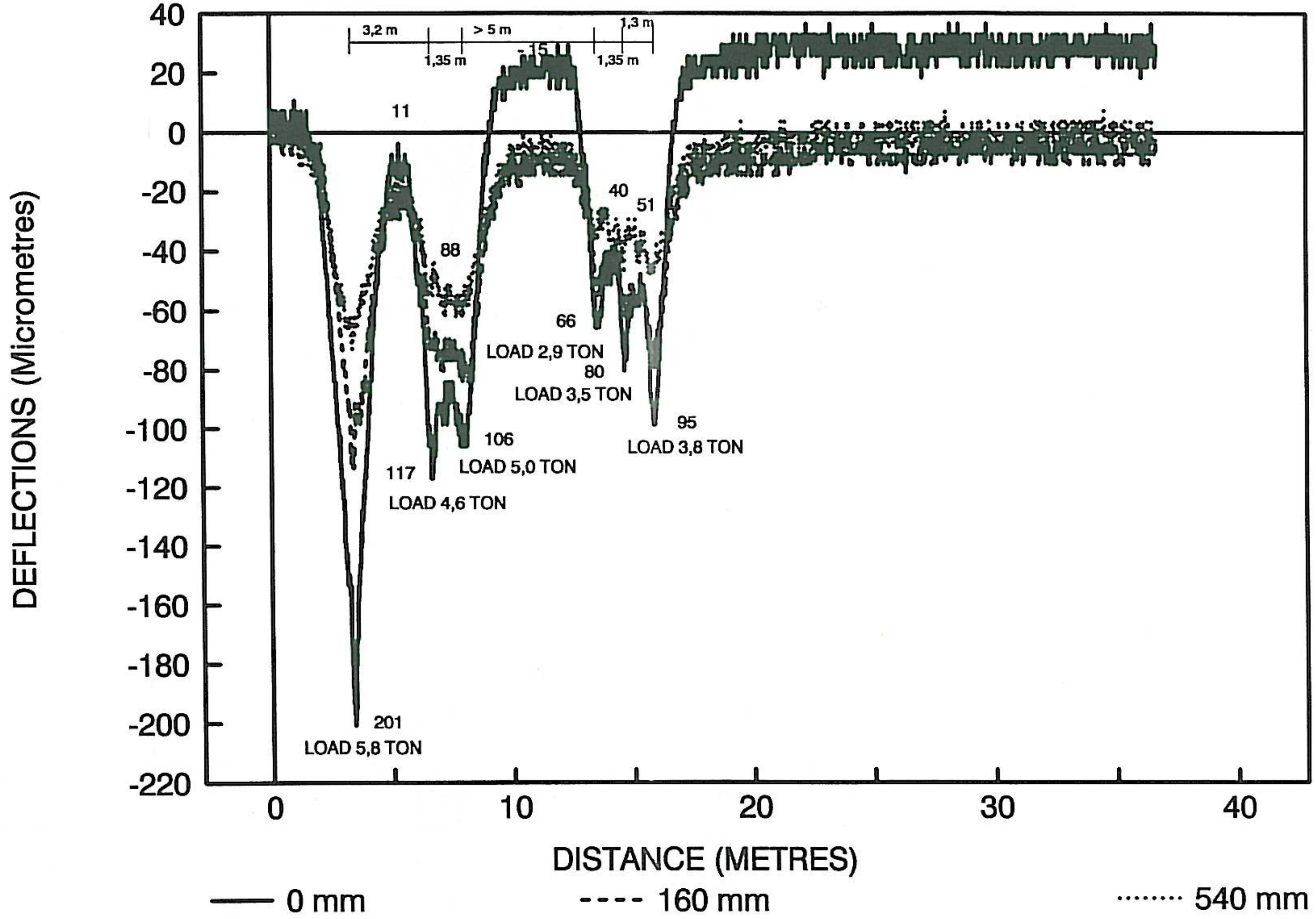


FIGURE A-20

NR24.DRW

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 kM

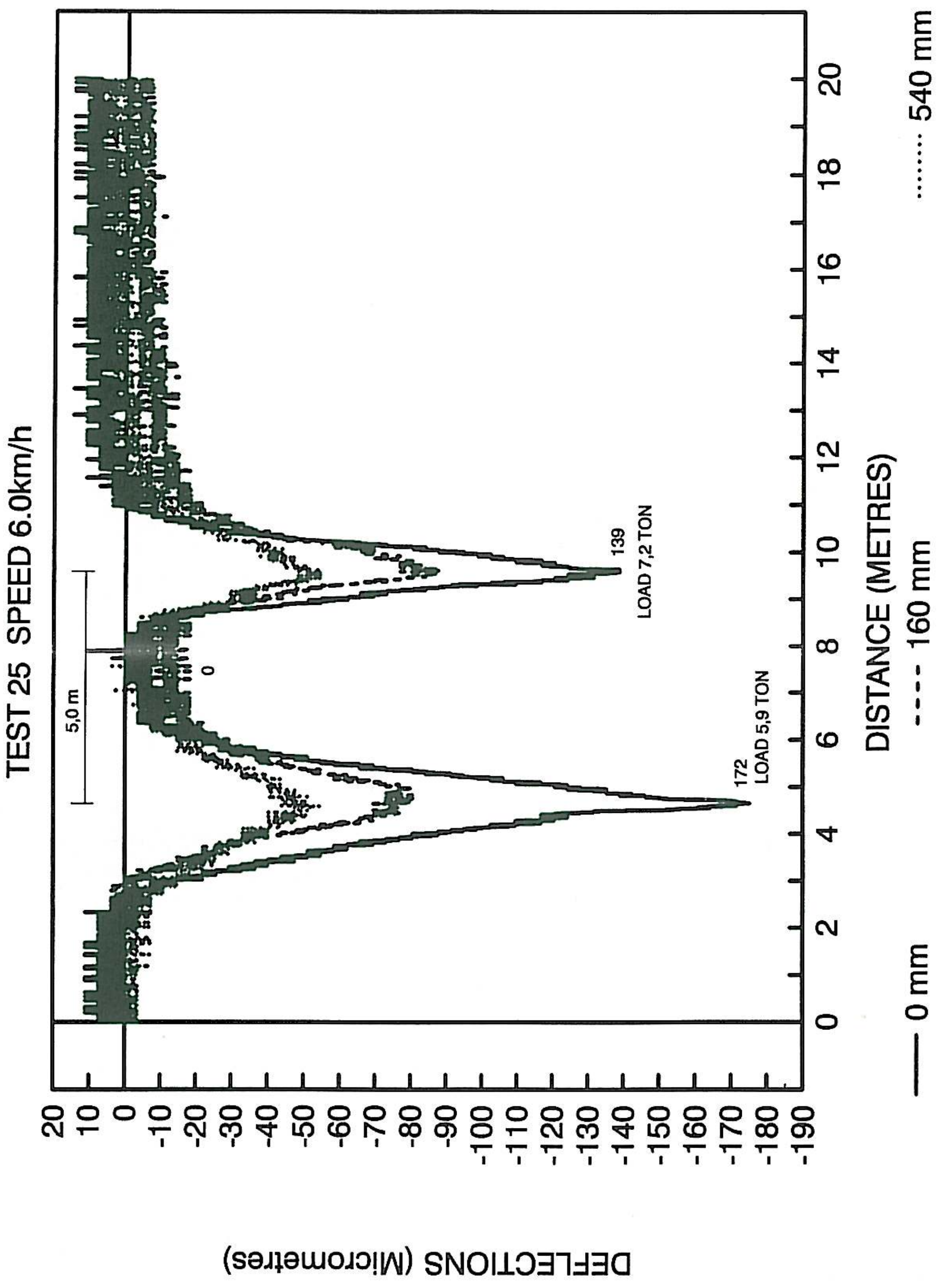


FIGURE A-21

NR25.DRW

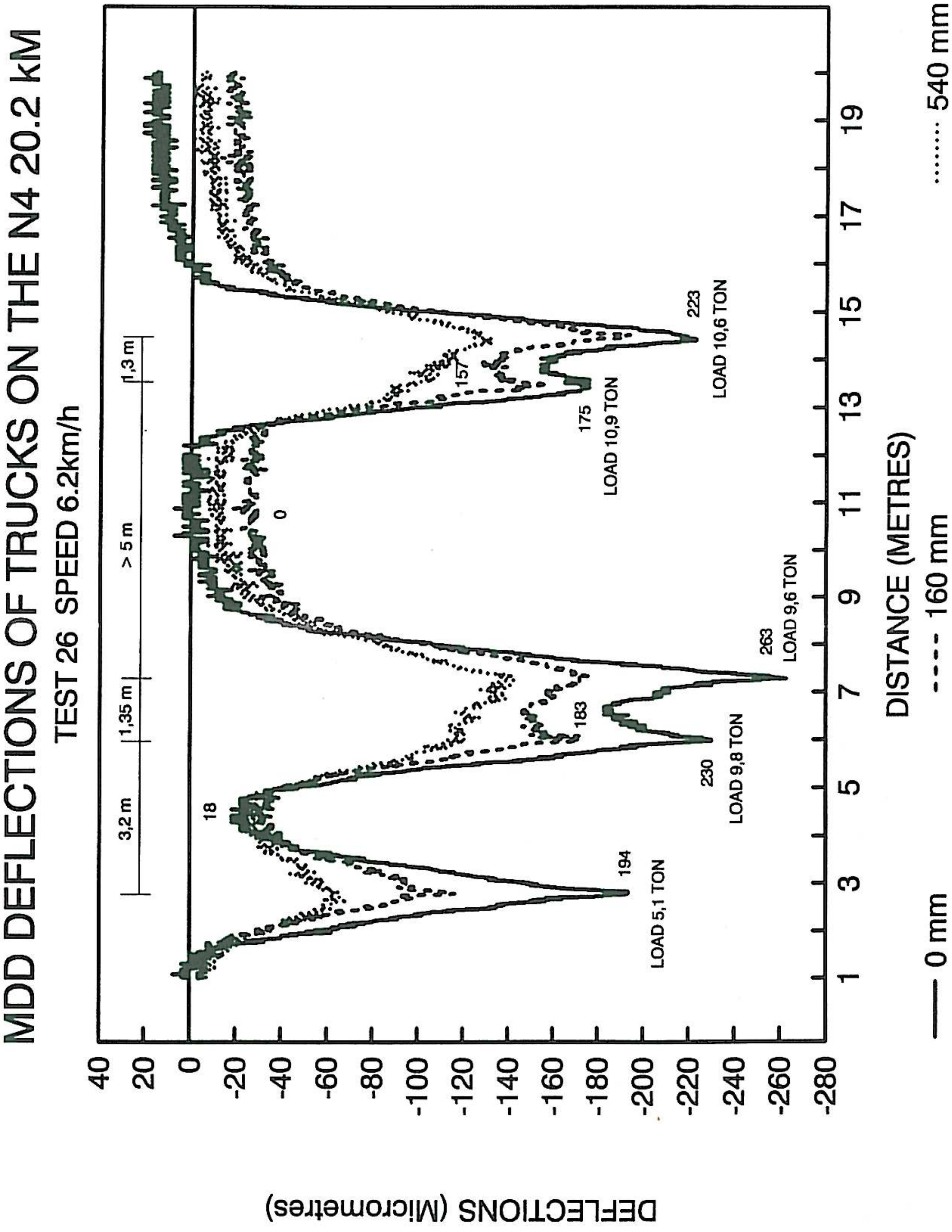


FIGURE A-22

NR26.DRW

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 km

TEST 27 SPEED 4.5 km/h

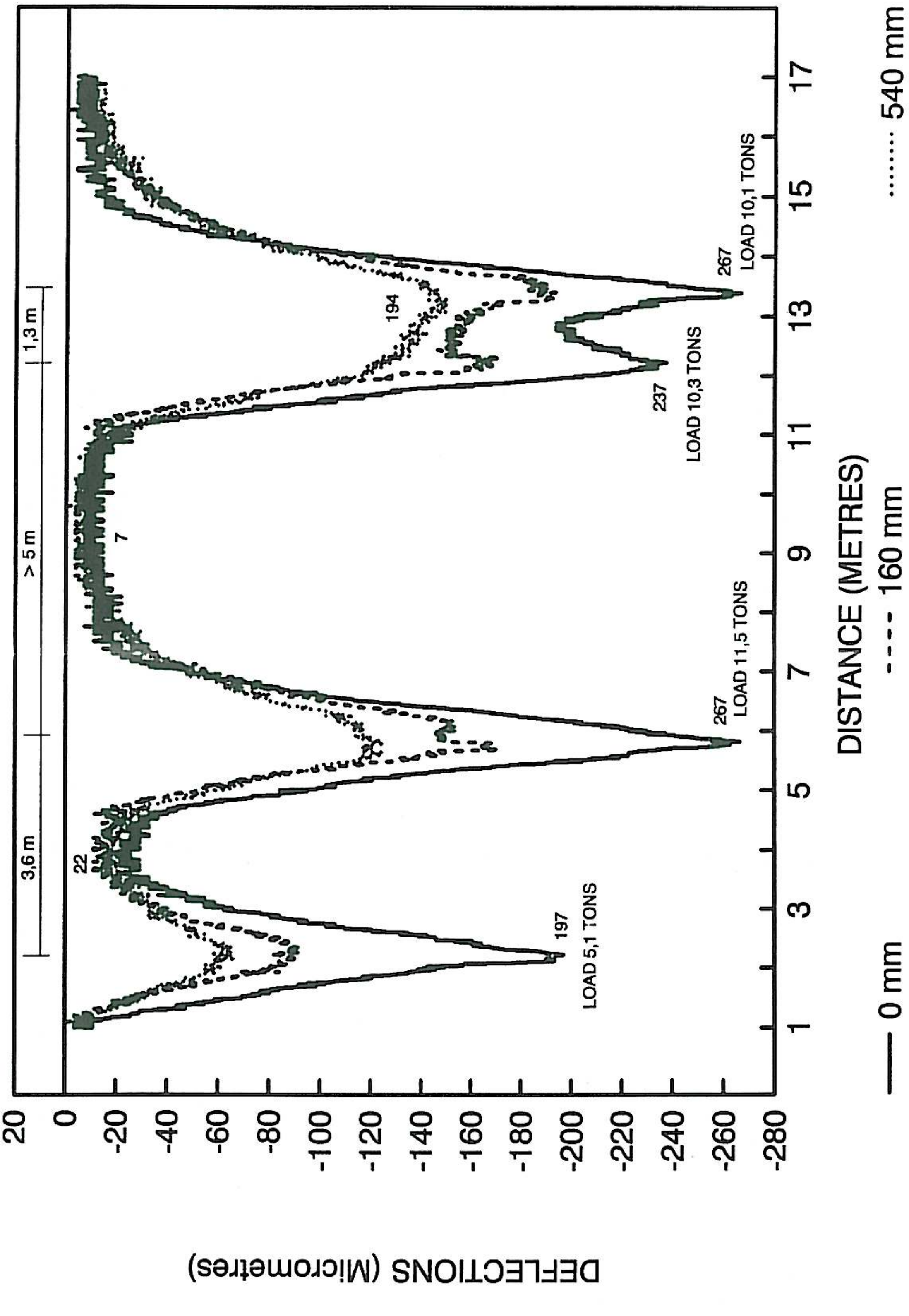
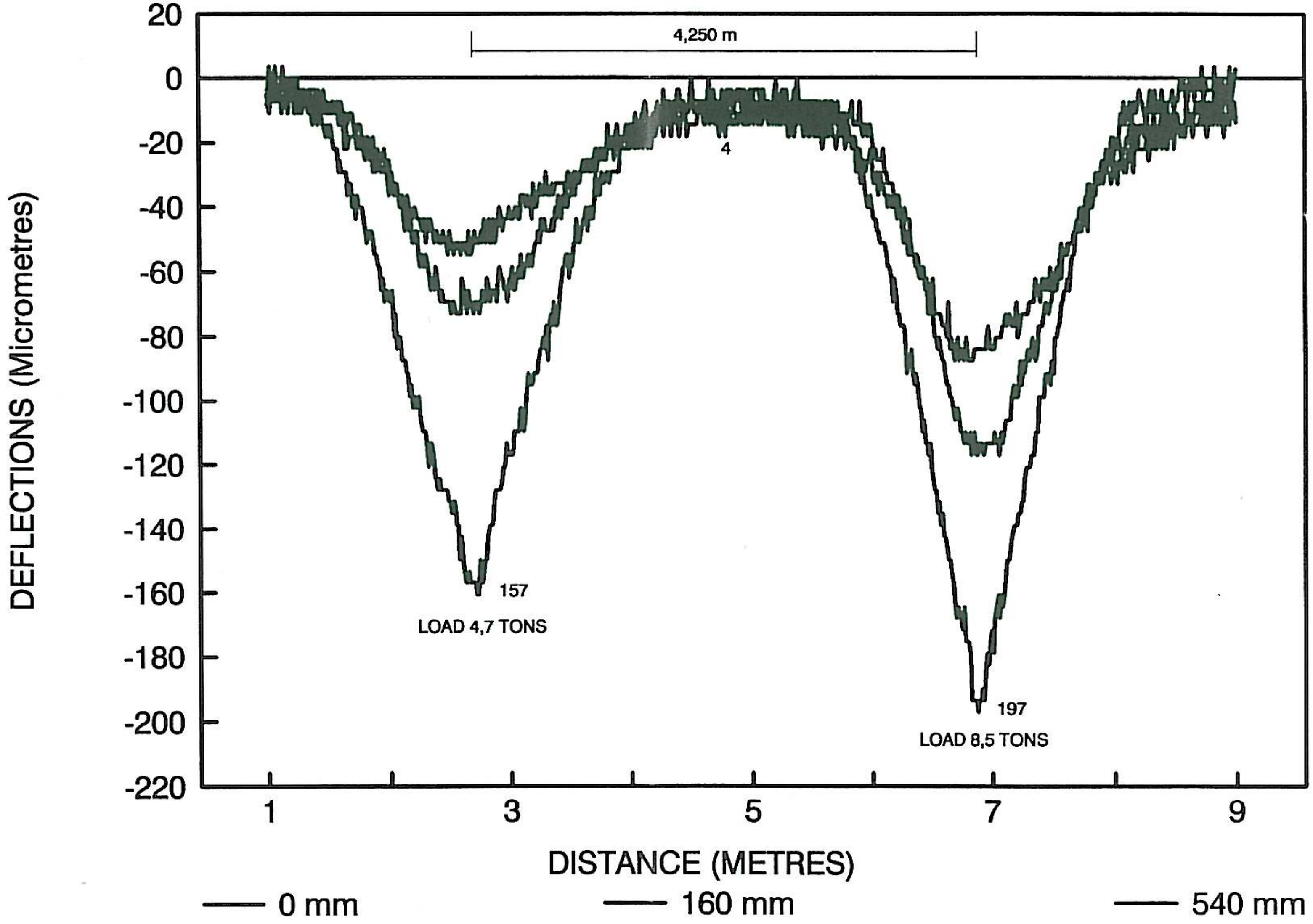


FIGURE A-23

NR27.DRW

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 km

TEST 28 SPEED 4.6 km/h



NR28.DRW

FIGURE A-24

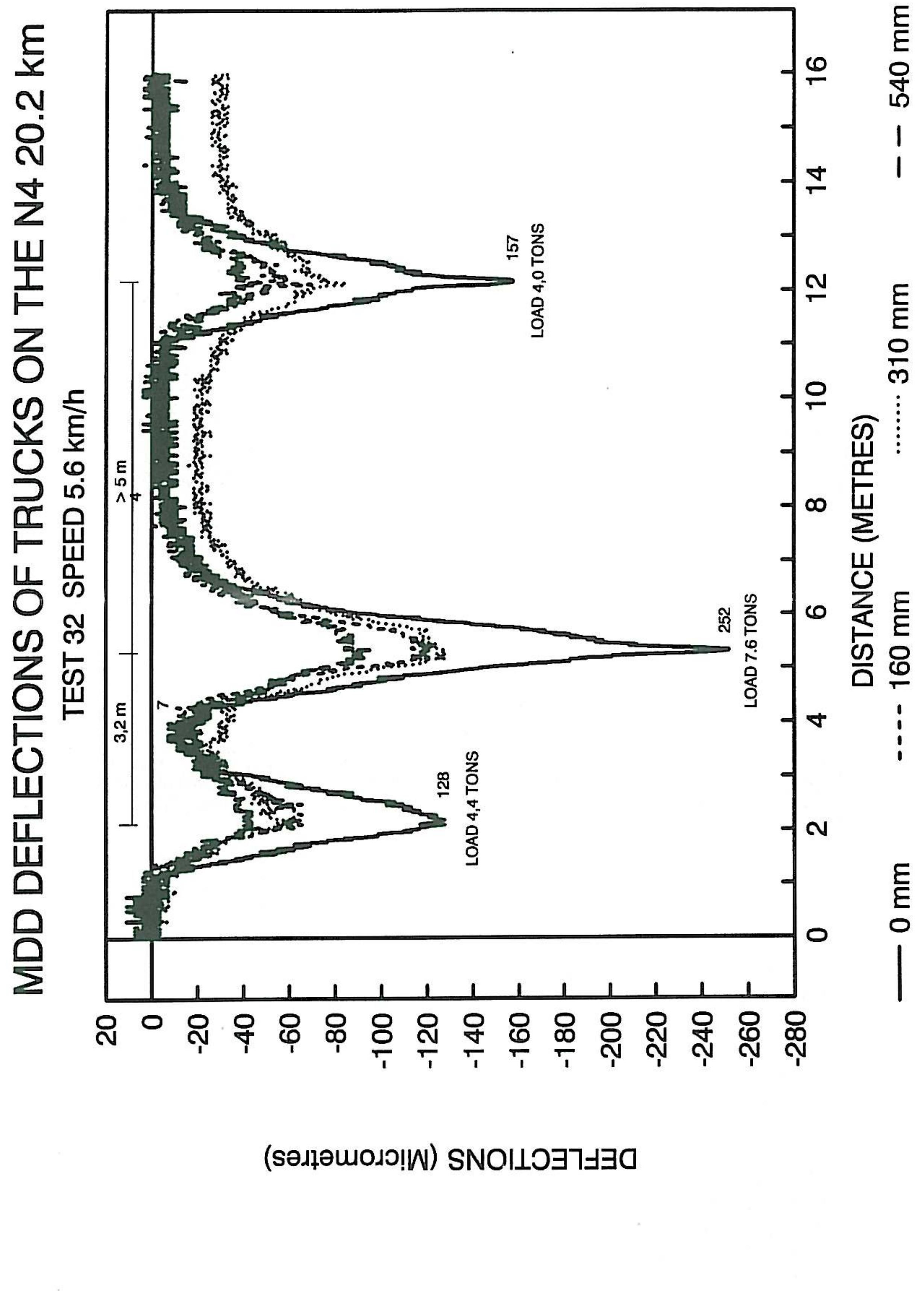
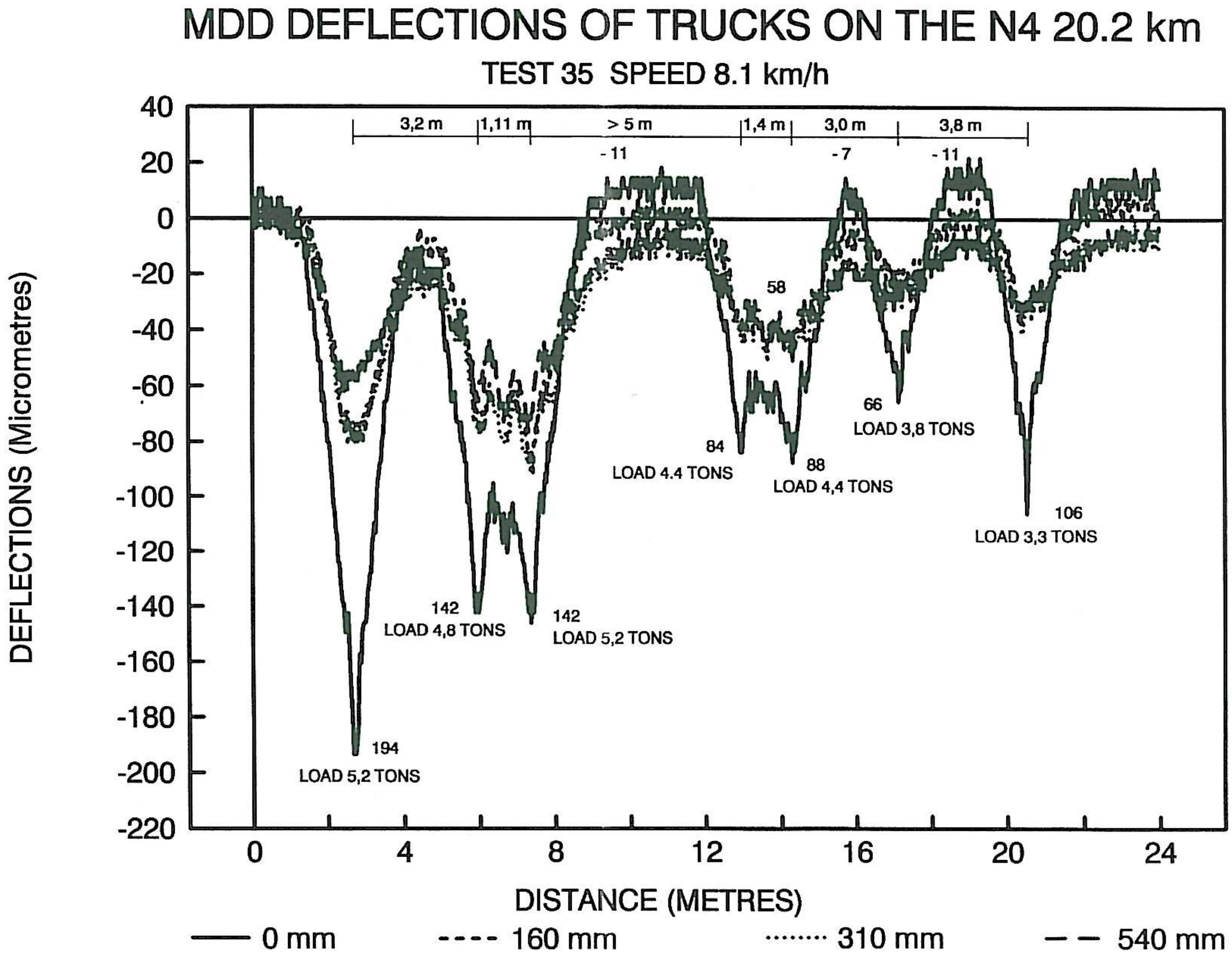


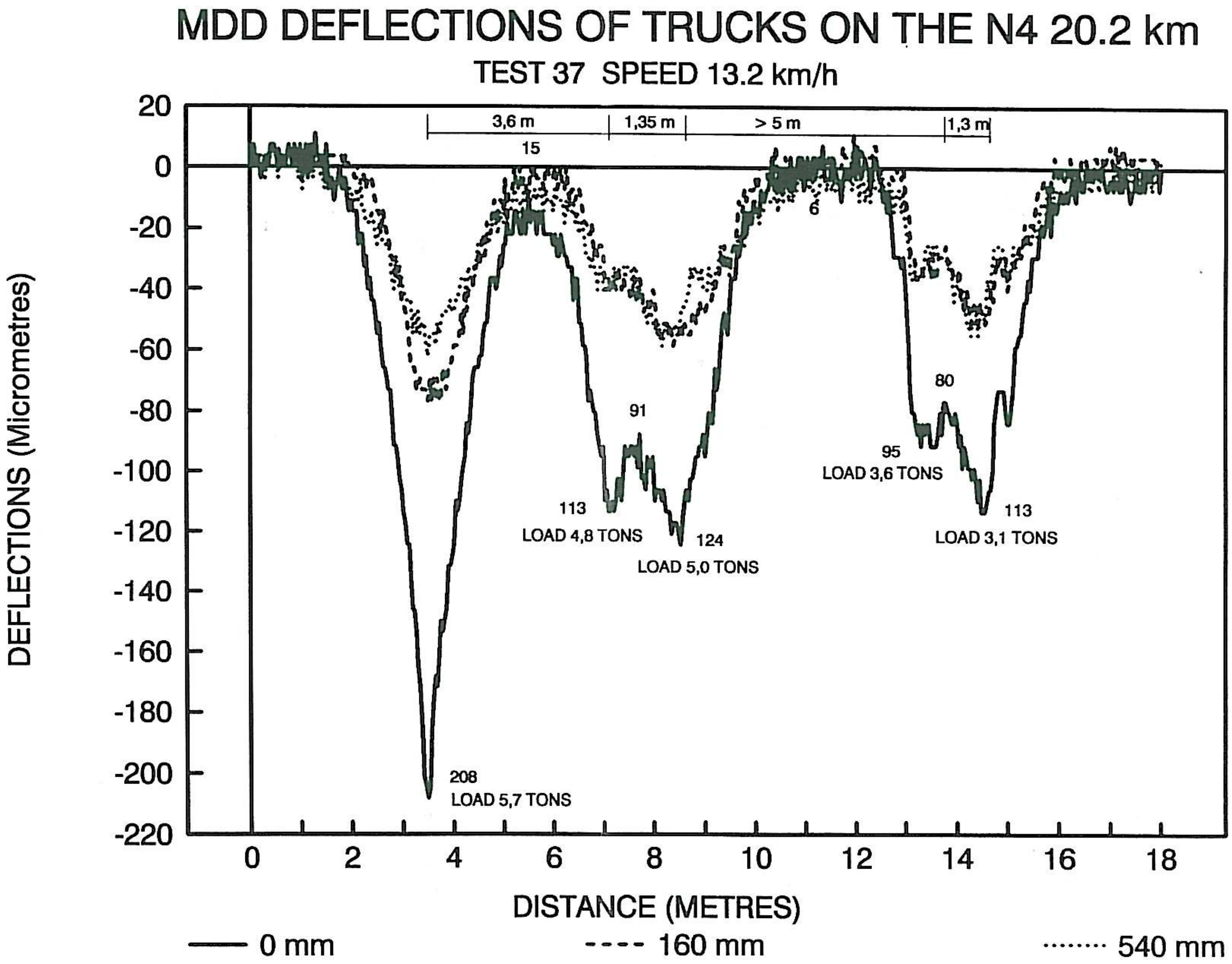
FIGURE A-25

NR32.DRW



NR35.DRW

FIGURE A-26

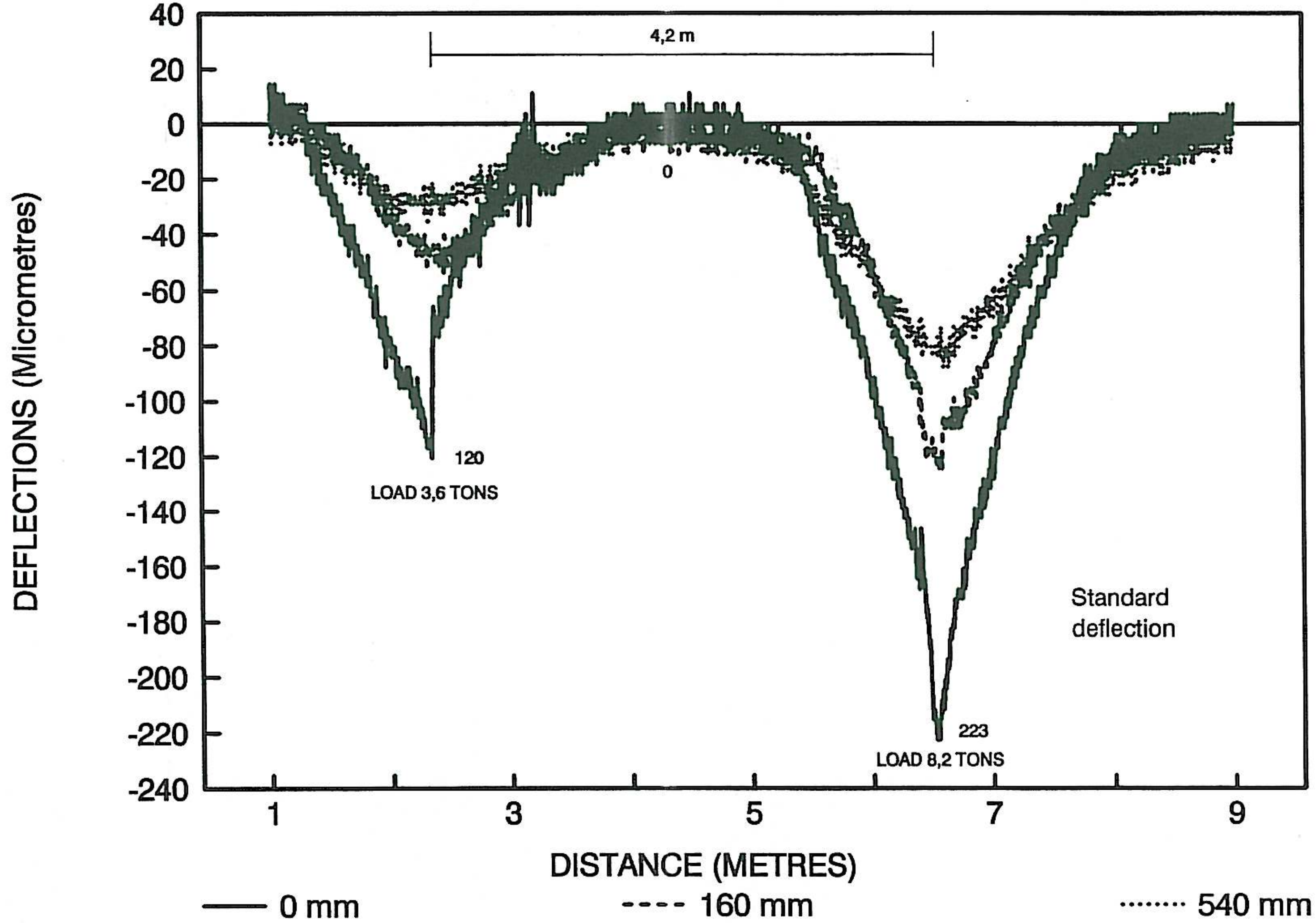


NR37.DRW

FIGURE A-27

MDD DEFLECTIONS OF TRUCKS ON THE N4 20.2 km

TEST 39 SPEED 3.0 km/h (RSD VEHICLE)



NR39.DRW

FIGURE A-28

TABLE A-1
DRTT VEHICLE WEIGHT, DIMENSION AND DEFLECTION STUDY

LOAD EQUIVALENCE FACTORS

ROAD : N4 km 20.0 (WITBANK)
AXLE CONFIGURATION : SINGLE
WHEEL CONFIGURATION : SINGLE (STEERING AXLE)

AXLE LOAD (tons)	AXLE LOAD (KN)	DEFL	AASHO	n=4	n=4,2	d=3	TEST VEHICLE NO.	
2.4	23.52	0.202	0.007	0.007	0.006	0.025	3	
2.8	27.44	0.049	0.011	0.014	0.011	0.04	4	
3.6	35.28	0.247	0.03	0.037	0.032	0.085	2	F\AXLE (RSD)
3.6	35.28	0.158	0.03	0.037	0.032	0.085	39	F\AXLE (RSD)
3.8	37.24	0.333	0.039	0.046	0.04	0.1	10	
4.4	43.12	0.202	0.072	0.083	0.073	0.155	32	
4.5	44.1	0.221	0.08	0.091	0.08	0.165	22	
4.7	46.06	0.439	0.098	0.108	0.097	0.188	28	
4.8	47.04	0.276	0.109	0.117	0.106	0.201	7	
5.1	49.98	0.518	0.142	0.15	0.136	0.241	23	
5.1	49.98	0.981	0.142	0.15	0.136	0.241	26	
5.1	49.98	1.04	0.142	0.15	0.136	0.241	27	
5.2	50.96	0.518	0.153	0.162	0.148	0.255	8	
5.2	50.96	0.981	0.153	0.162	0.148	0.255	35	
5.2	50.96	1.278	0.153	0.162	0.148	0.255	37	
5.8	56.84	1.122	0.246	0.25	0.234	0.354	24	
5.9	57.82	0.621	0.264	0.25	0.234	0.354	25	
5.9	57.82	0.568	0.264	0.268	0.2541	0.373	14	
6	58.8	1.999	0.283	0.287	0.269	0.392	12	
6	58.8	1.221	0.283	0.287	0.269	0.392	18	
6.1	59.78	0.722	0.302	0.306	0.289	0.412	19	
6.2	60.76	1.554	0.321	0.327	0.309	0.432	17	
6.3	61.74	0.722	0.339	0.348	0.331	0.454	11	
6.7	65.66	1.665	0.448	0.446	0.428	0.546	5	
7	68.6	0.836	0.534	0.531	0.515	0.622	16	

TABLE A-2

DRTT VEHICLE WEIGHT, DIMENSION AND DEFLECTION STUDY

LOAD EQUIVALENCE FACTORS

ROAD : N4 km 20.0 (WITBANK)
AXLE CONFIGURATION : SINGLE
WHEEL CONFIGURATION : DUAL

AXLE LOAD (tons)	AXLE LOAD (kN)	DEFL	AASHO	n=4	n=4,2	d=3	TEST VEHICLE NO.	WHEEL SPACING (mm)
3.1	30.38	0.099	0.018	0.02	0.017	0.054	3	280
3.9	38.22	0.088	0.045	0.051	0.044	0.108	4	280
4	39.2	0.398	0.05	0.057	0.049	0.116	32	350
4.6	45.08	0.786	0.087	0.099	0.088	0.177	7	320
4.8	47.04	0.202	0.109	0.117	0.106	0.201	23	330
5.2	50.96	0.227	0.153	0.162	0.148	0.255	8	350
5.2	50.96	0.247	0.153	0.162	0.148	0.255	10	340
5.5	53.9	0.221	0.18	0.202	0.187	0.302	10	340
5.8	56.84	0.247	0.246	0.25	0.234	0.354	22	320
7.2	70.56	0.276	0.591	0.594	0.579	0.677	25	310
8.2	80.36	1.000	1.000	1.000	1.000	1.000	std axle config load *	330
8.3	81.34	2.381	1.077	1.05	1.052	1.037	32	350
8.5	83.3	0.962	1.198	1.155	1.163	1.114	28	290
11.5	112.7	2.381	4.225	3.868	4.139	2.758	27	350
13.5	132.3	4.008	8.349	7.346	8.116	4.462	12	330

* Std. deflection = Average deflection measured under RSD vehicle

TABLE A-3

DRTT VEHICLE WEIGHT, DIMENSION AND DEFLECTION STUDY

LOAD EQUIVALENCE FACTORS

ROAD : N4 km 20.0 (WITBANK)
AXLE CONFIGURATION : TANDEM
WHEEL CONFIGURATION : DUAL

AXLE LOAD (tons)	AXLE LOAD (kN)	DEFL	AASHO	n=4	n=4,2	d=3	TEST VEHICLE NO.	WHEEL SPACING (mm)	AXLE SPACING (m)
5.4	52.92	0.055	0.01	0.017	0.019	0.071	23	340	1.3
6	58.8	0.03	0.022	0.026	0.03	0.098	14	360	1.4
6.7	65.66	0.052	0.038	0.04	0.048	0.139	37	350	1.3
7.1	69.58	0.056	0.046	0.05	0.06	0.162	8	350	1.3
8	78.4	0.1	0.074	0.081	0.099	0.233	14	360	1.3
8.9	87.22	0.252	0.112	0.124	0.154	0.32	19	360	1.35
9.8	96.04	0.074	0.16	0.167	0.231	0.427	37	350	1.35
9.9	97.02	0.223	0.17	0.19	0.244	0.443	14	360	1.35
12.5	122.5	0.341	0.449	0.482	0.646	0.89	19	360	1.35
16.9	165.62	1.473	1.591	1.612	2.295	2.2	18	350	1.35
17.4	170.52	0.73	1.79	1.811	2.574	2.392	5	360	1.35
18.7	183.26	1.712	2.425	2.416	3.621	3.022	5	360	1.3
19.4	190.12	1.408	2.829	2.798	4.053	3.312	26	350	1.35
20.4	199.92	1.897	3.488	3.421	5.001	3.85	27	350	1.3
20.8	203.84	3.548	3.775	3.698	5.457	4.09	18	350	1.35
21.1	206.78	1.686	4.014	3.916	5.771	4.262	11	350	1.35
21.1	206.78	1.284	4.014	3.916	5.869	4.294	17	350	1.3
21.5	210.7	0.679	4.344	4.22	6.245	4.509	26	350	1.3
21.6	211.68	1.365	4.427	4.3	6.374	4.574	17	350	1.35
22.7	222.46	0.772	5.416	5.245	7.86	5.311	16	360	1.35
24.2	237.16	3.226	7.03	6.774	10.254	6.427	11	360	1.3
25.3	247.94	2.815	8.427	8.094	12.459	7.371	16	360	1.3

Appendix B - Pavement structures used for the mechanistic analysis in Section 3

1. INTRODUCTION

The purpose of the mechanistic analysis was to supplement and/or verify the findings of the field study, which was based on results obtained from a single test. Four pavement section were chosen, i.e. two pavements with granular base and subbase layers (Section 1 and 4), a pavement with a granular base and a cemented subbase layer (Section 2), and a pavement with a cemented base and subbase layer (Section 3). The pavement structures are shown in Figure B-1 and represent typical pavement structures generally encountered in South Africa. A brief outline of the mechanistic analysis is given in the following paragraphs. The results of the analysis were presented and discussed earlier and are therefore not included in this appendix.

2. SELECTED PAVEMENT SECTIONS

The detail of the selected pavement structures are shown in Figure B-1. The materials used in the respective pavement layers are classified according to the TRH4 classification of pavement materials (CSRA, 1989)).

3. MECHANISTIC ANALYSIS

3.1 METHODOLOGY

A multi-layer linear elastic computer programme (ELSYM5) was used for the analysis. The respective surface deflections under a standard axle load and axle configuration, i.e. a 80 kN dual-wheel single axle (spacing of 360 mm centre to centre between wheels), were calculated for each of the pavement structures shown in Figure B-1. The corresponding axle load on a single axle with single wheels which resulted in the same maximum deflection (surface deflection) as that caused under the standard axle was then determined by means of an iteration process. For the case of the tandem axle arrangement (centre to centre spacings of wheels and axles of 360 mm and 1,4 m respectively), the calculation process differed slightly as the effect of the second axle also needed to be incorporated. In this instance, the load, with concomitant deflections which resulted in a calculated LEF of 1 (from Equation 6 in Appendix A), was determined by means of an iteration process. The subsequent results were presented and discussed earlier.

3.2 INPUT PARAMETERS

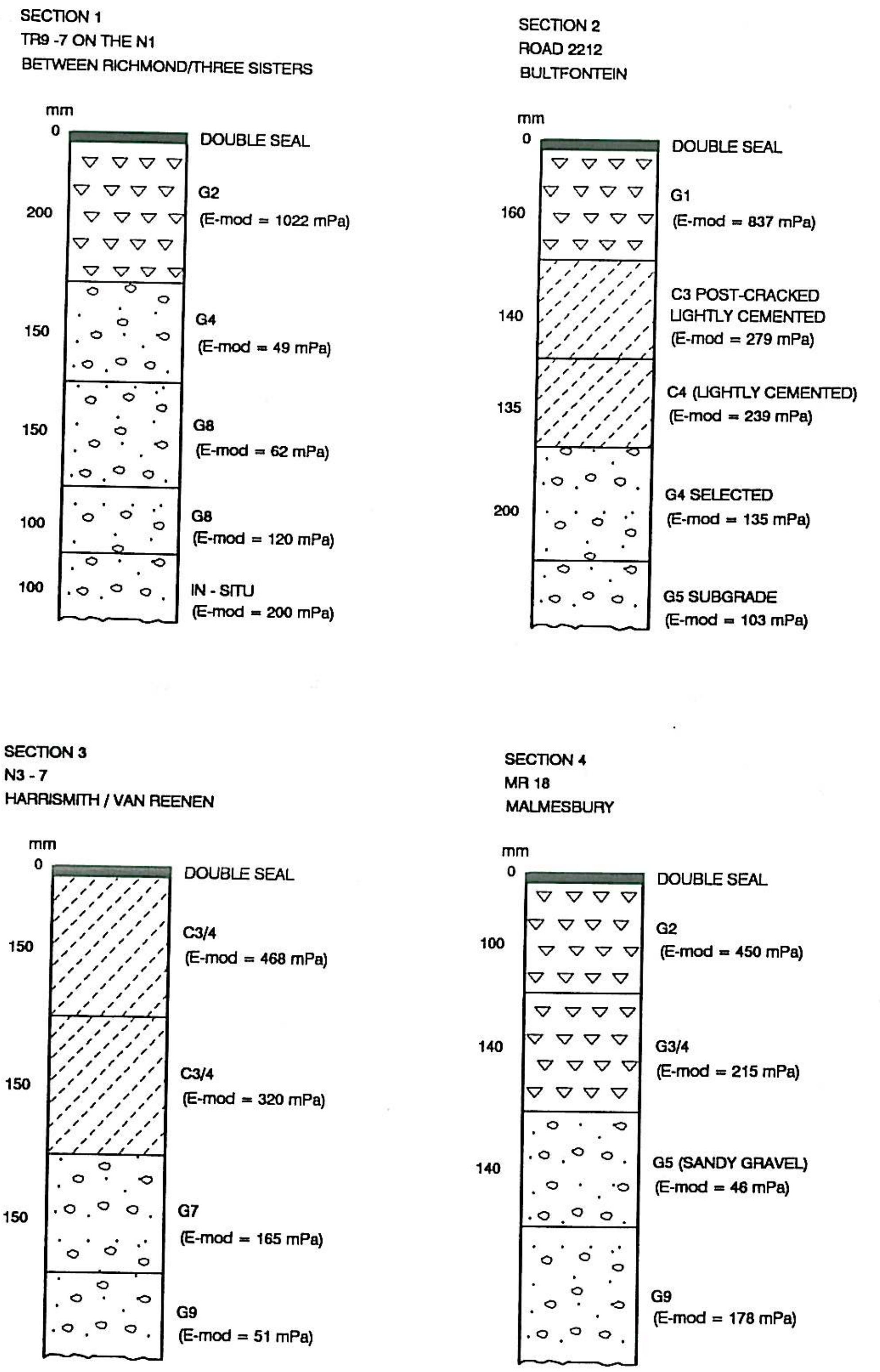
The input parameters required for the analysis, mainly consisted of:

- Layer data - number of layers and the respective layer thicknesses
- Material data - modulus of elasticity (E-mod) and Poisson's ratio (υ) for each layer
- Load data - wheel load and wheel configuration

The load data depended on the individual cases evaluated and was chosen to conform to the respective cases evaluated during the field study, i.e. single axles with single wheels, single axles with dual wheels (wheel spacing of 360 mm), or tandem axles with dual wheels (axle spacings of 1,4 m and wheel spacings of 360 mm). The layer data (number of layers and layer thicknesses) and the material data (E-modulus and Poisson's ratio) which were chosen for the respective pavement layers and used for the analysis, are given in Figure B-1 and were obtained from previous HVS test results on the pavement structures.

4. REFERENCES (APPENDIX B)

1. COMMITTEE OF STATE ROAD AUTHORITIES (CSRA) (1989). TRH4: 1985, Structural design of interurban and rural road pavements. Department of Transport, Pretoria, South Africa.



NOTE : Poisson Ratio (v) of 0,35 used for all pavement layers

FIGURE B - 1

SELECTED PAVEMENT SECTIONS USED FOR THE MECHANISTIC ANALYSIS

Appendix C - Graphic presentation of results from Section 4

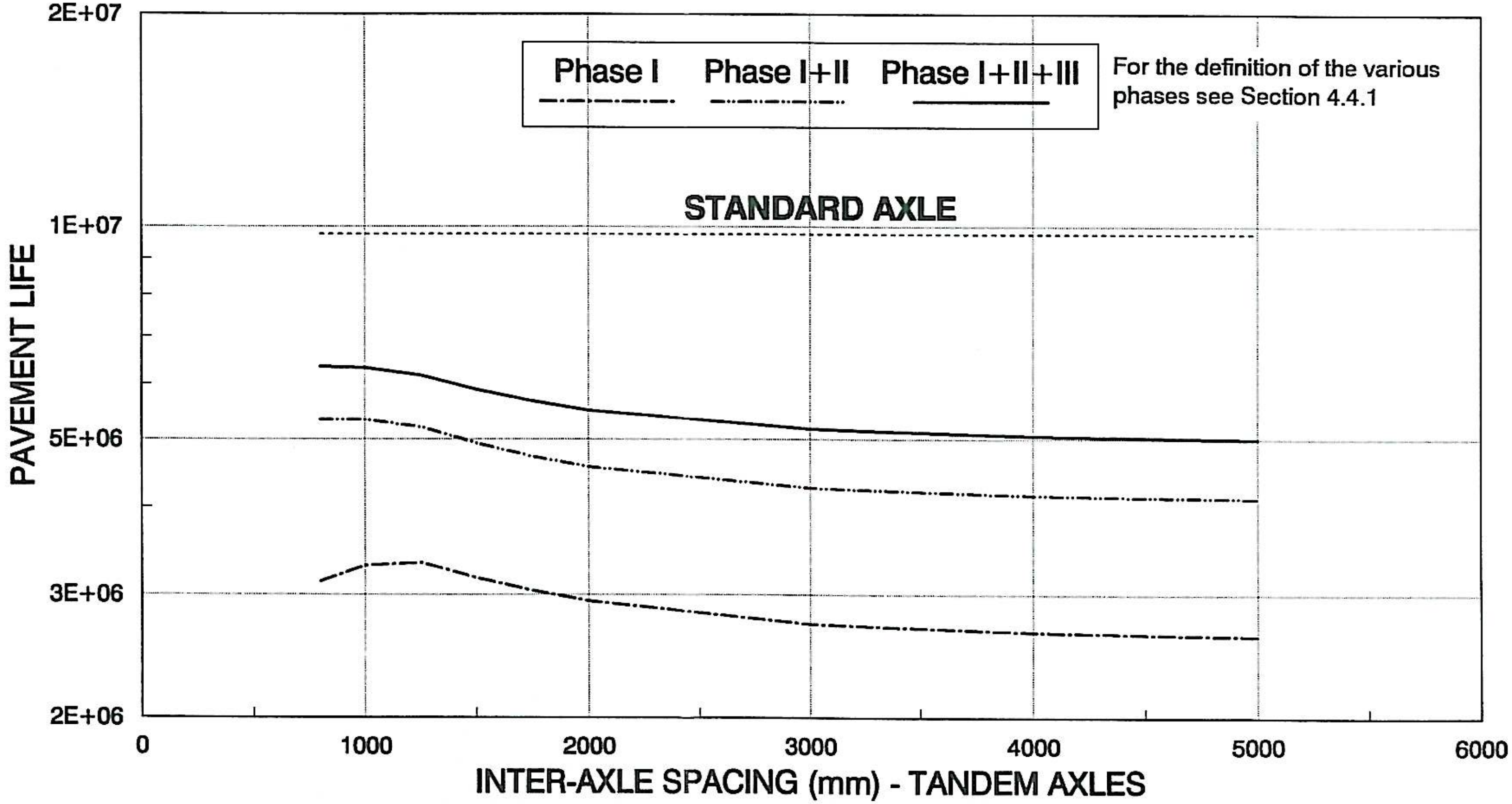


FIGURE C1: Calculated pavement life for different inter-axle spacing for tandem axles on Pavement A (granular base-dry)

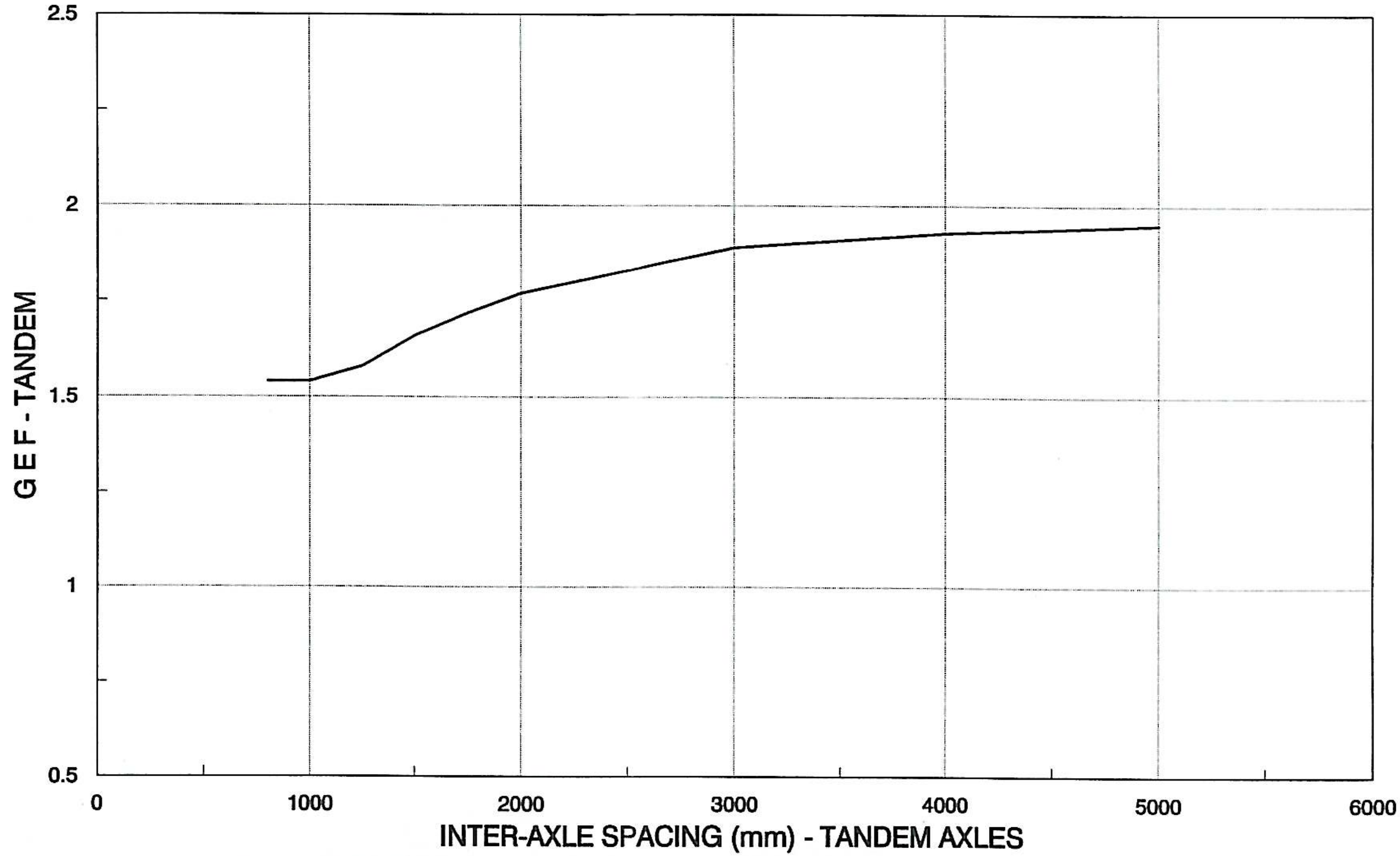


FIGURE C2: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tandem axles on Pavement A

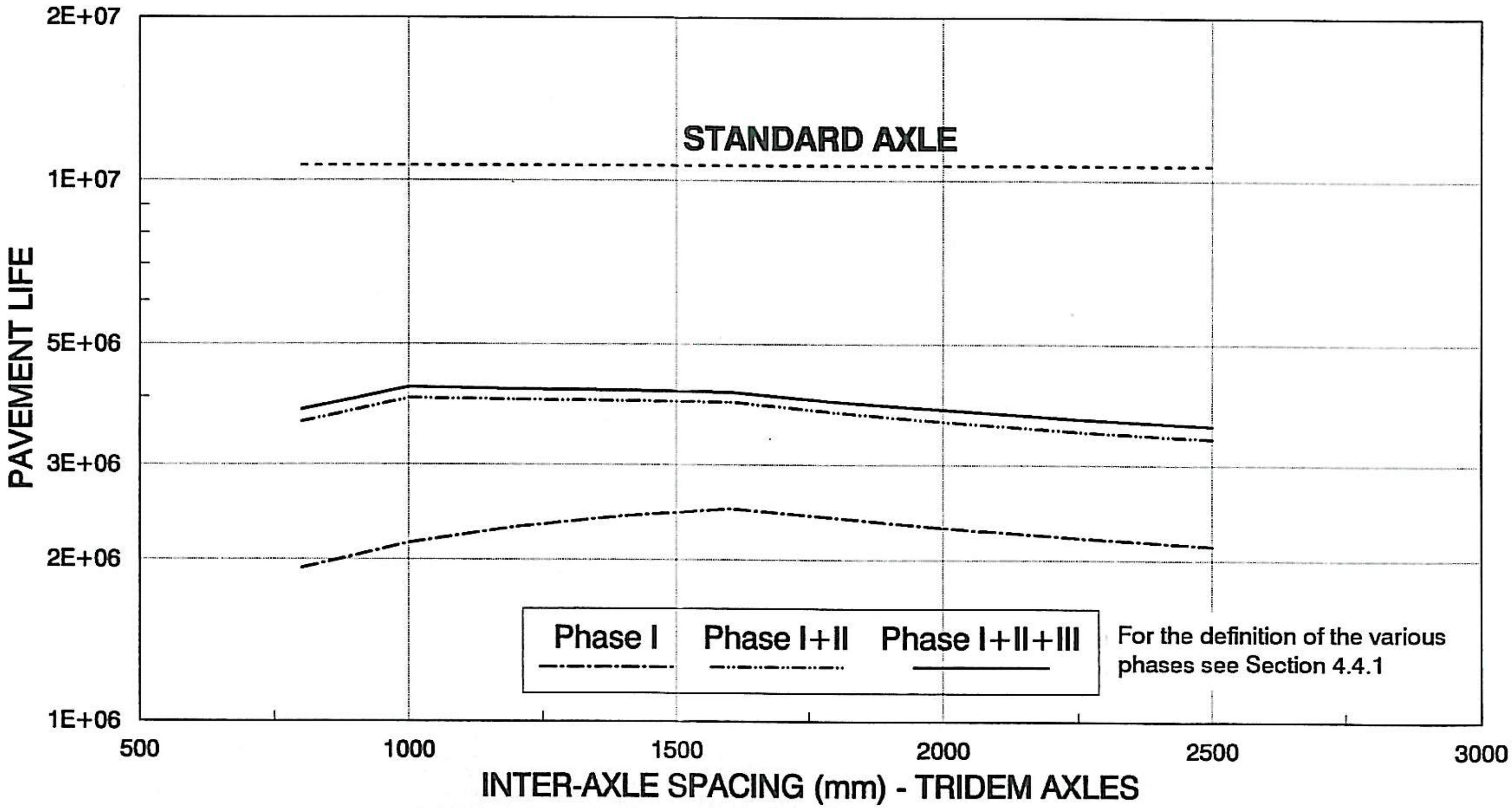


FIGURE C3: Calculated pavement life for different inter-axle spacing for tridem axles on Pavement A (granular base-dry)

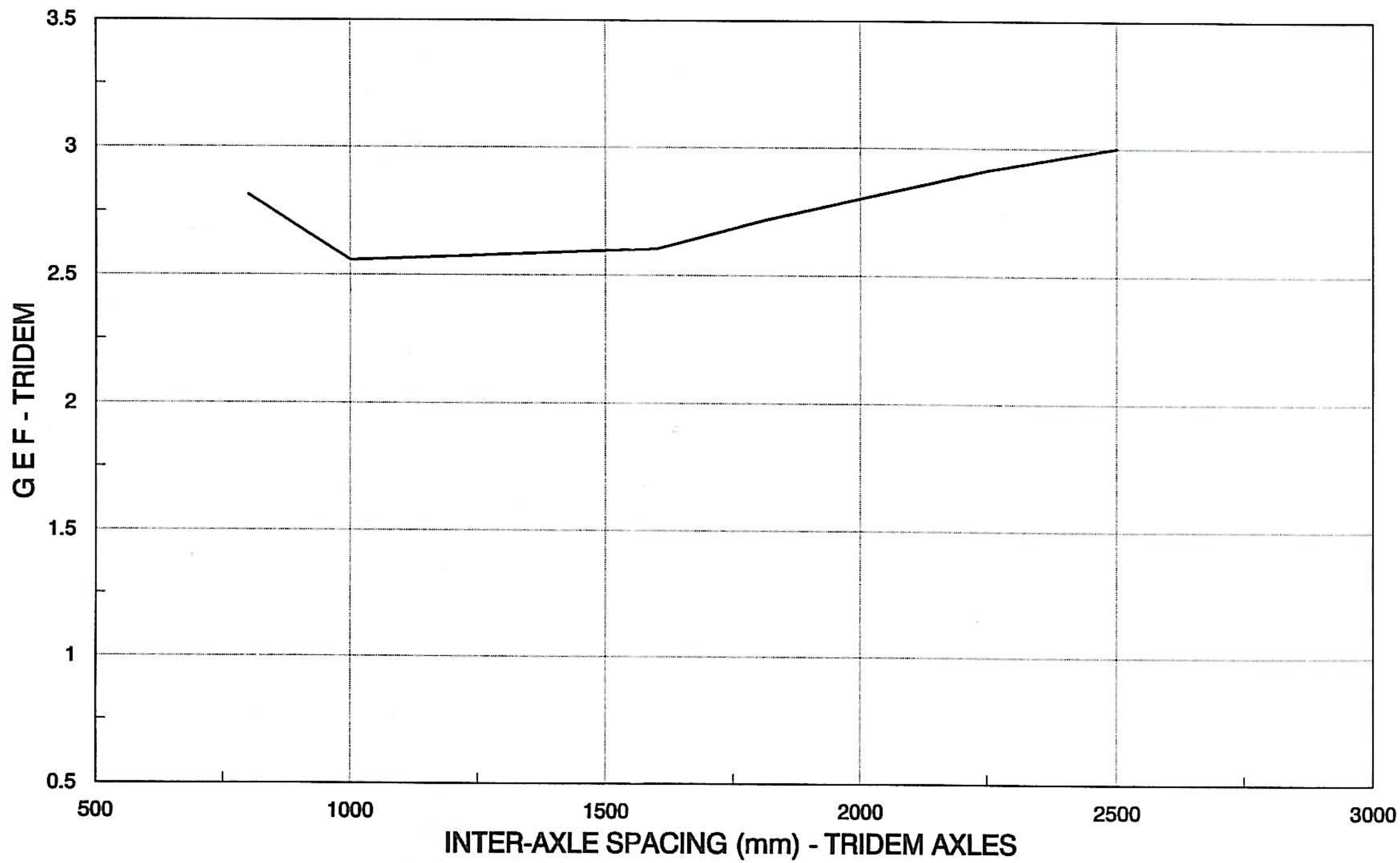


FIGURE C4: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tridem axles on Pavement A

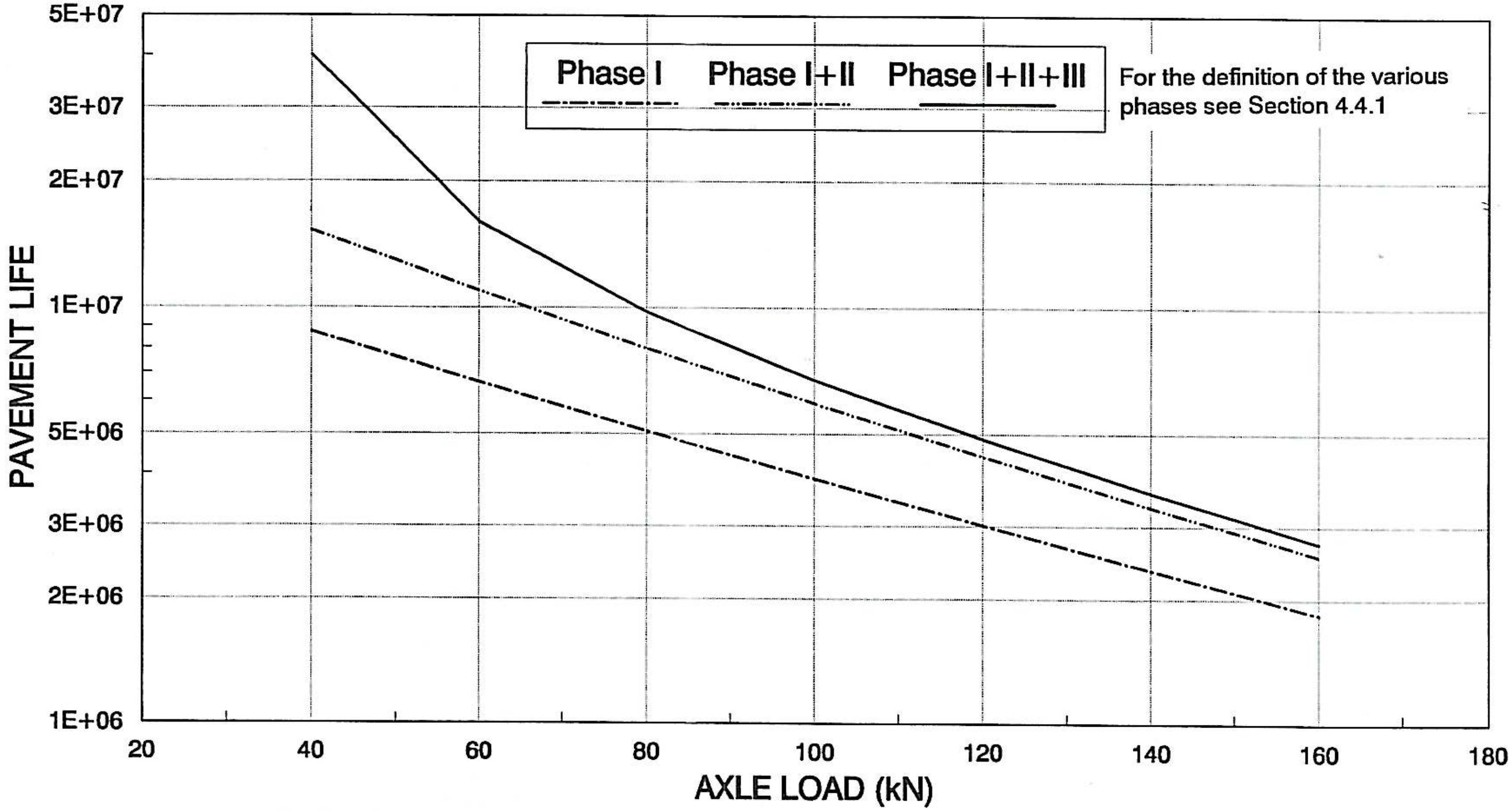


FIGURE C5: Calculated pavement life under a standard dual-wheel axle for different axle loads on Pavement A

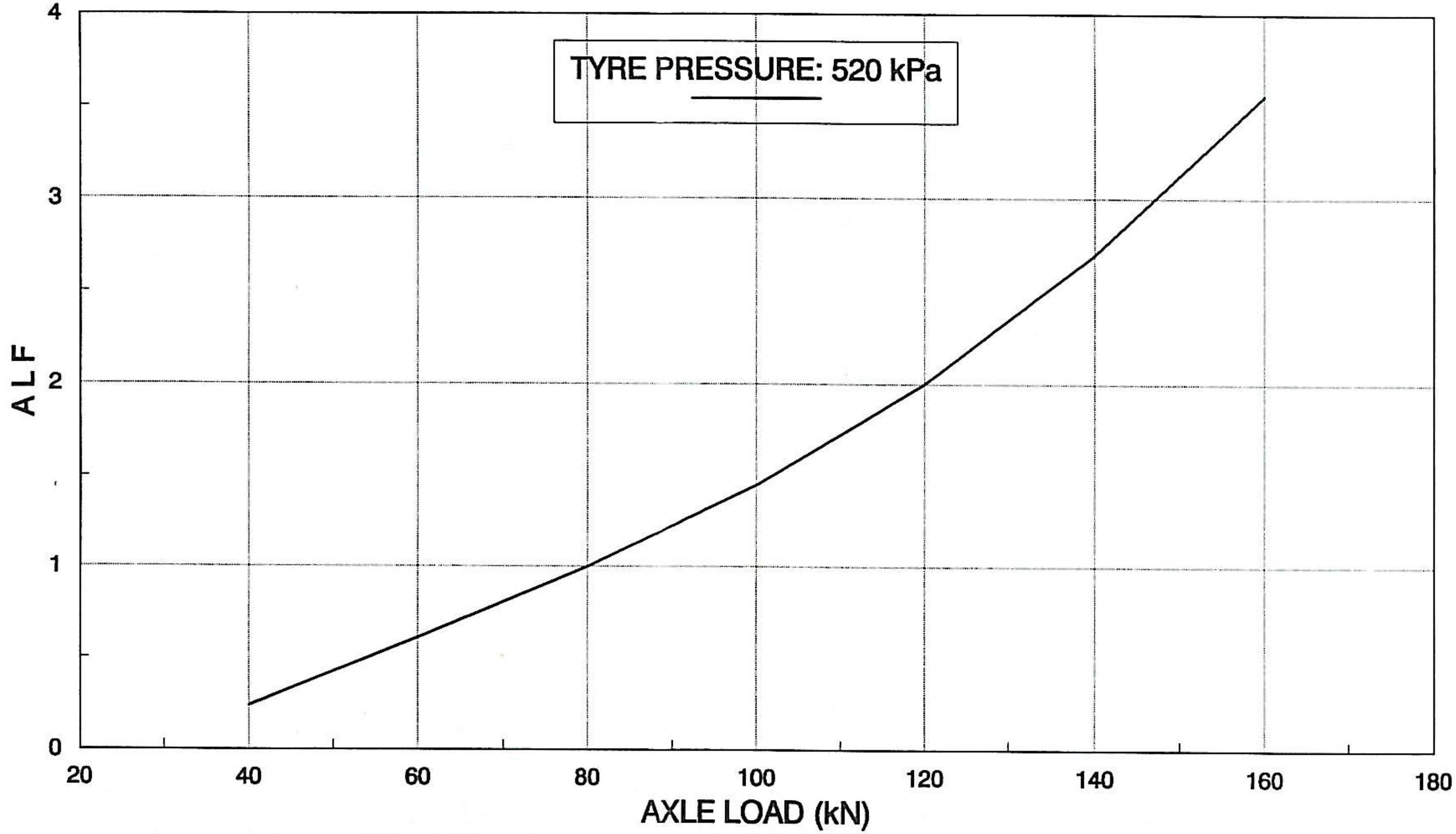


FIGURE C6: Calculated Axle Equivalent Factors (ALF) for different axle loads on Pavement A

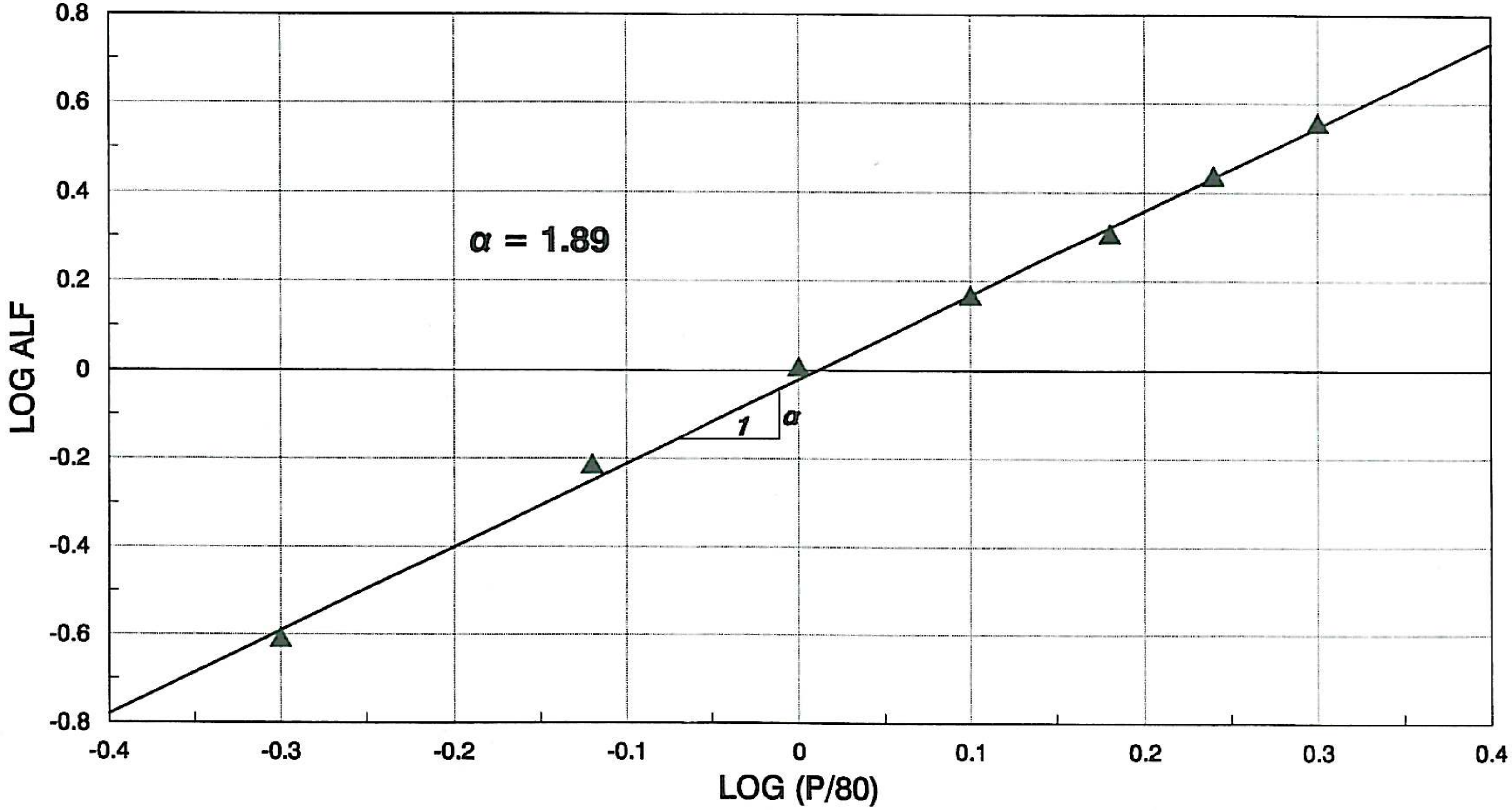


FIGURE C7: Load damage coefficient 'α' for Pavement A as determined by regression analysis of the calculated Axle Load Factors (ALF)

FIGURE C8: Calculated pavement life for a single wheel axle configuration on Pavement A

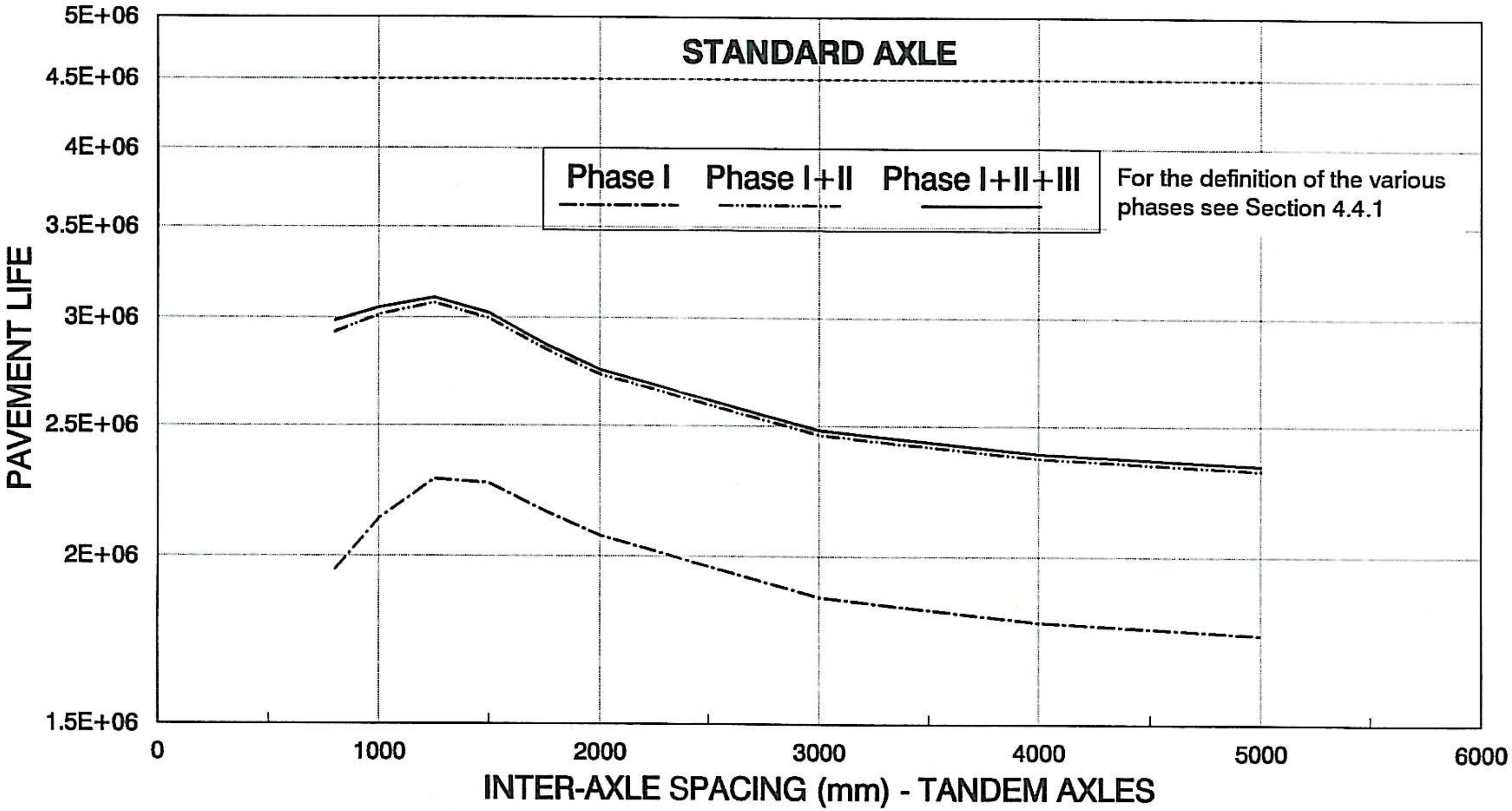


FIGURE C9: Calculated pavement life for different inter-axle spacing for tandem axles on Pavement B (granular base-wet)

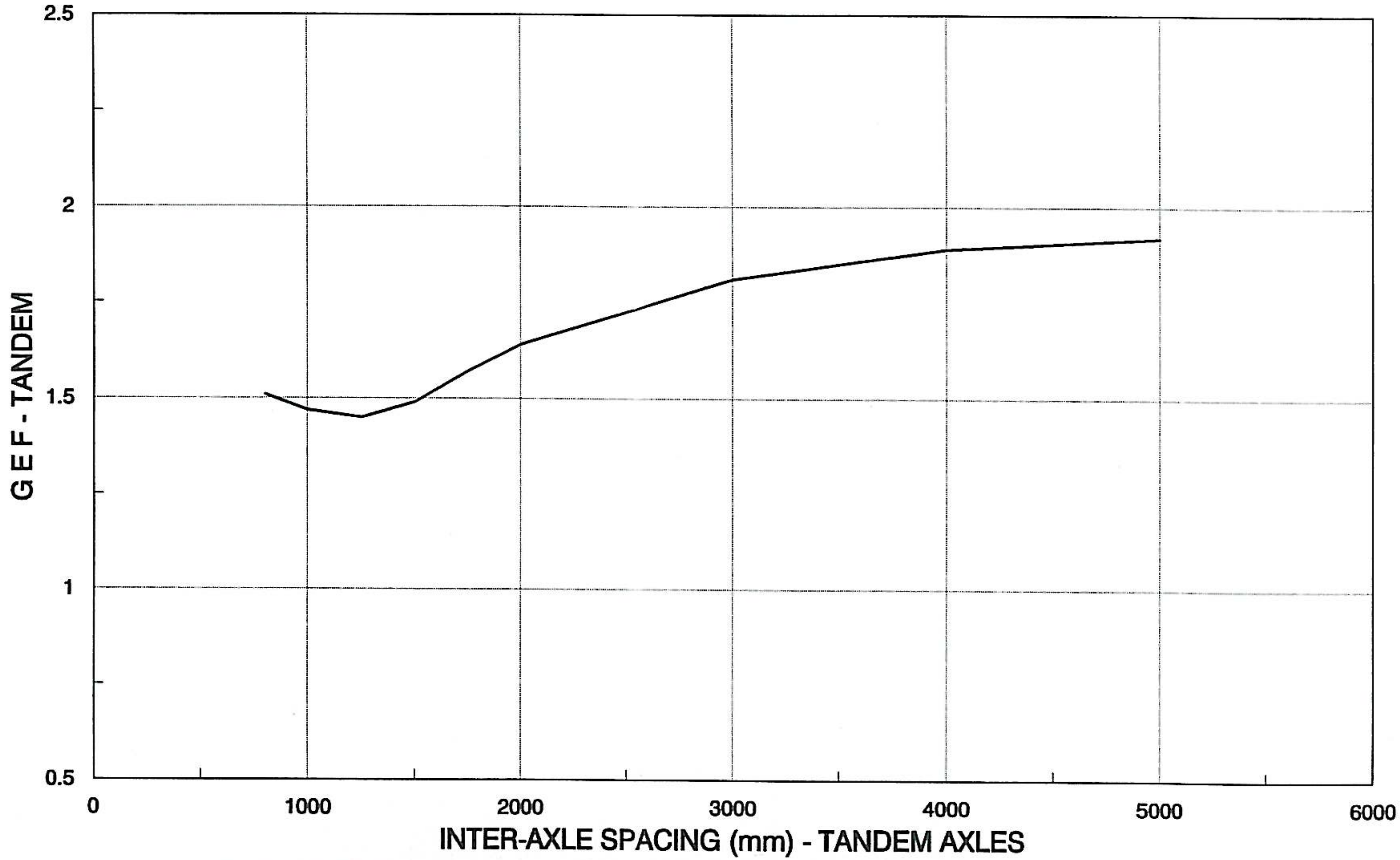


FIGURE C10: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tandem axles on Pavement B

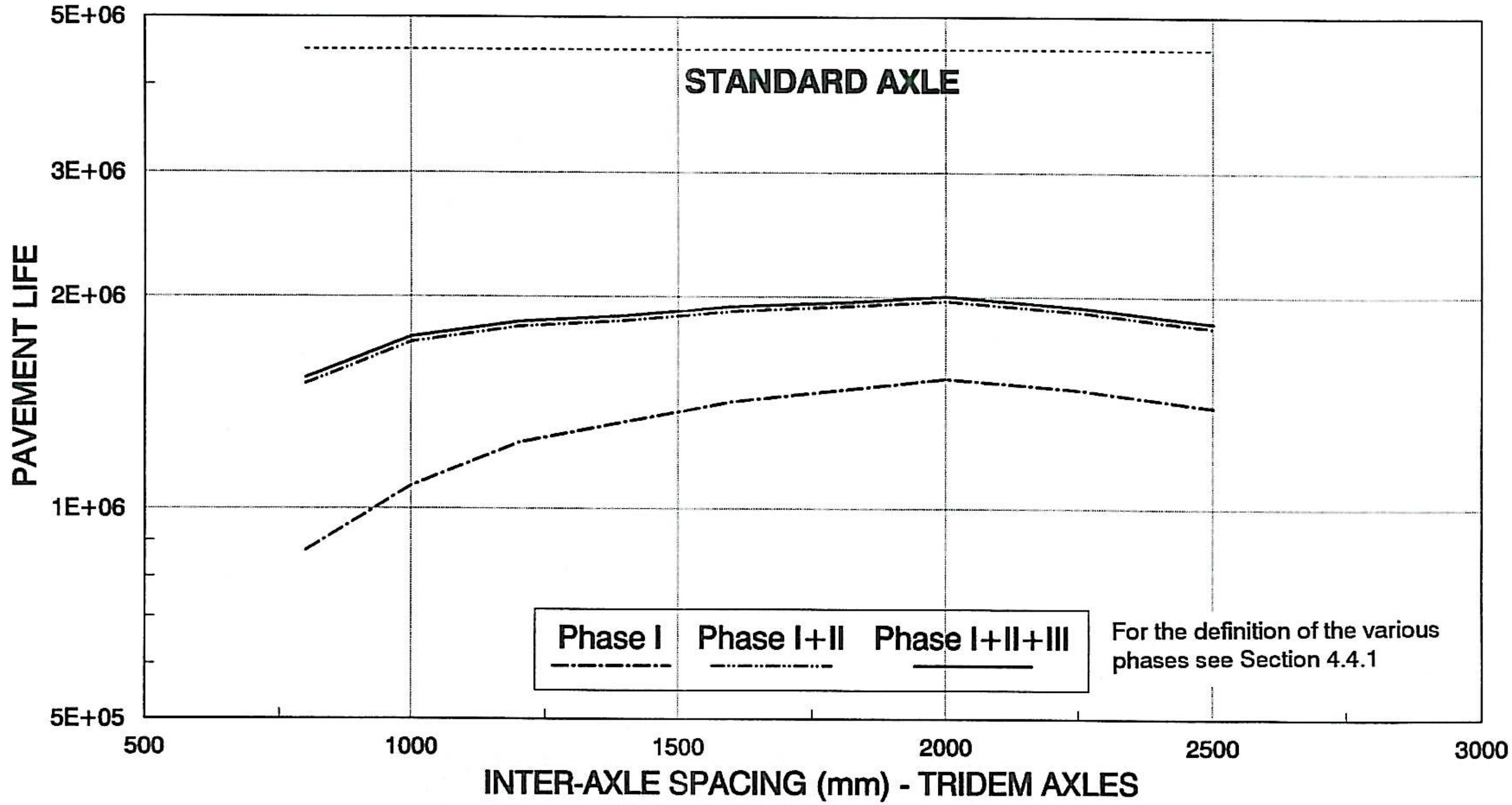


FIGURE C11: Calculated pavement life for different inter-axle spacing for tridem axles on Pavement B (granular base-wet)

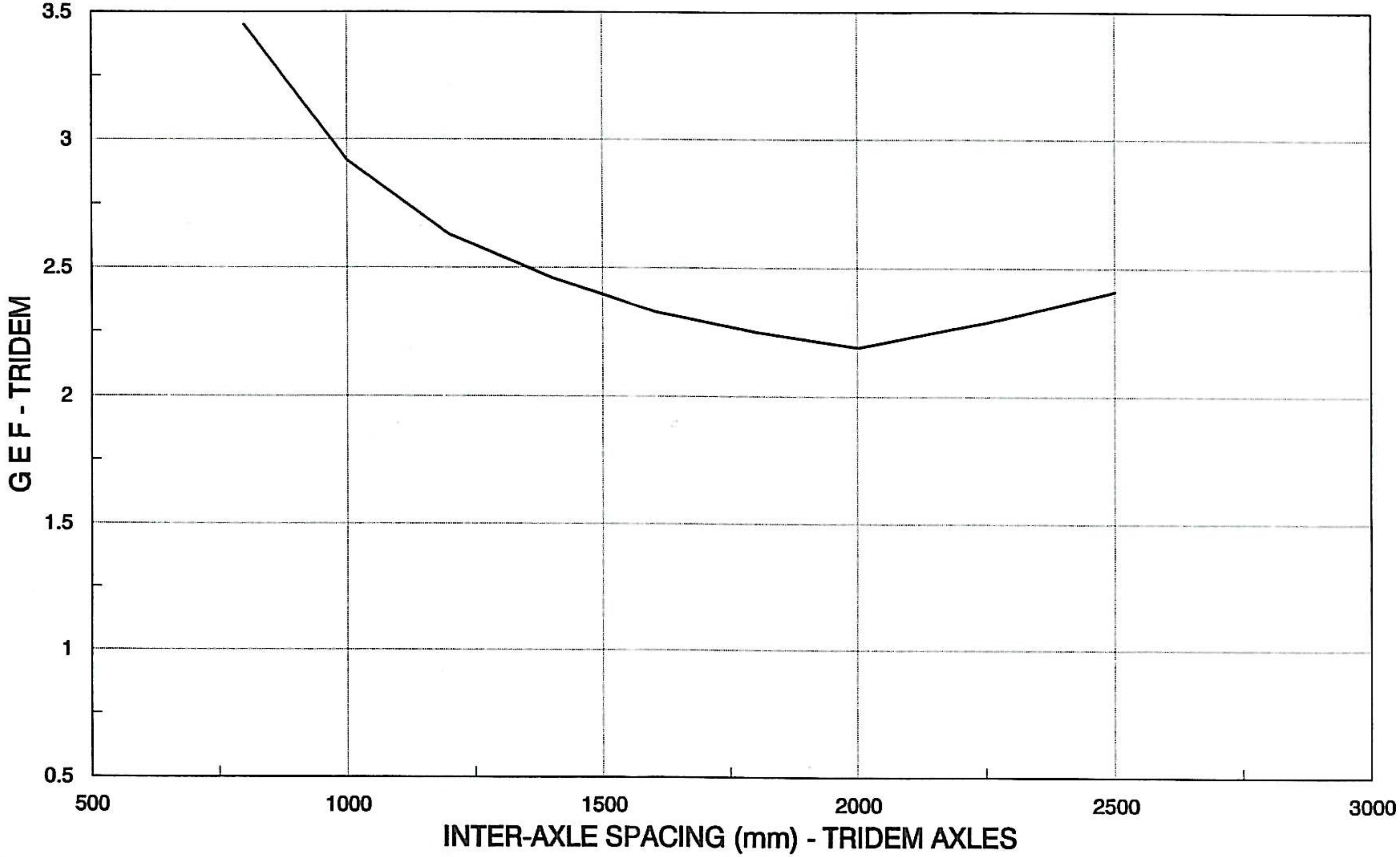


FIGURE C12: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tridem axles on Pavement B

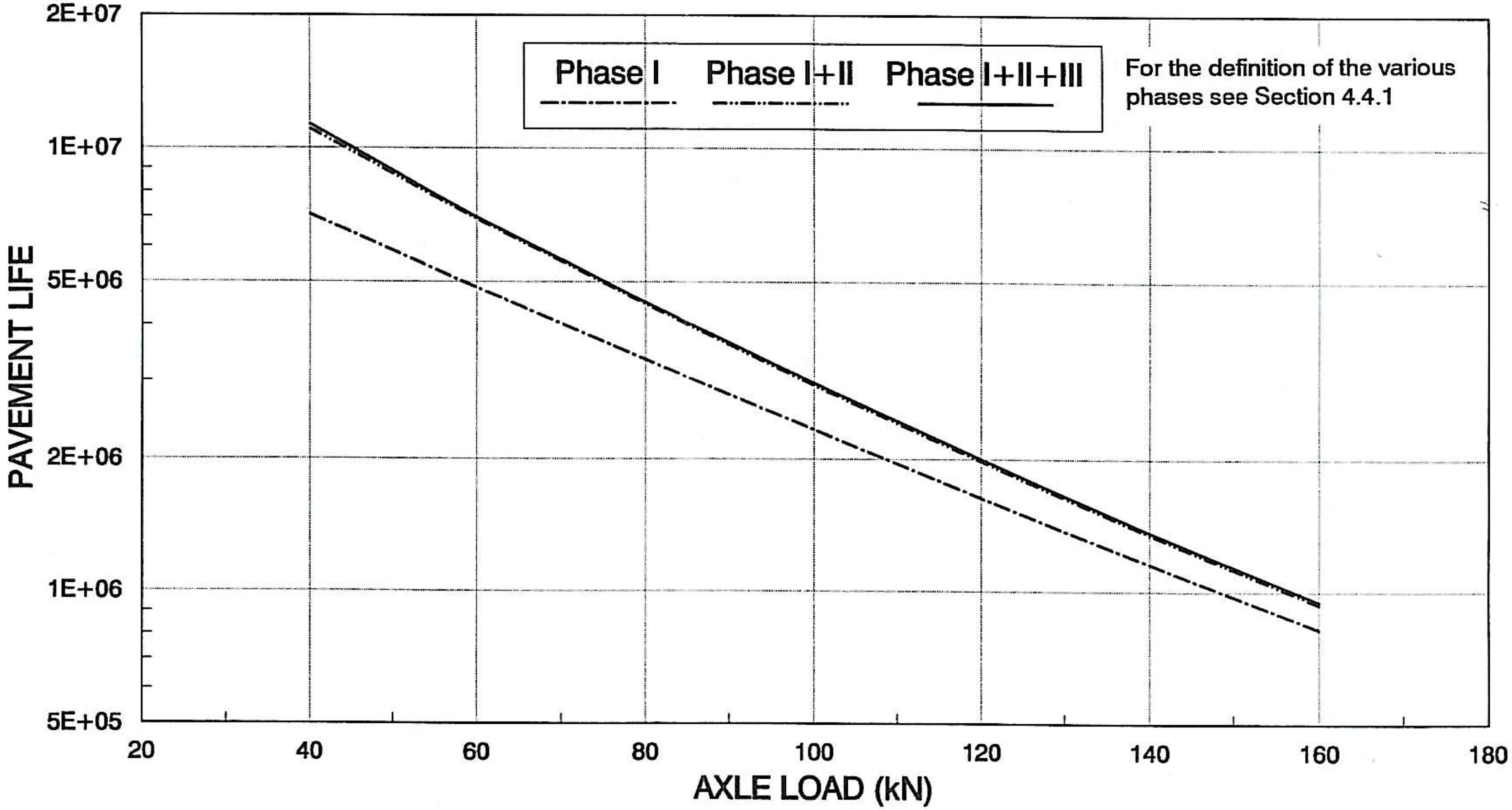


FIGURE C13: Calculated pavement life under a standard dual-wheel axle for different axle loads on Pavement B

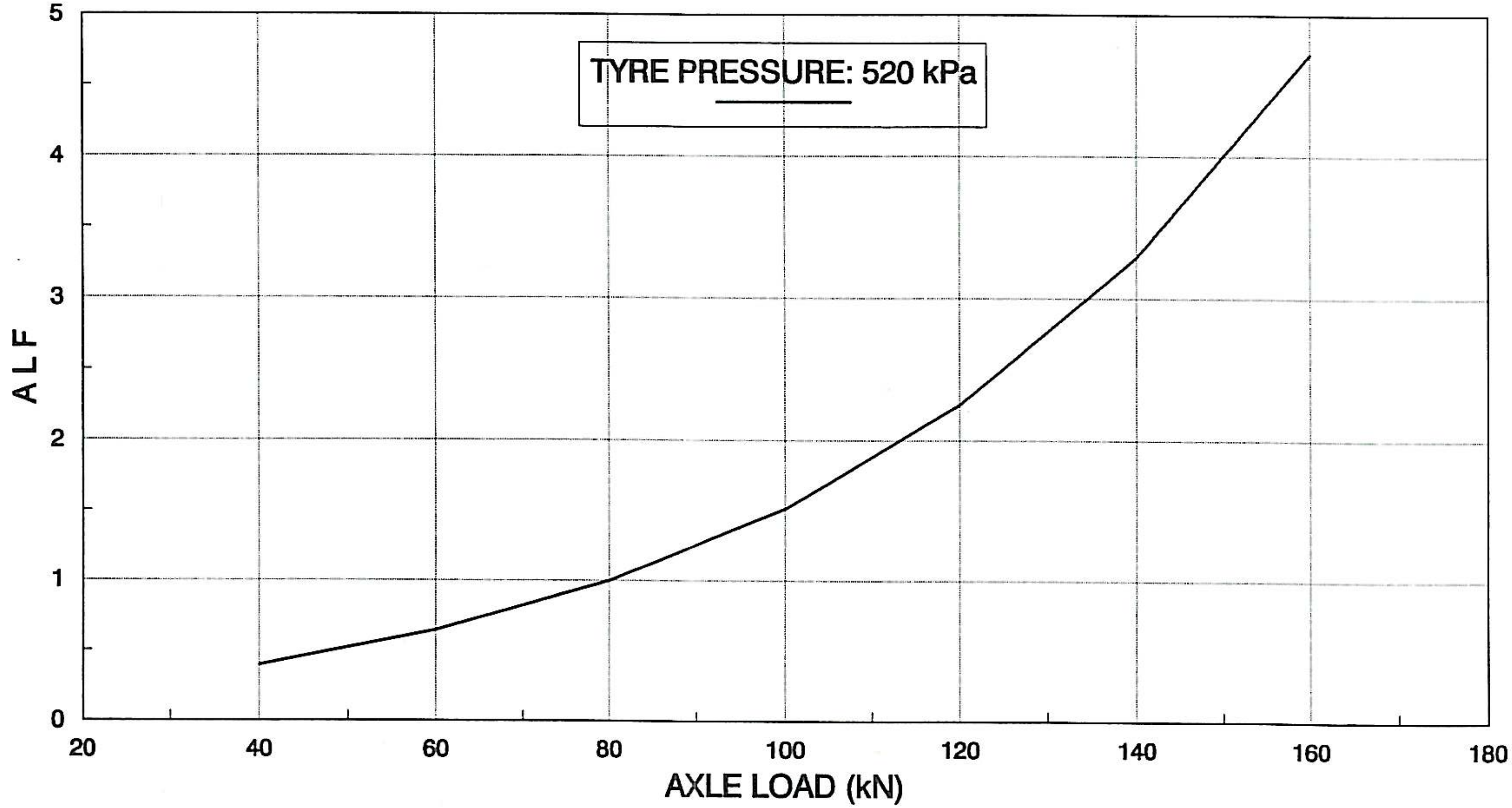


FIGURE C14: Calculated Axle Equivalent Factors (ALF) for different axle loads on Pavement B

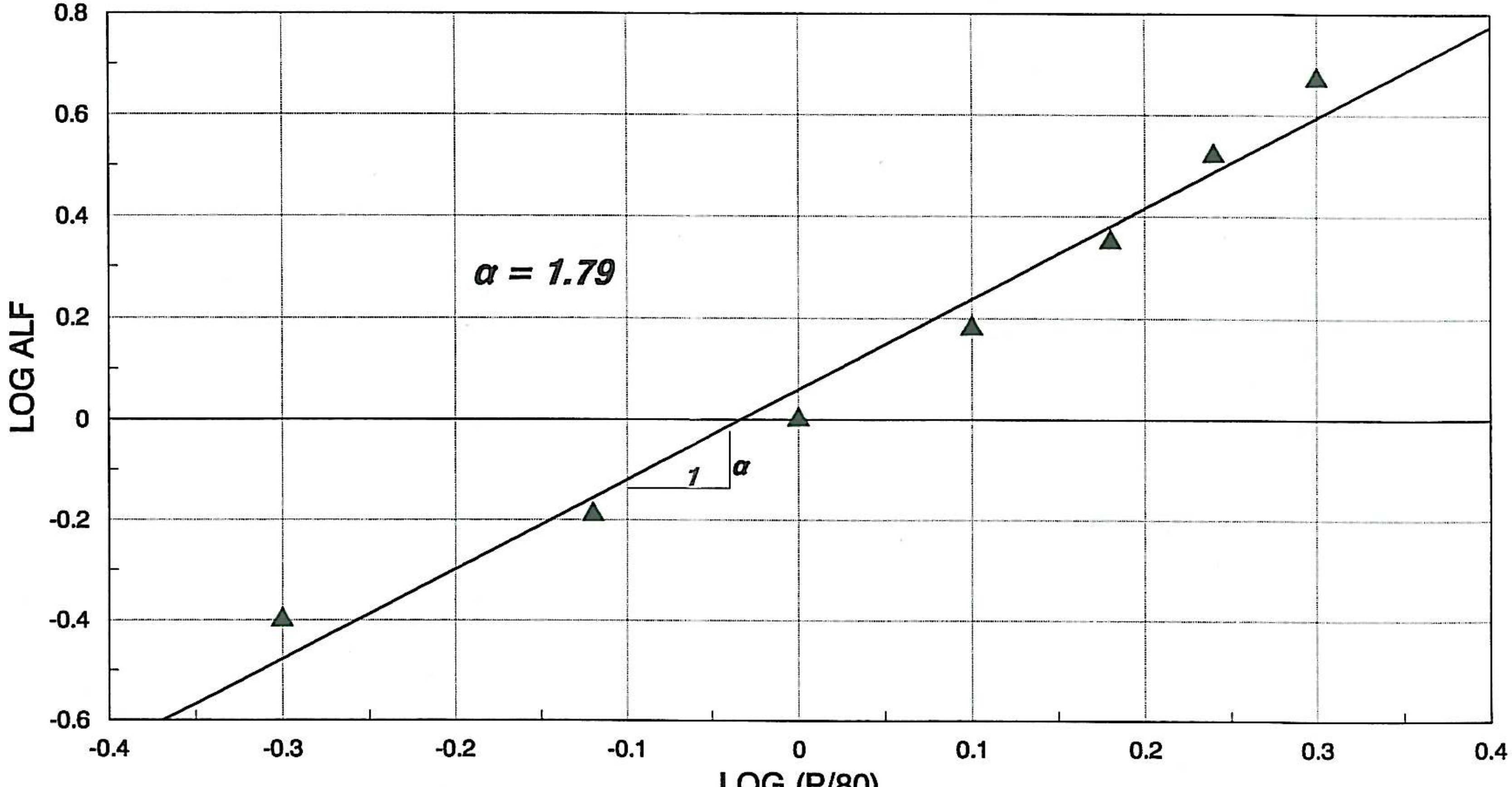


FIGURE C15: Load damage coefficient 'α' for Pavement B as determined by regression analysis of the calculated Axle Load Factors (ALF)

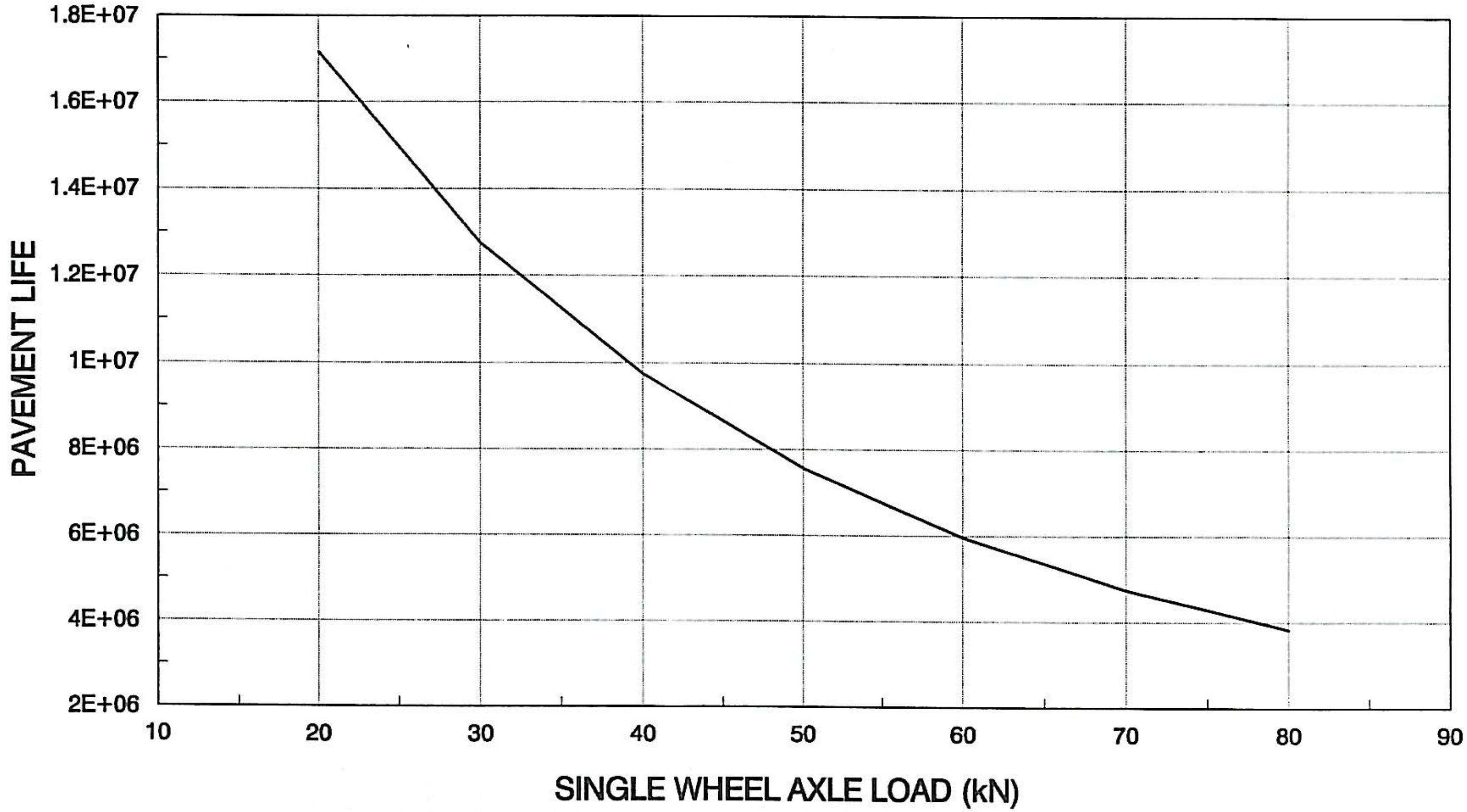


FIGURE C16: Calculated pavement life for a single wheel axle configuration on Pavement B

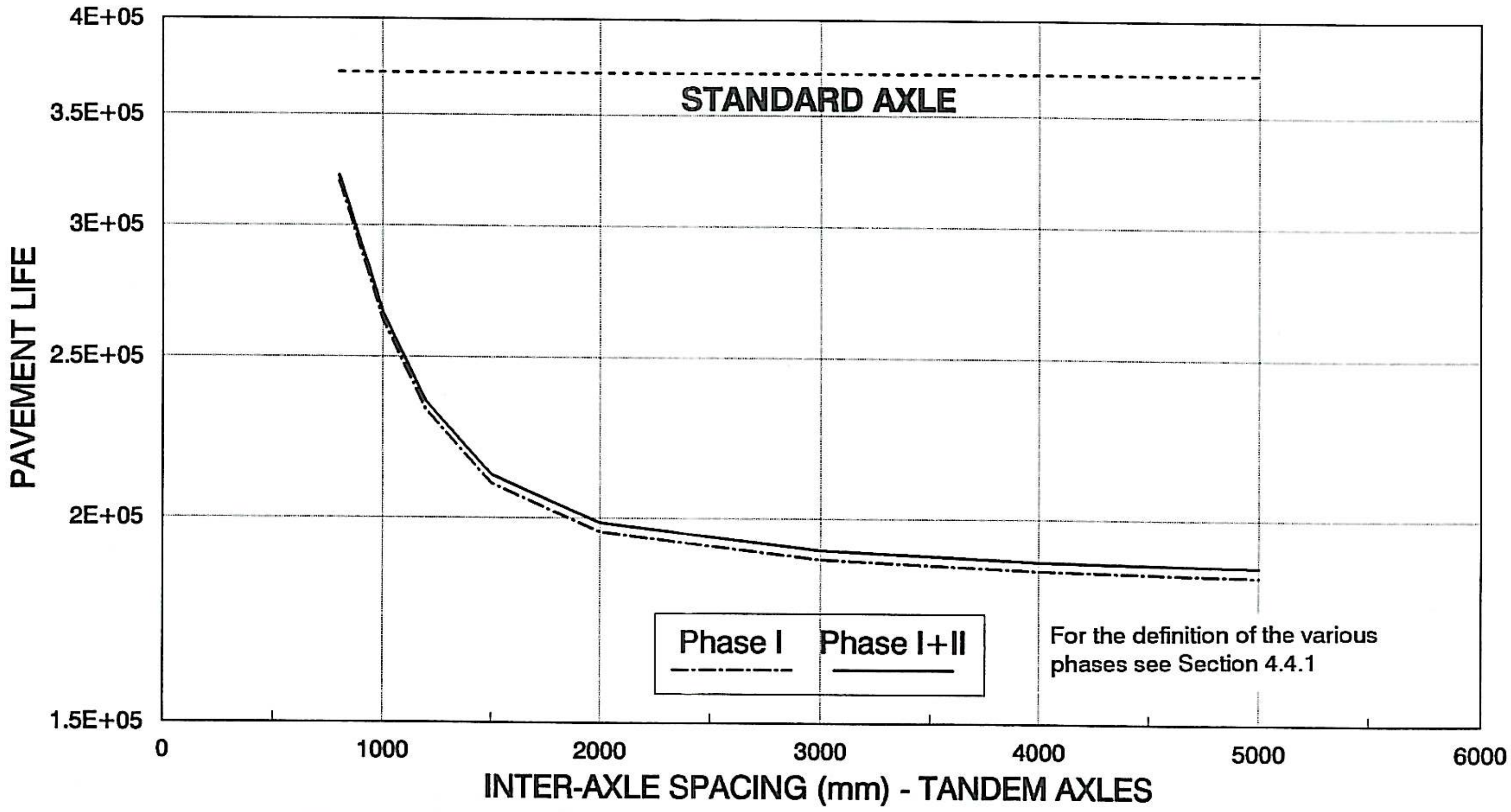


FIGURE C17: Calculated pavement life for different inter-axle spacing for tandem axles on Pavement C (granular base-dry)

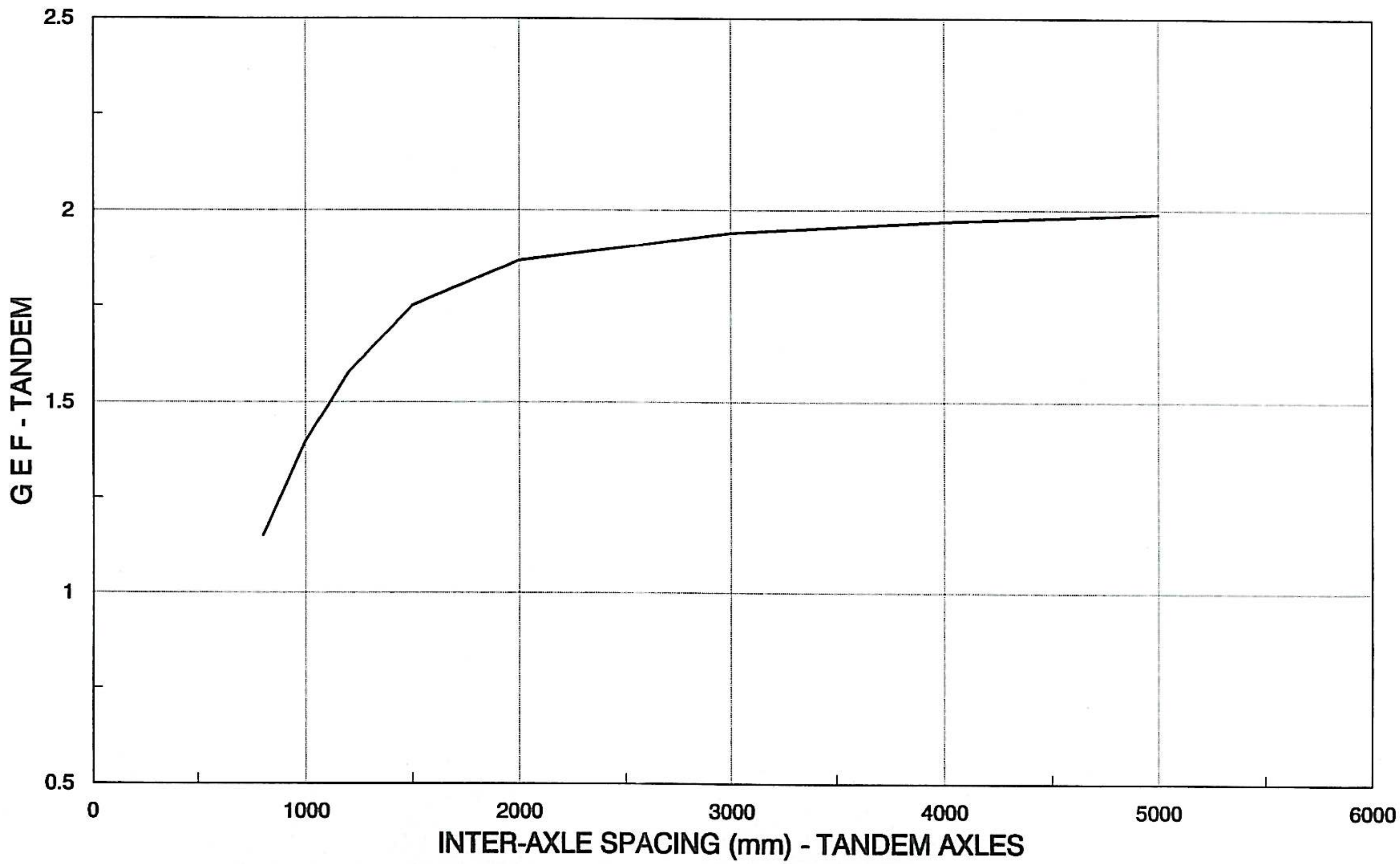


FIGURE C18: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tandem axles on Pavement C

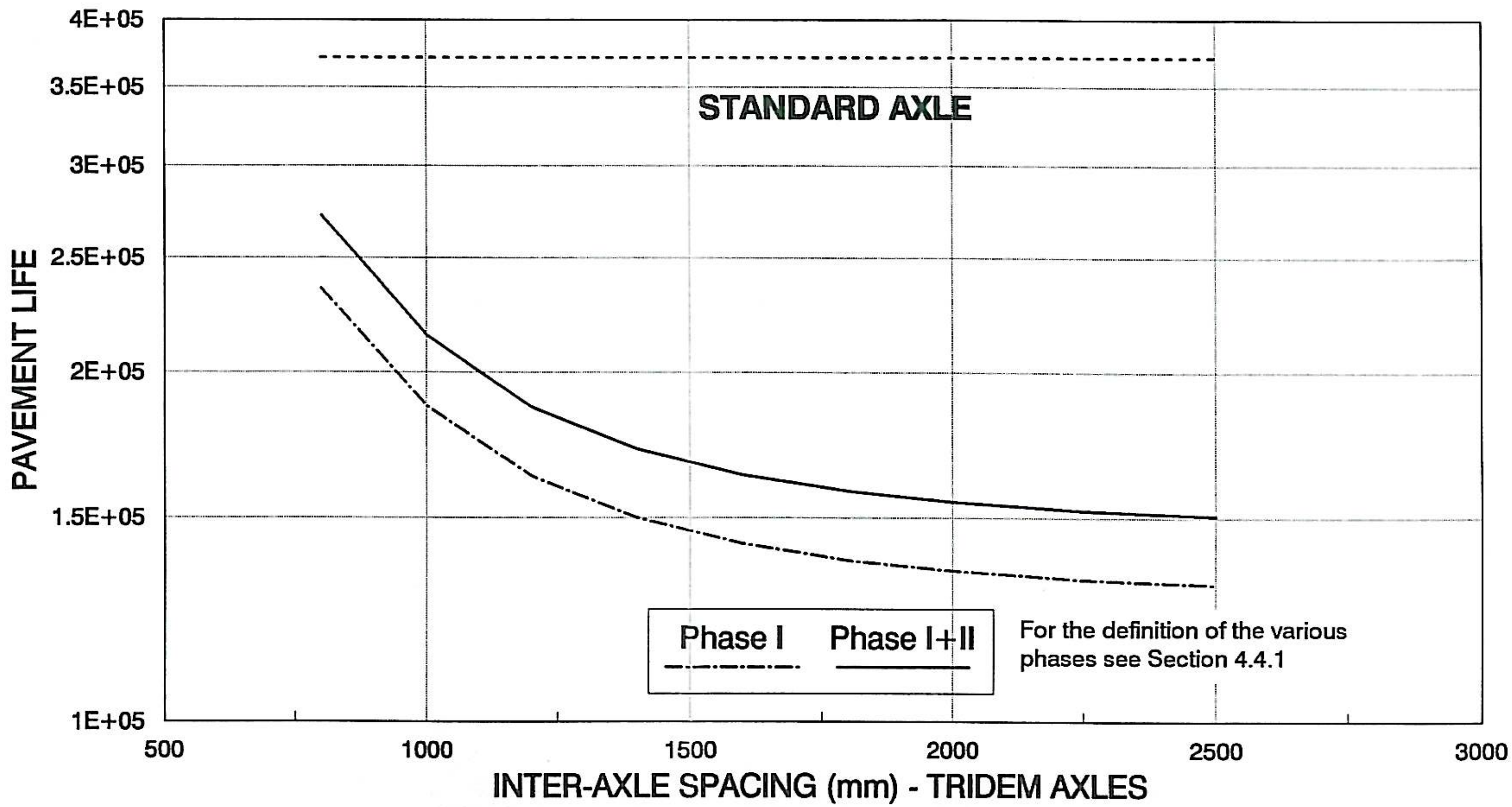


FIGURE C19: Calculated pavement life for different inter-axle spacing for tridem axles on Pavement C (granular base-dry)

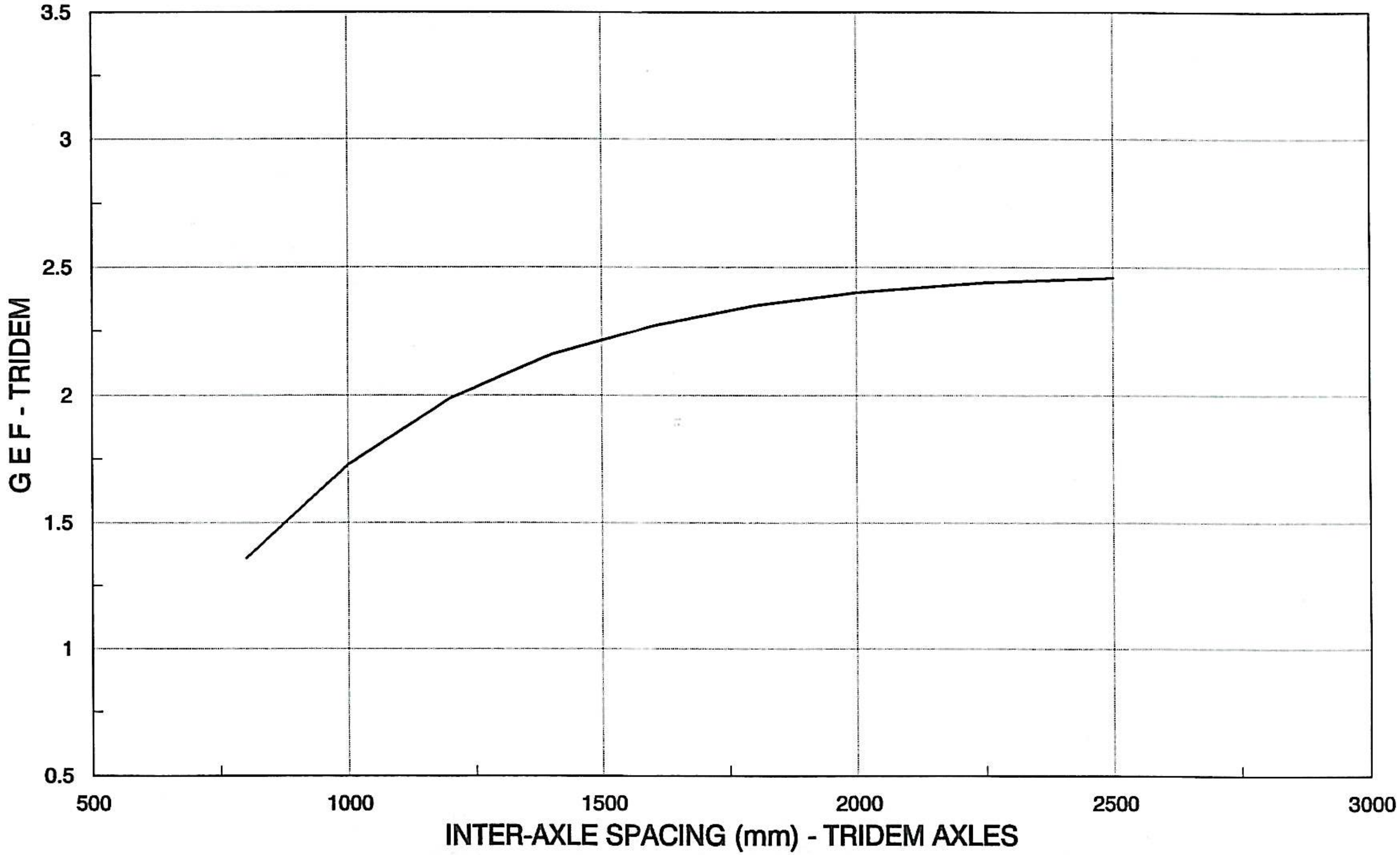


FIGURE C20: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tridem axles on Pavement C

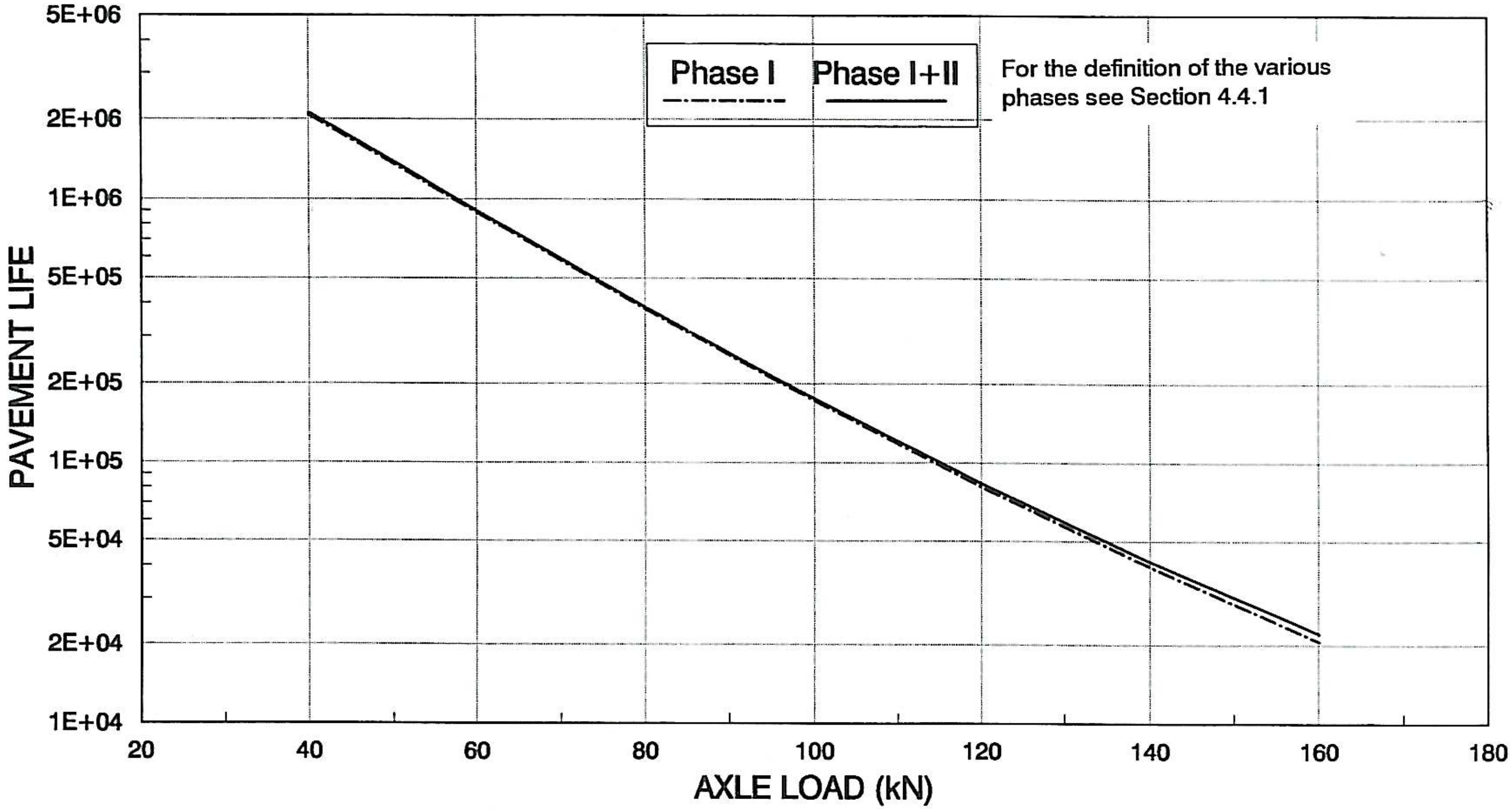


FIGURE C21: Calculated pavement life under a standard dual-wheel axle for different axle loads on Pavement C

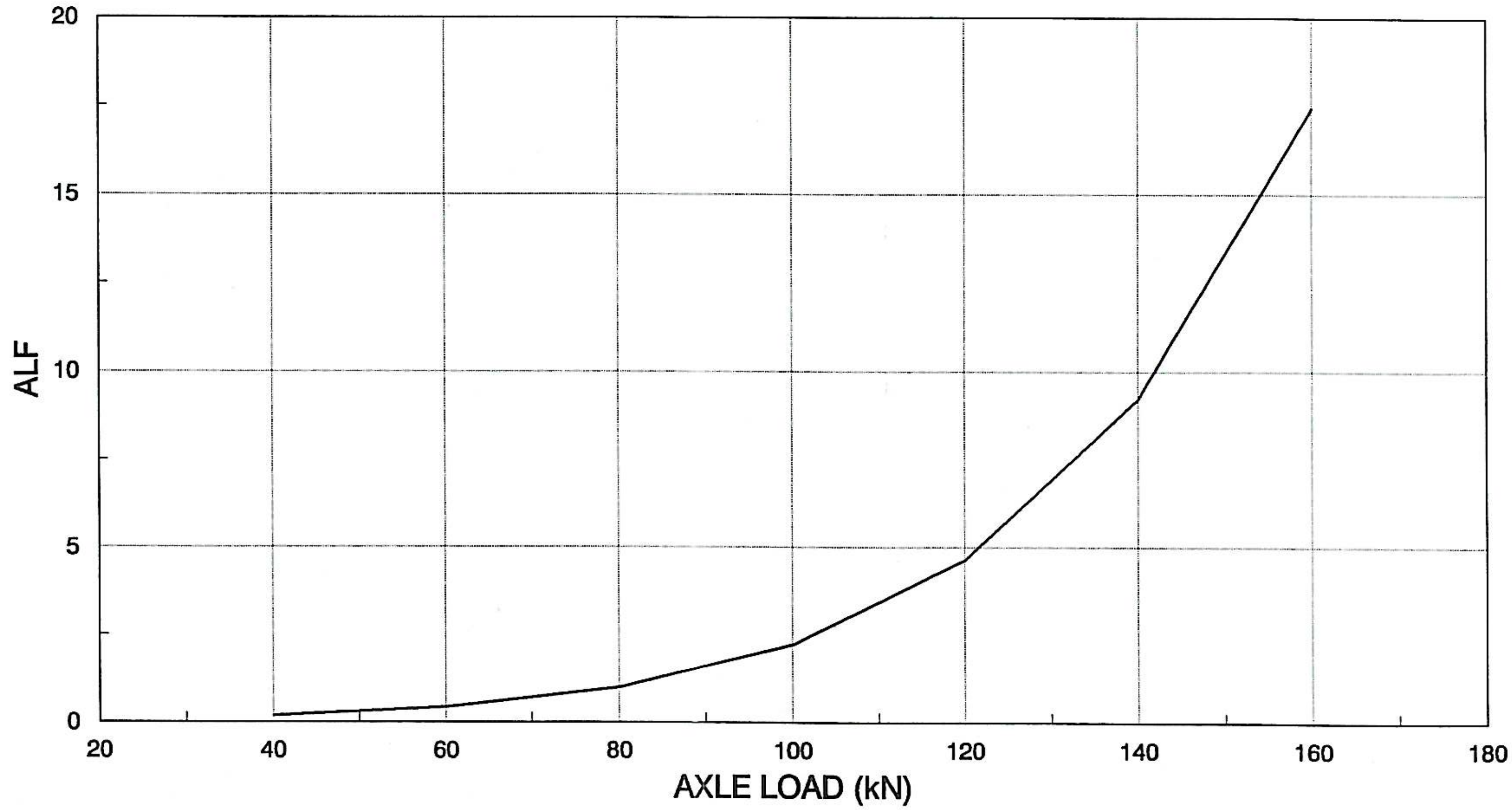


FIGURE C22: Calculated Axle Equivalent Factors (ALF) for different axle loads on Pavement C

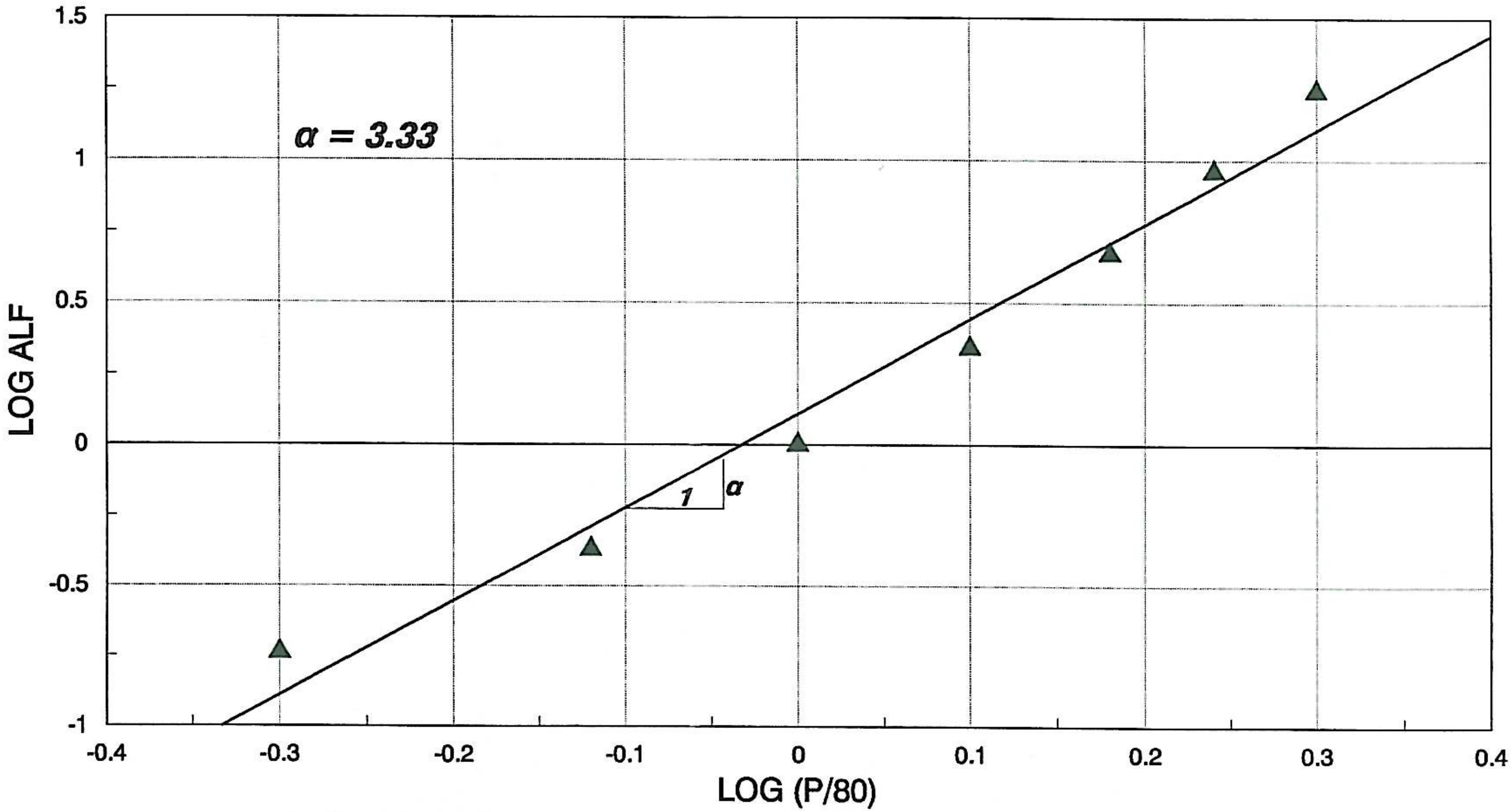


FIGURE C23: Load damage coefficient 'α' for Pavement C as determined by regression analysis of the calculated Axle Load Factors (ALF)

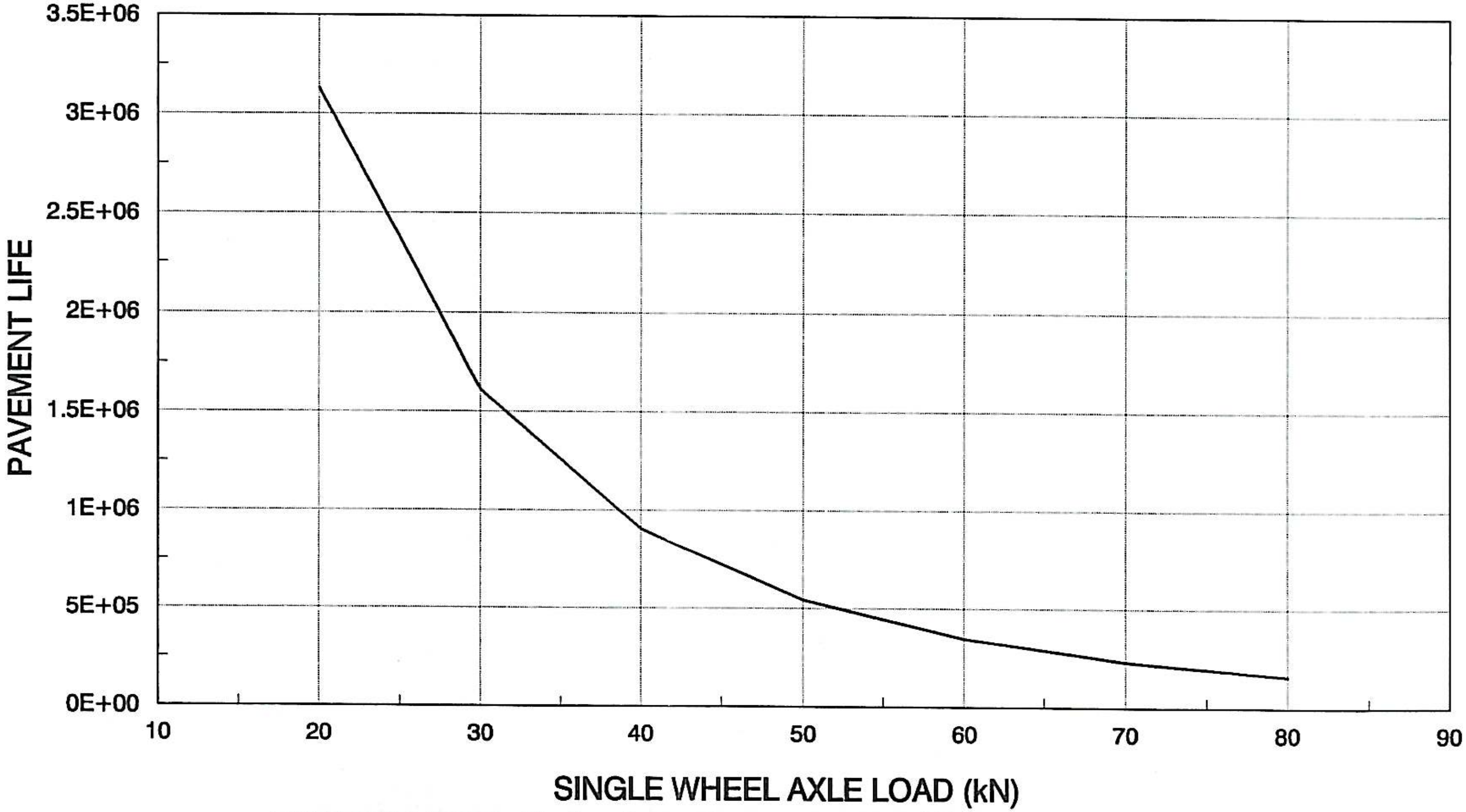


FIGURE C24: Calculated pavement life for a single wheel axle configuration on Pavement C

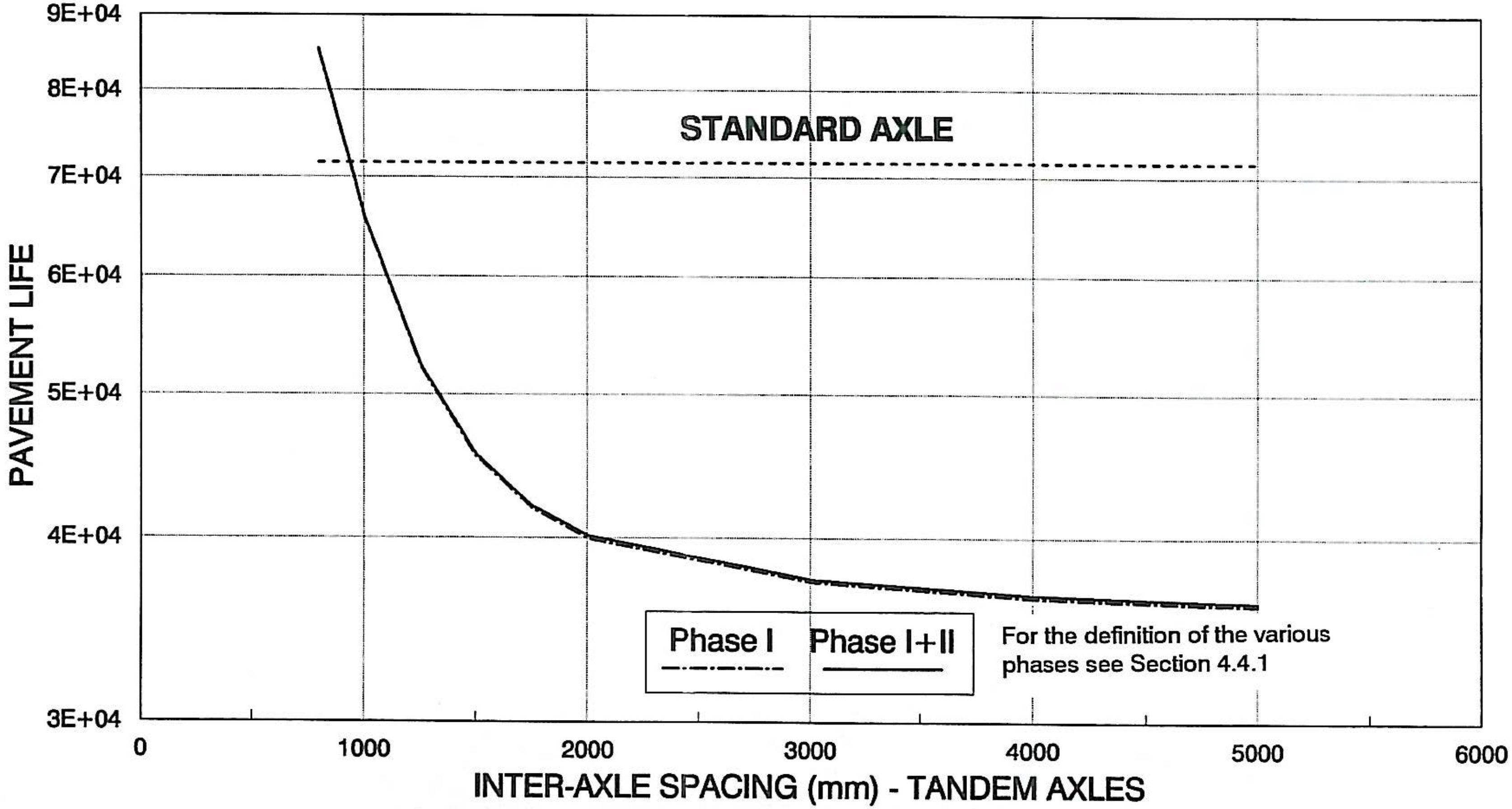


FIGURE C25: Calculated pavement life for different inter-axle spacing for tandem axles on Pavement D (granular base-wet)

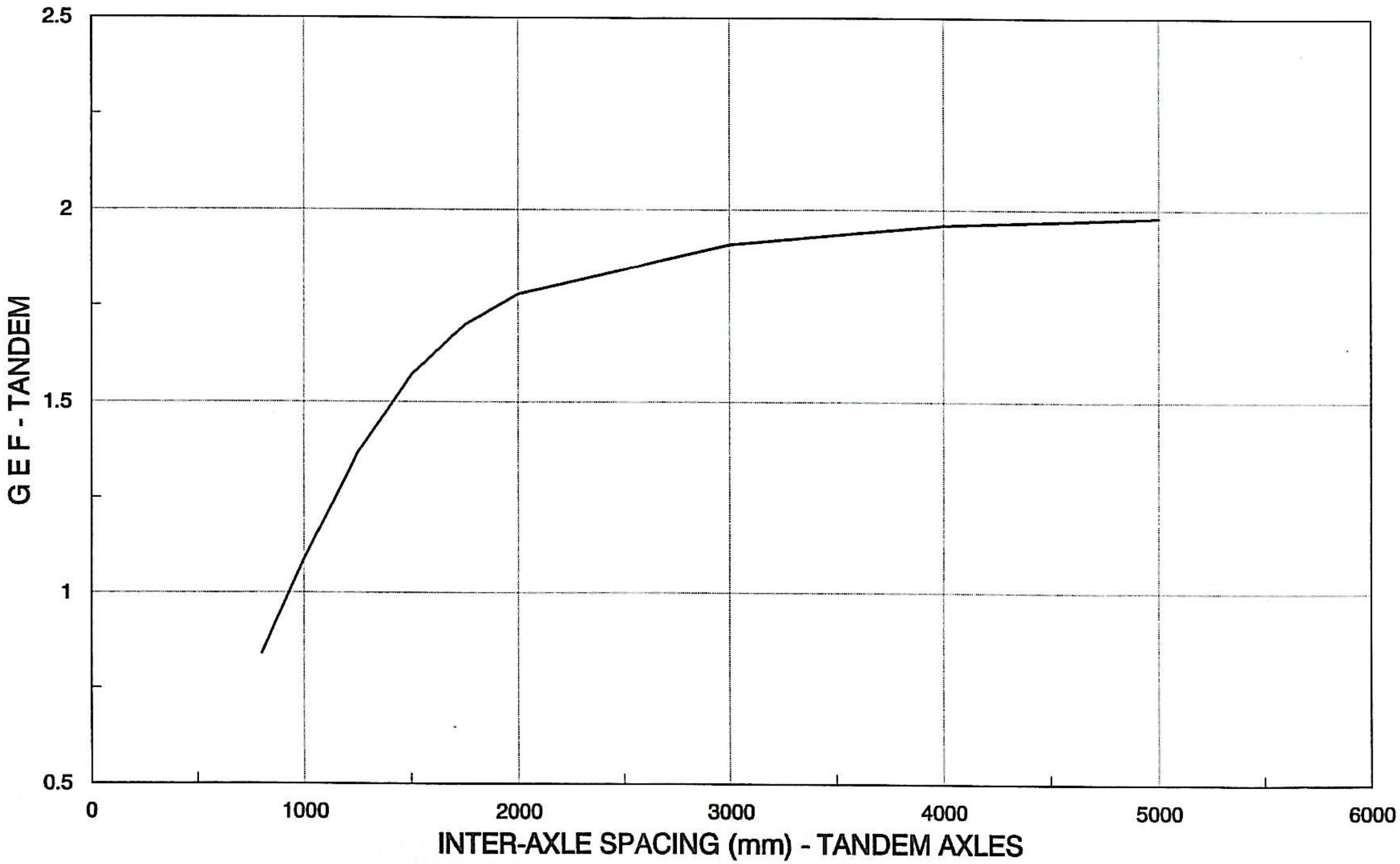


FIGURE C26: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tandem axles on Pavement D

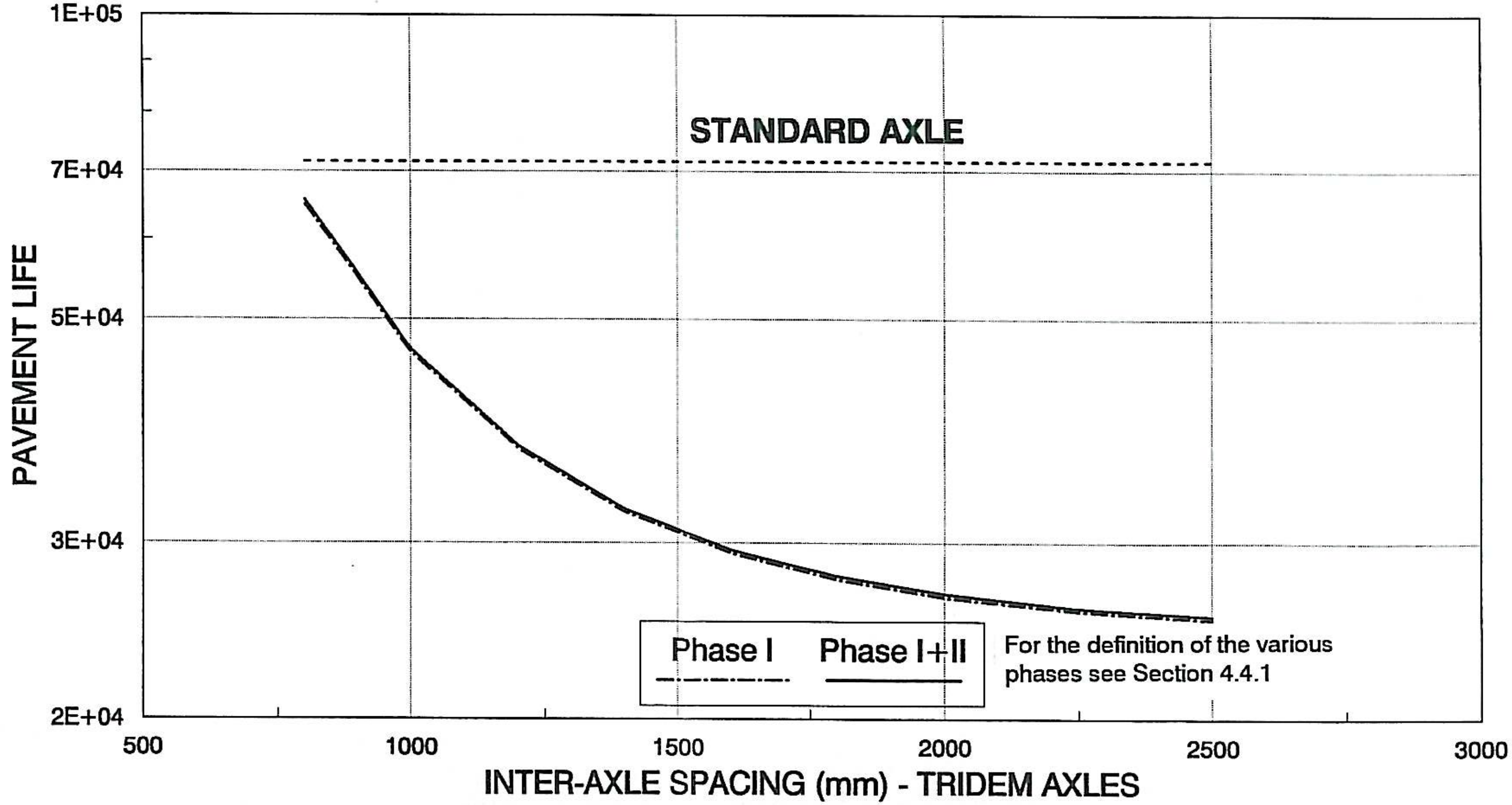


FIGURE C27: Calculated pavement life for different inter-axle spacing for tridem axles on Pavement D (granular base-wet)

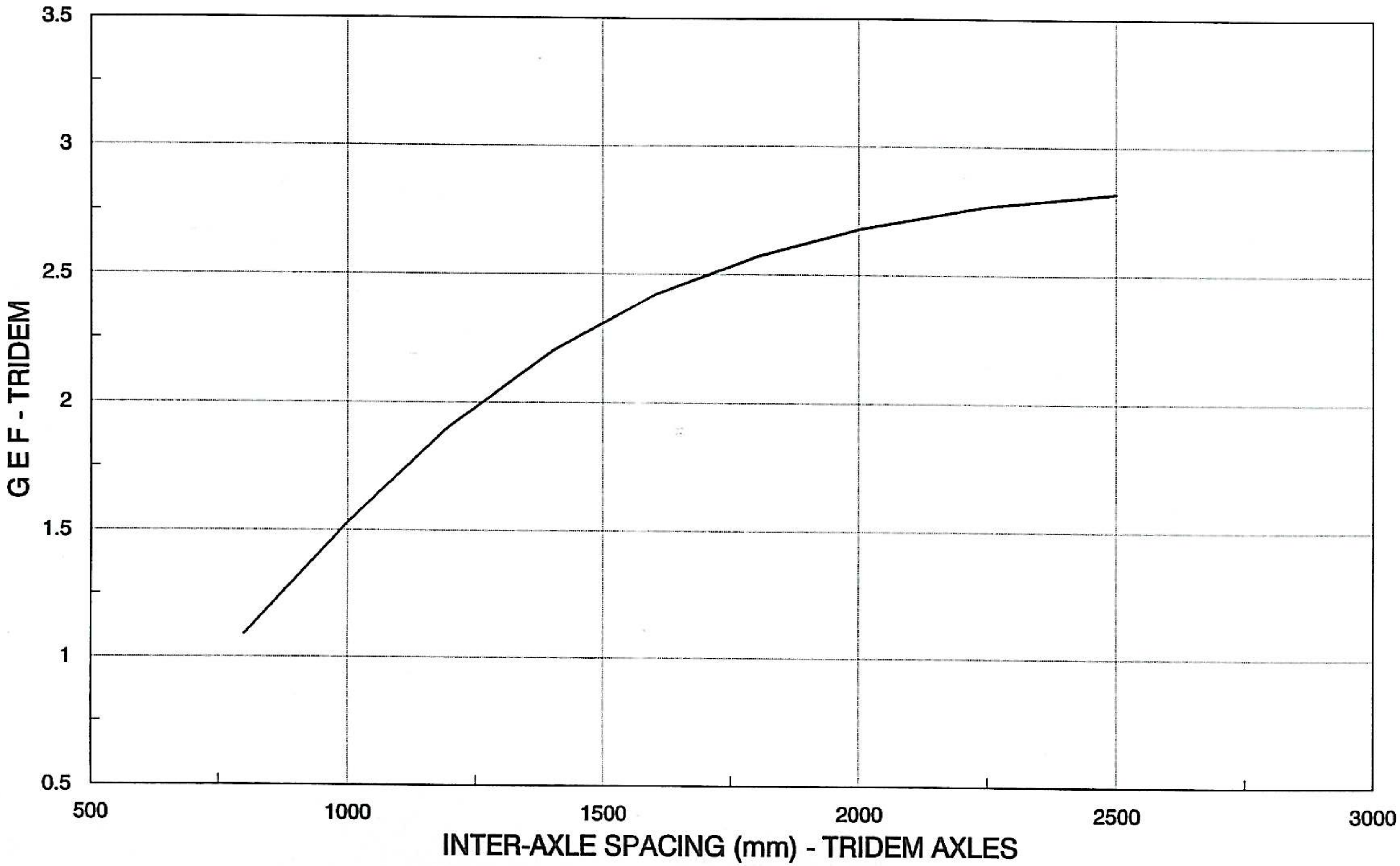


FIGURE C28: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tridem axles on Pavement D

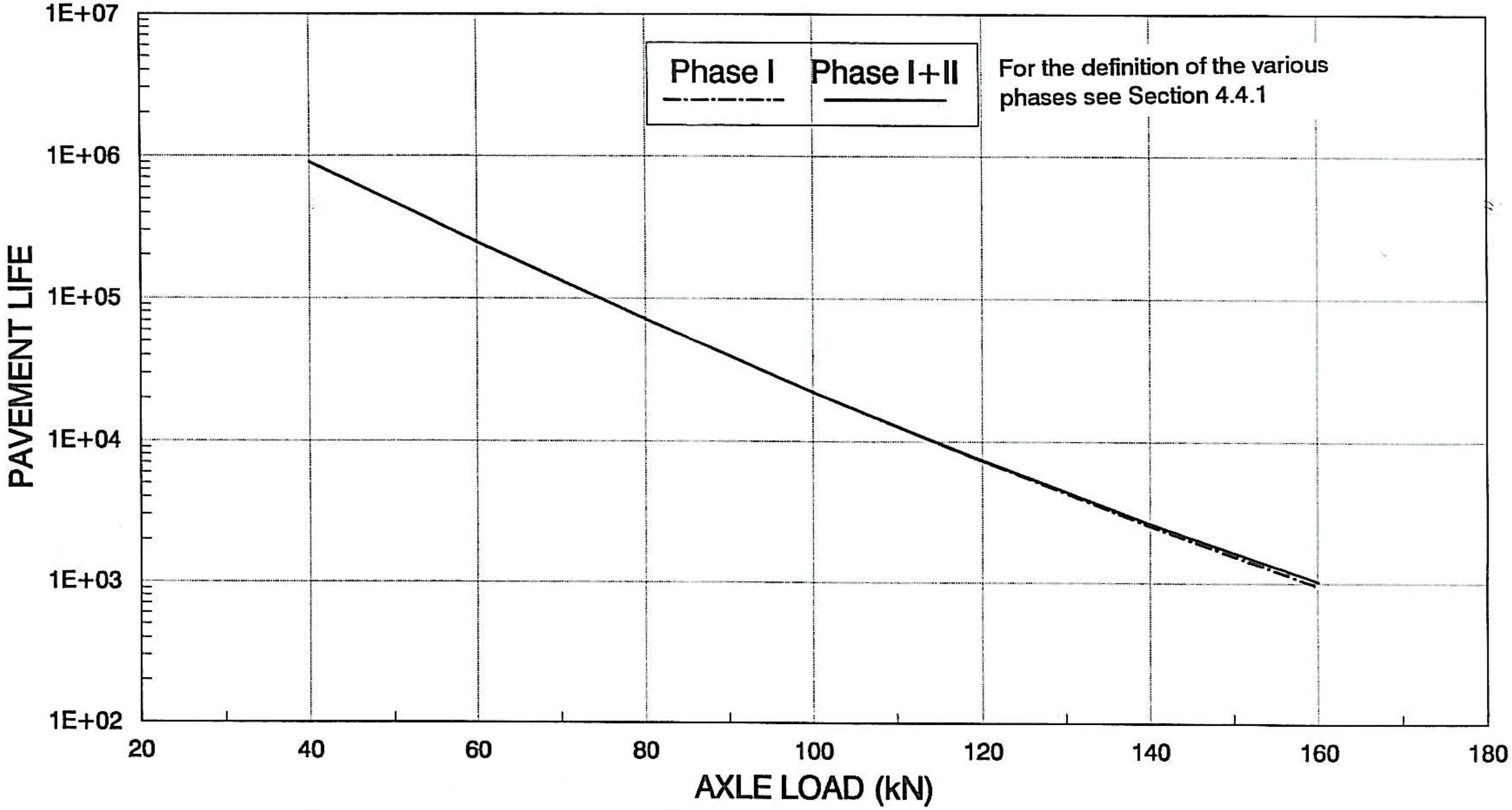


FIGURE C29: Calculated pavement life under a standard dual-wheel axle for different axle loads on Pavement D

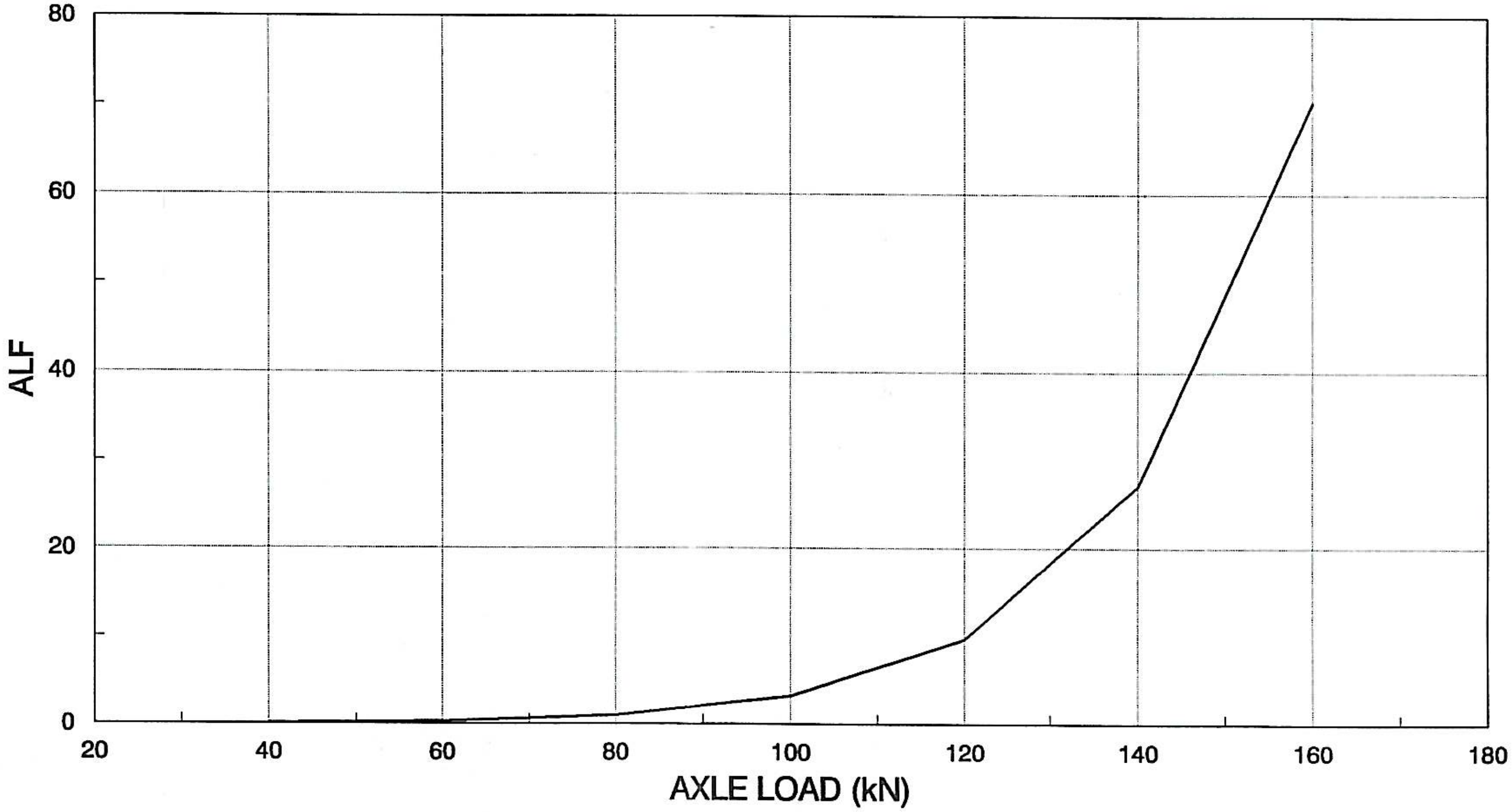


FIGURE C30: Calculated Axle Equivalent Factors (ALF) for different axle loads on Pavement D

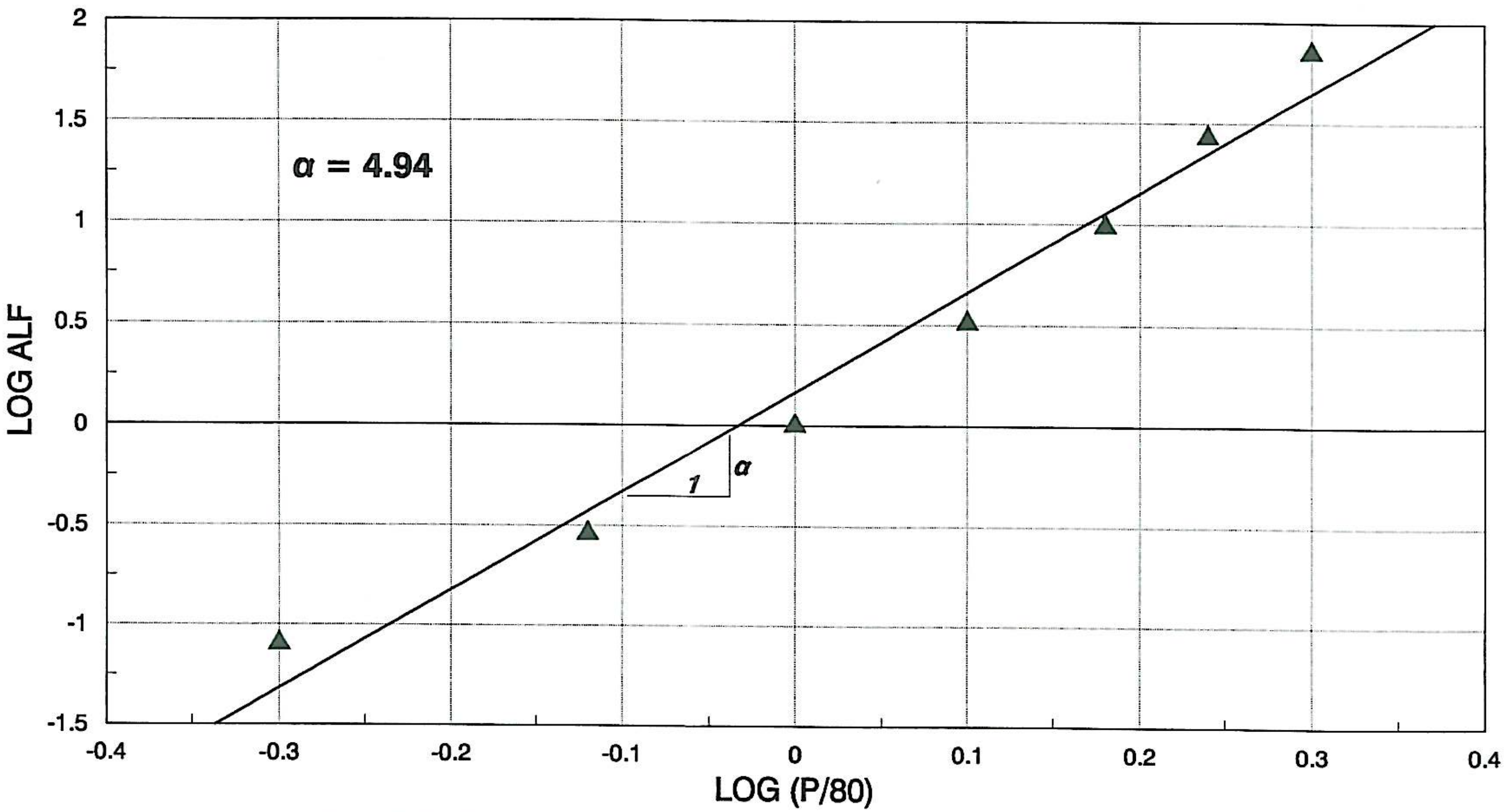


FIGURE C31: Load damage coefficient 'α' for Pavement D as determined by regression analysis of the calculated Axle Load Factors (ALF)

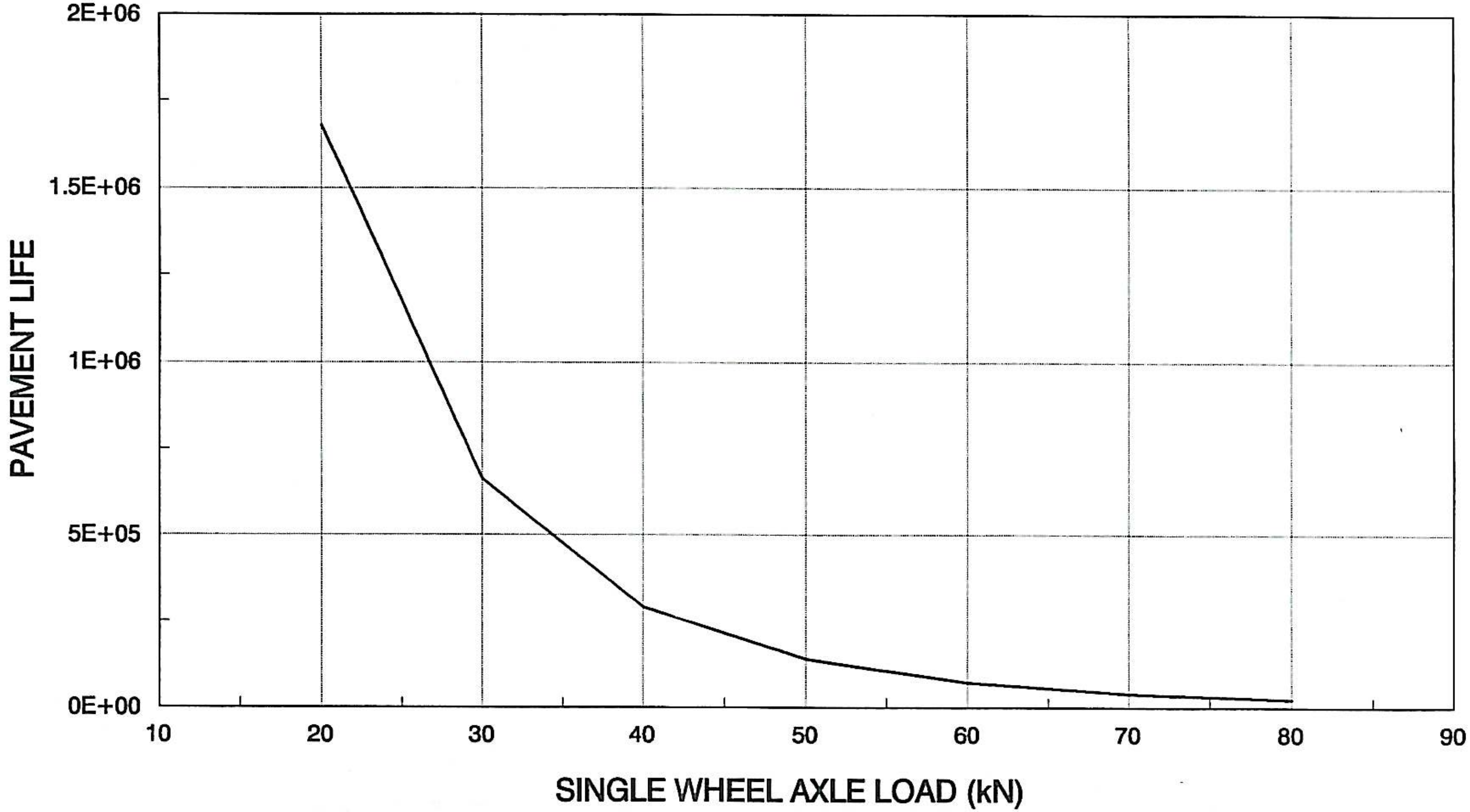


FIGURE C32: Calculated pavement life for a single wheel axle configuration on Pavement D

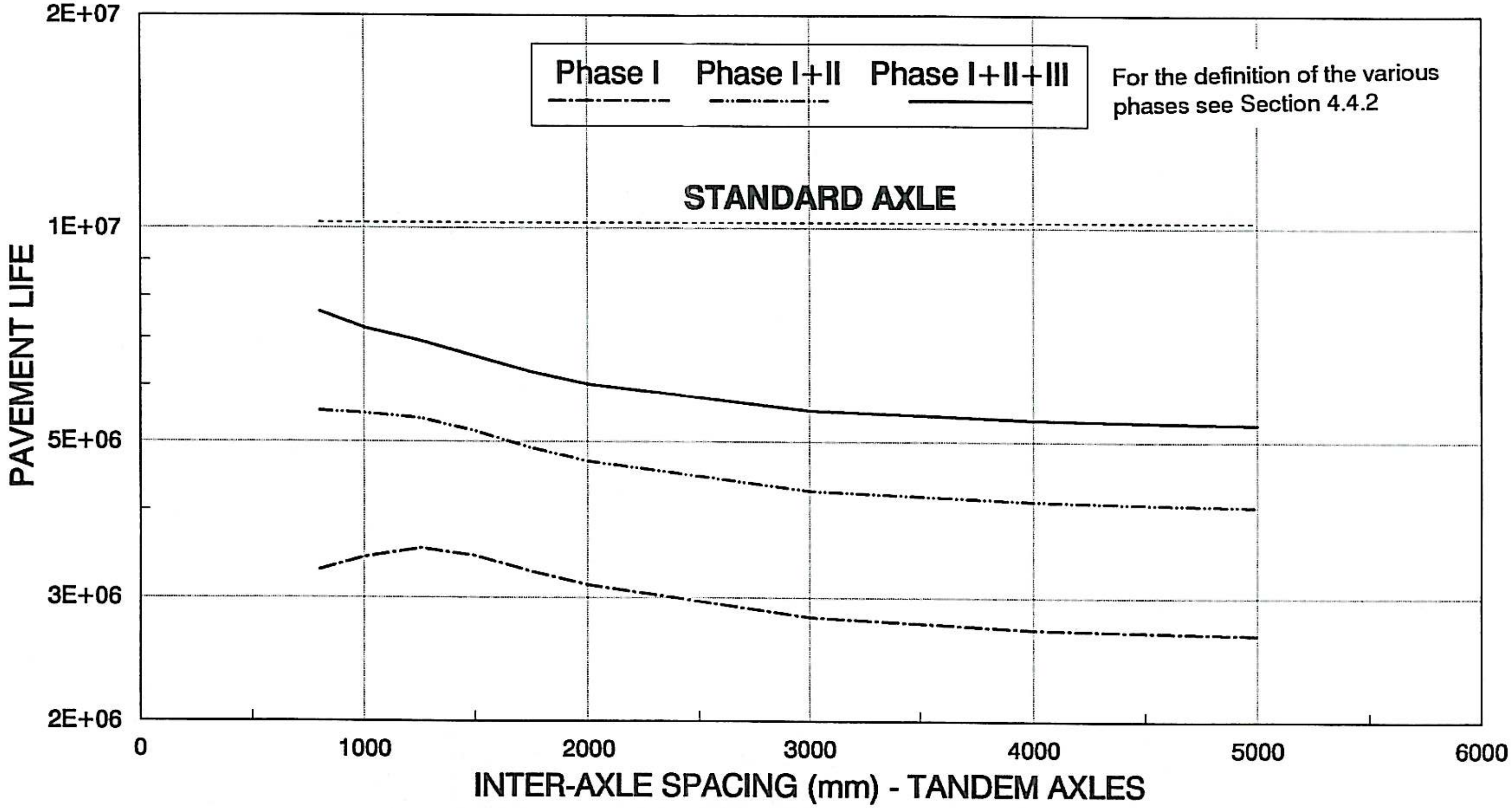


FIGURE C33: Calculated pavement life for different inter-axle spacing for tandem axles on Pavement E (bituminous base)

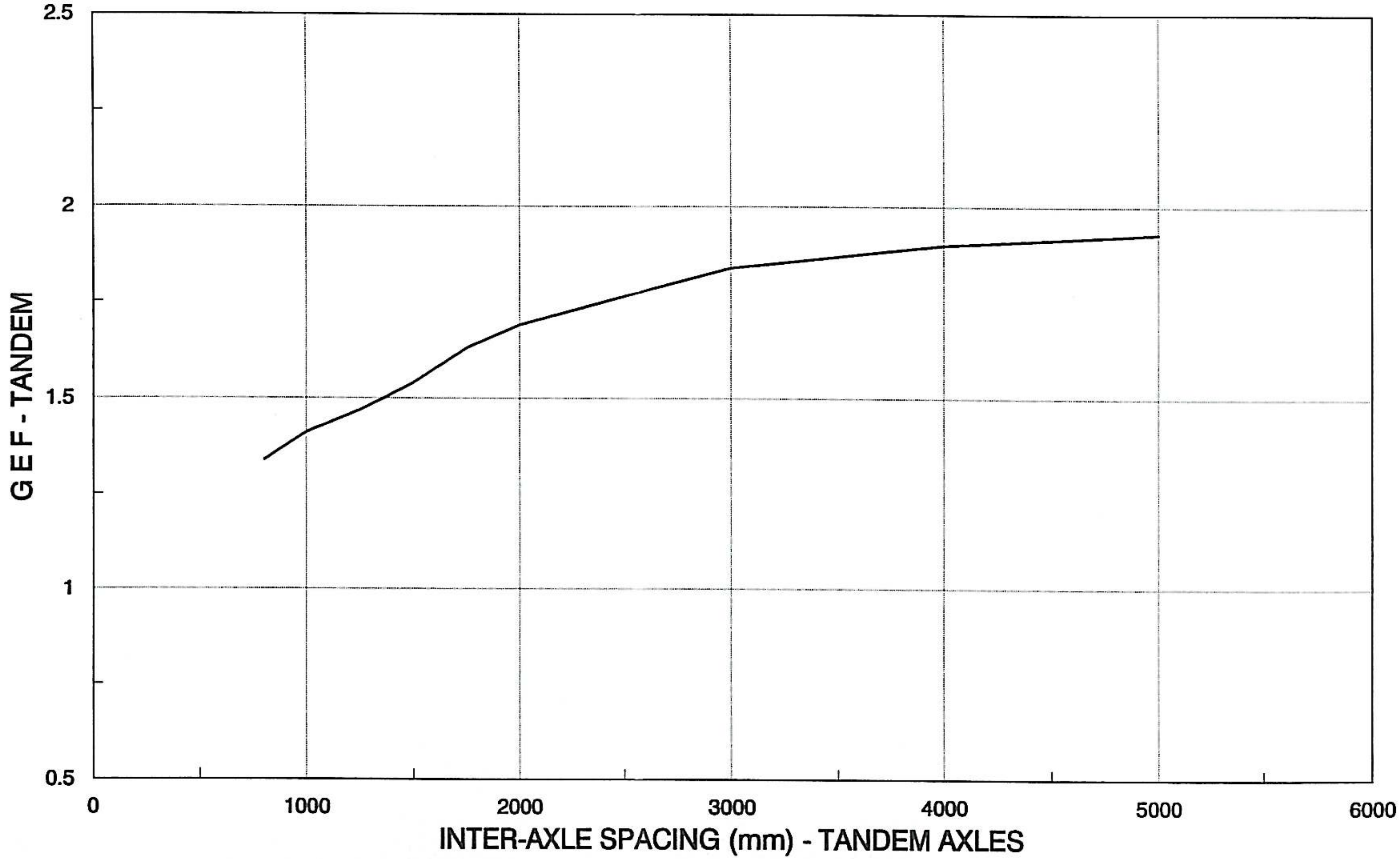


FIGURE C34: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tandem axles on Pavement E

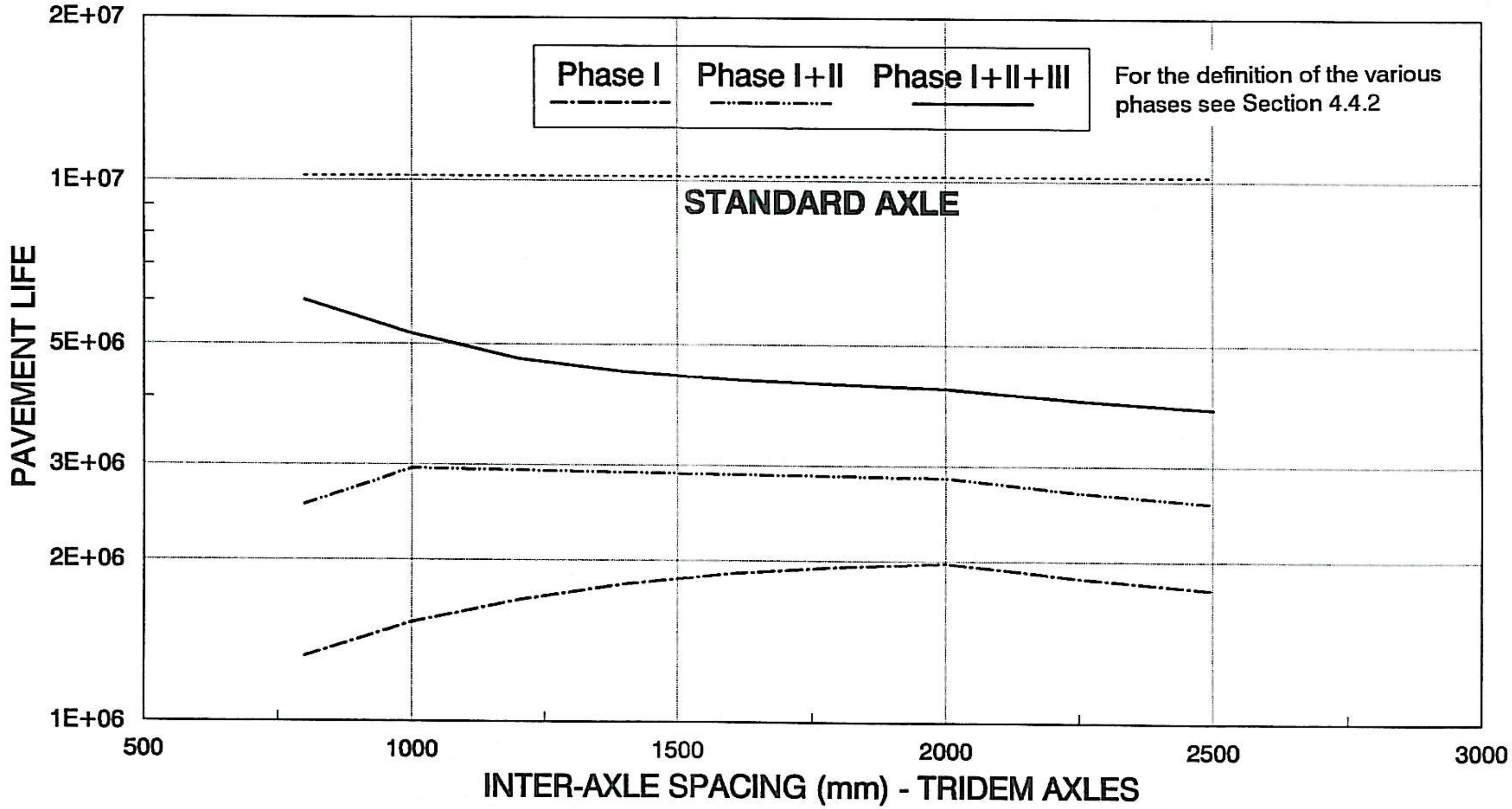


FIGURE C35: Calculated pavement life for different inter-axle spacing for tridem axles on Pavement E (bituminous base)

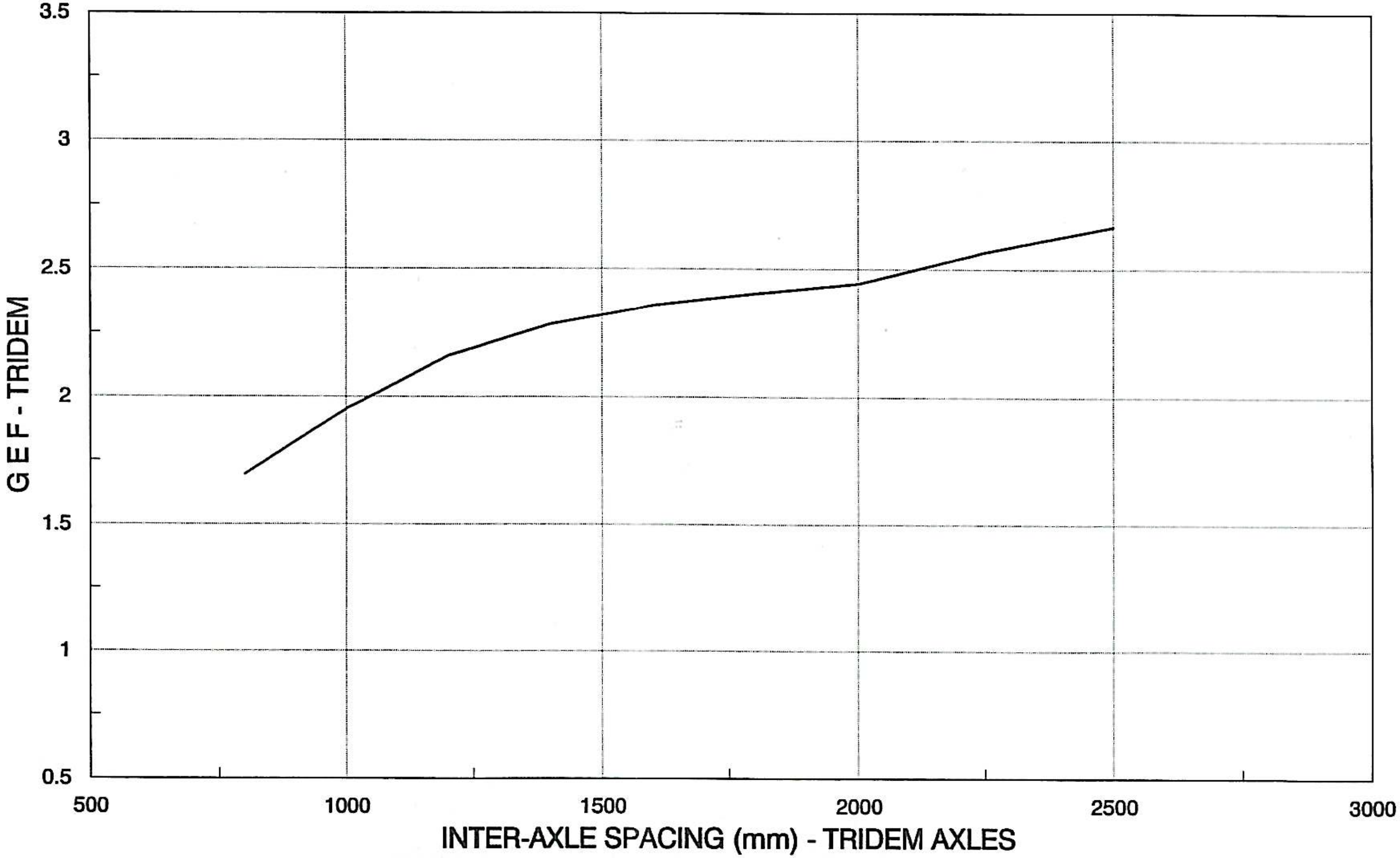


FIGURE C36: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tridem axles on Pavement E

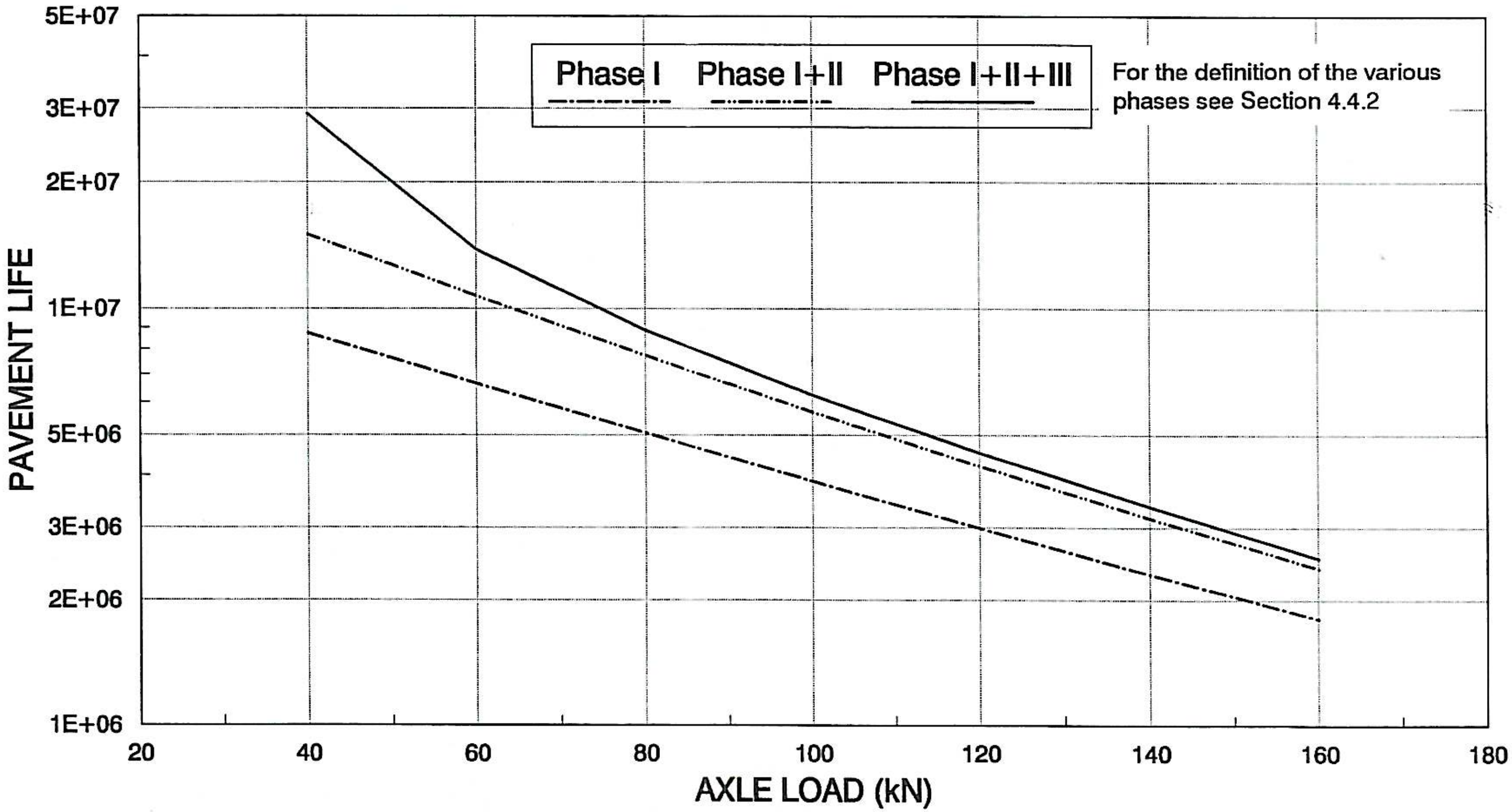


FIGURE C37: Calculated pavement life under a standard dual-wheel axle for different axle loads on Pavement E

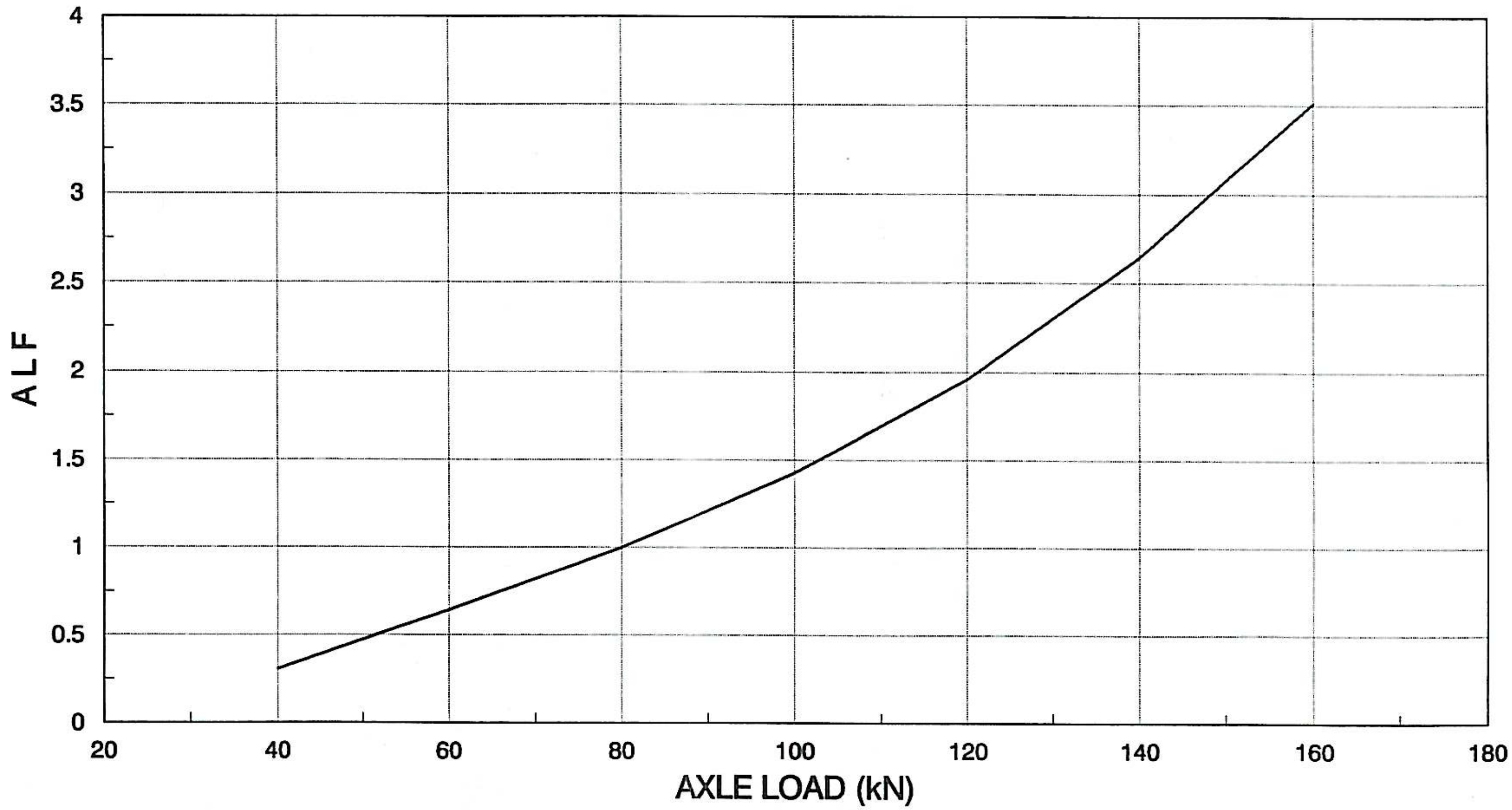


FIGURE C38: Calculated Axle Equivalent Factors (ALF) for different axle loads on Pavement E

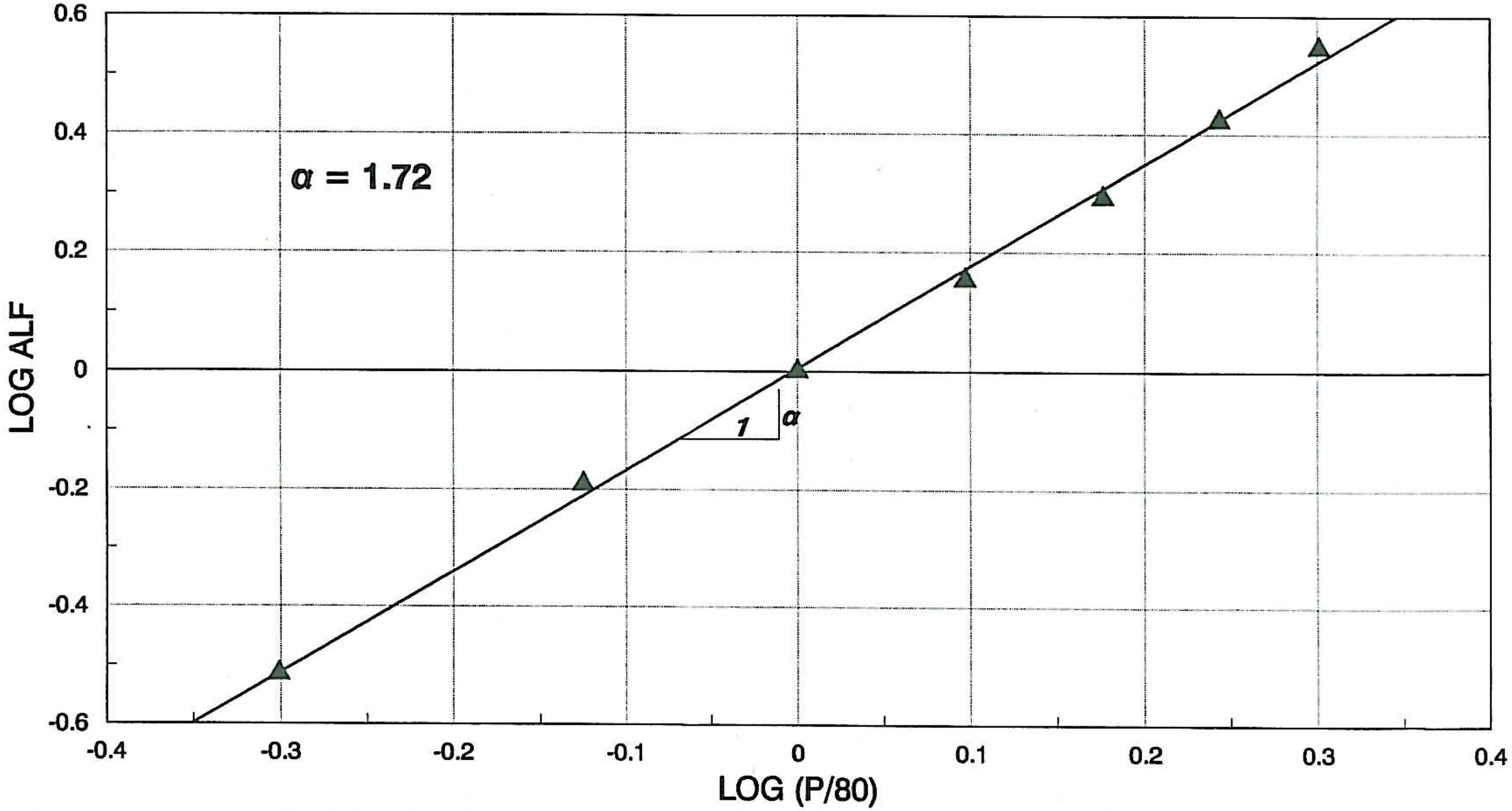


FIGURE C39: Load damage coefficient 'α' for Pavement E as determined by regression analysis of the calculated Axle Load Factors (ALF)

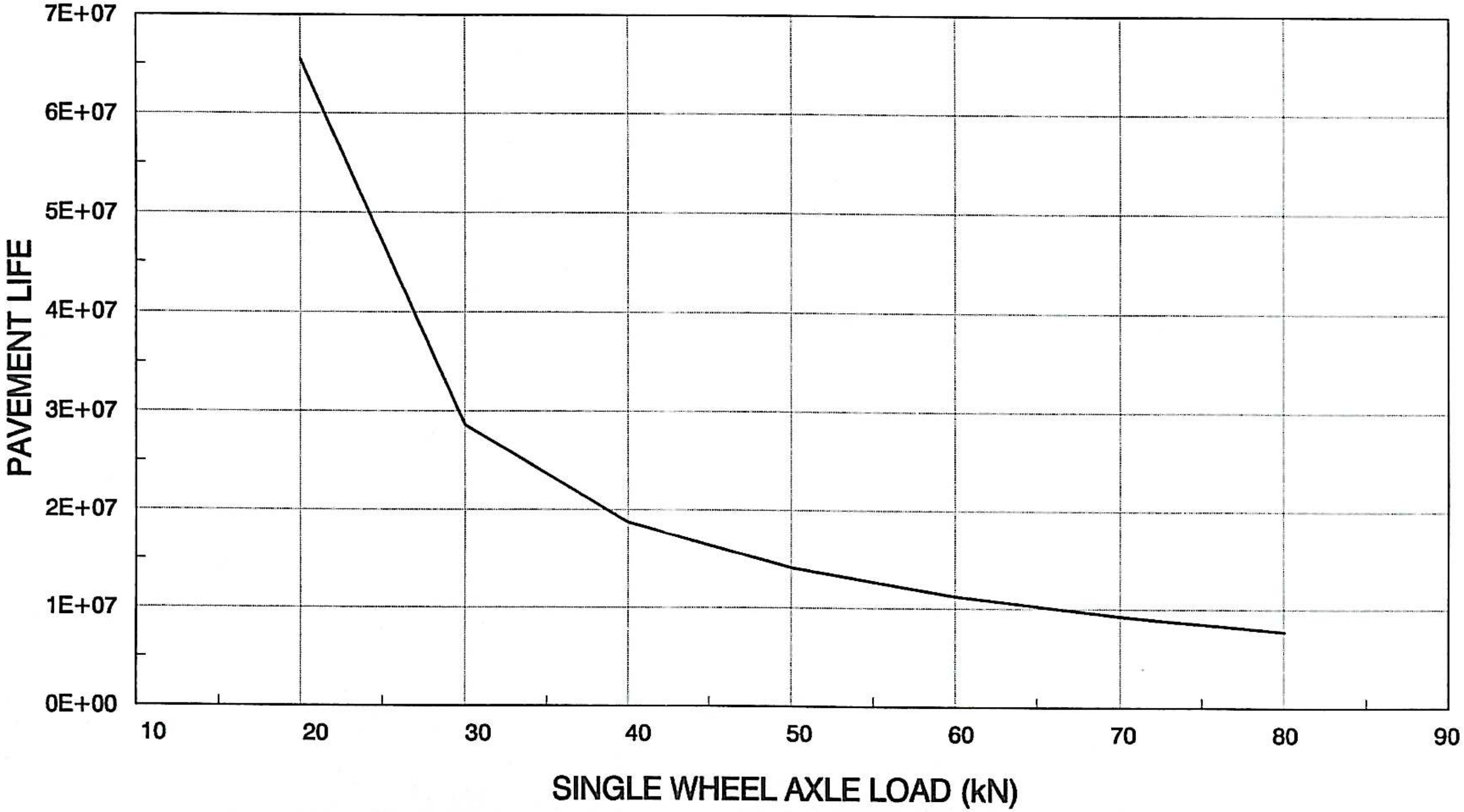


FIGURE C40: Calculated pavement life for a single wheel axle configuration on Pavement E

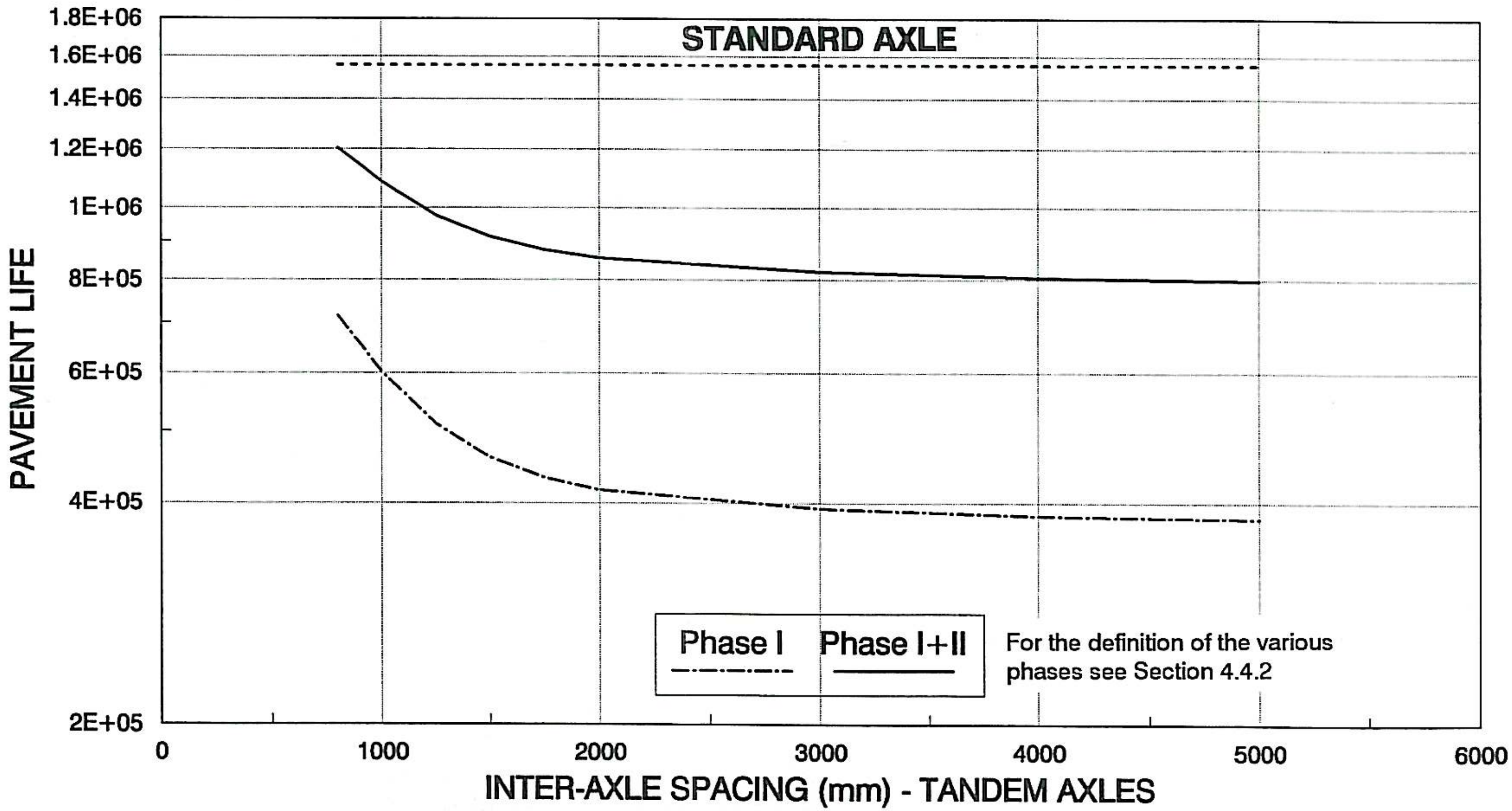


FIGURE C41: Calculated pavement life for different inter-axle spacing for tandem axles on Pavement F (bituminous base)

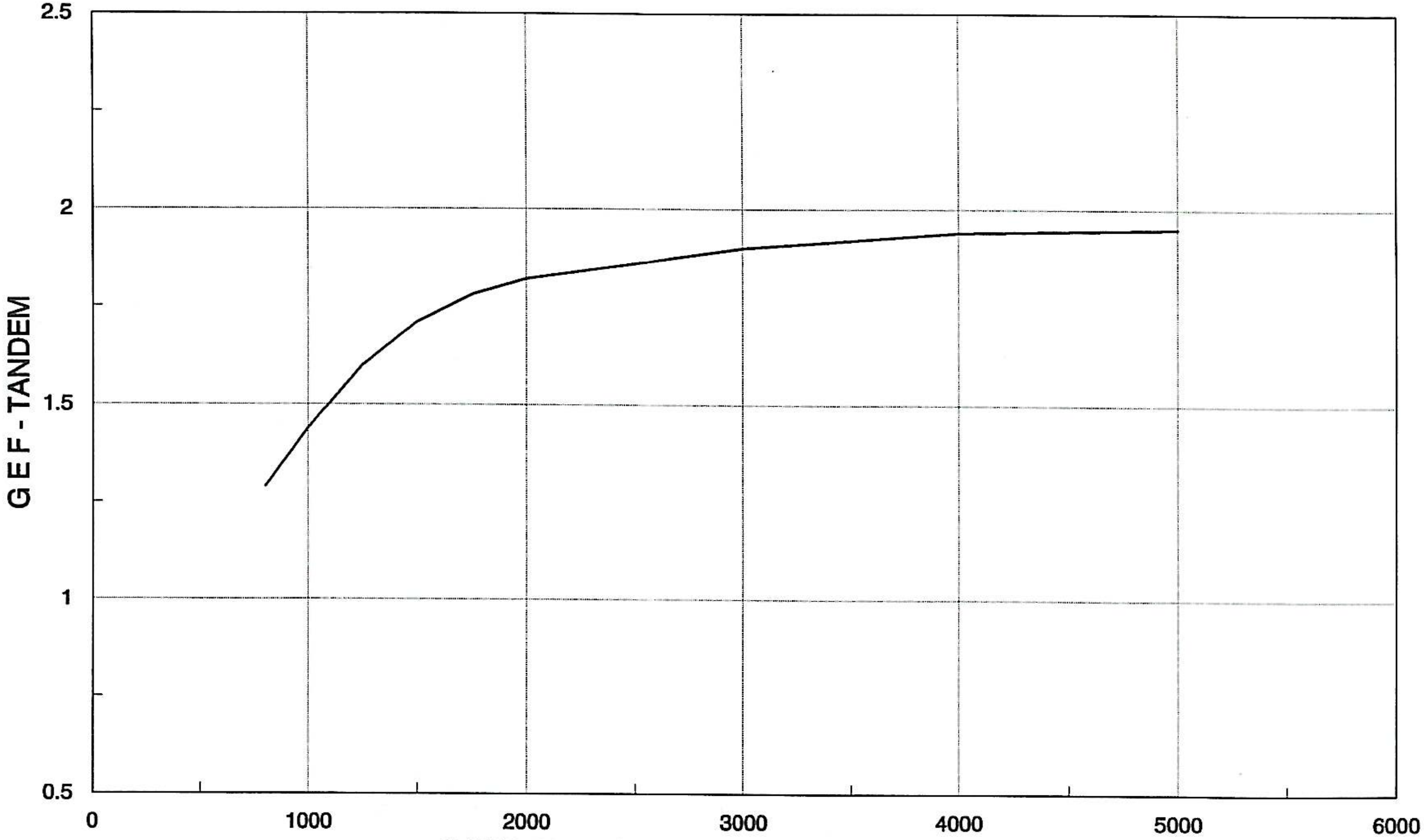


FIGURE C42: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tandem axles on Pavement F

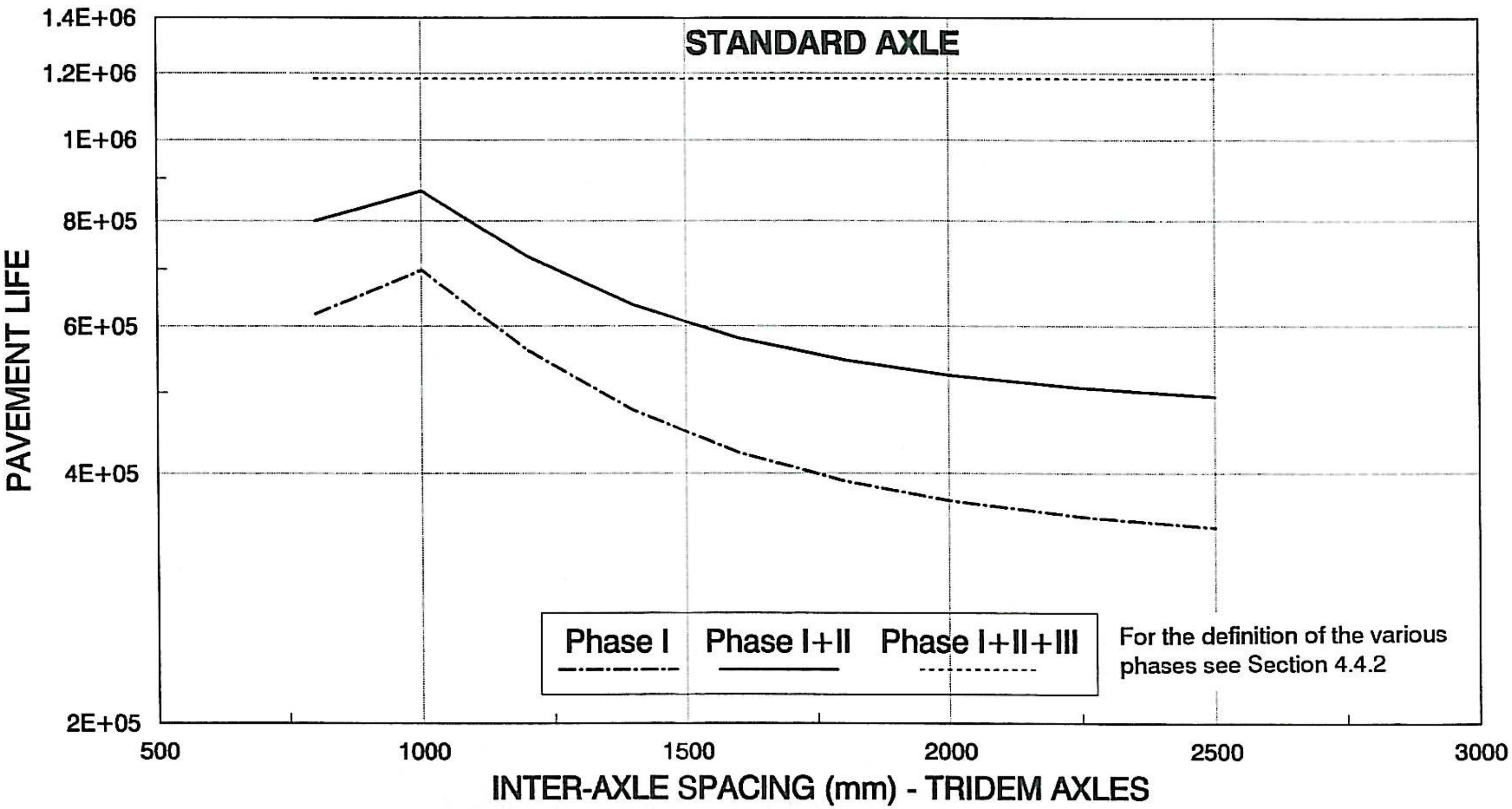


FIGURE C43: Calculated pavement life for different inter-axle spacing for tridem axles on Pavement F (bituminous base)

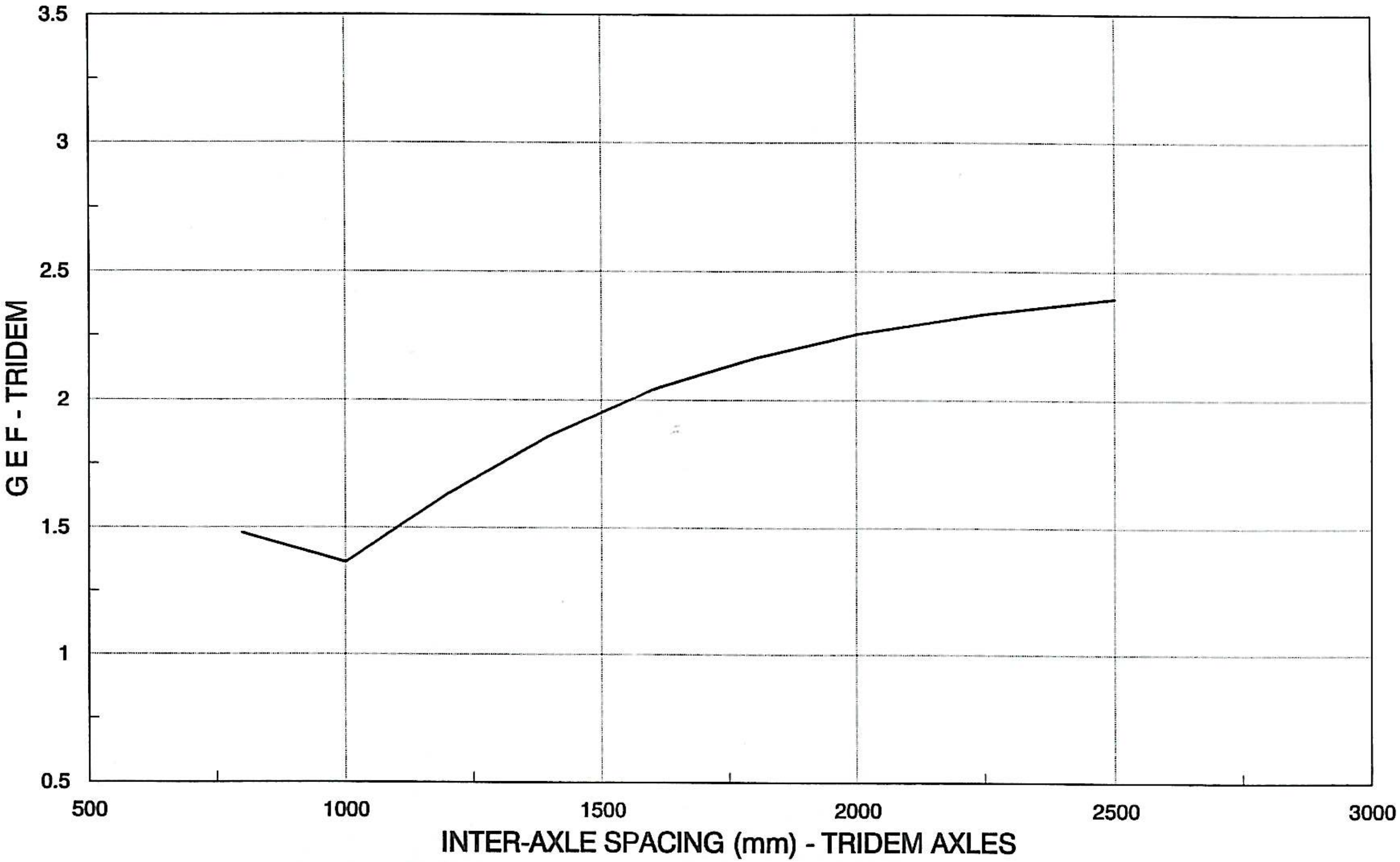


FIGURE C44: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tridem axles on Pavement F

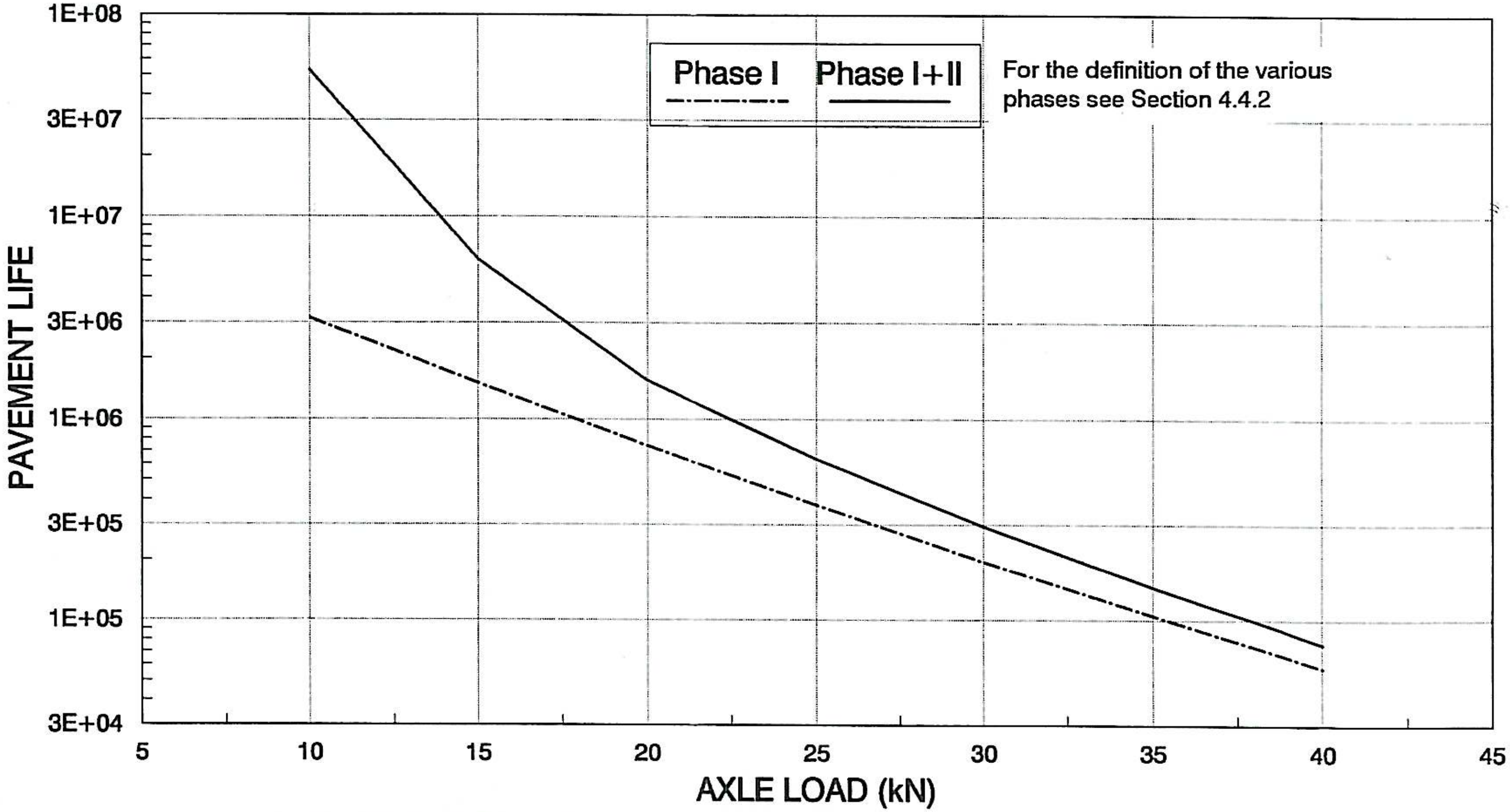


FIGURE C45: Calculated pavement life under a standard dual-wheel axle for different axle loads on Pavement F

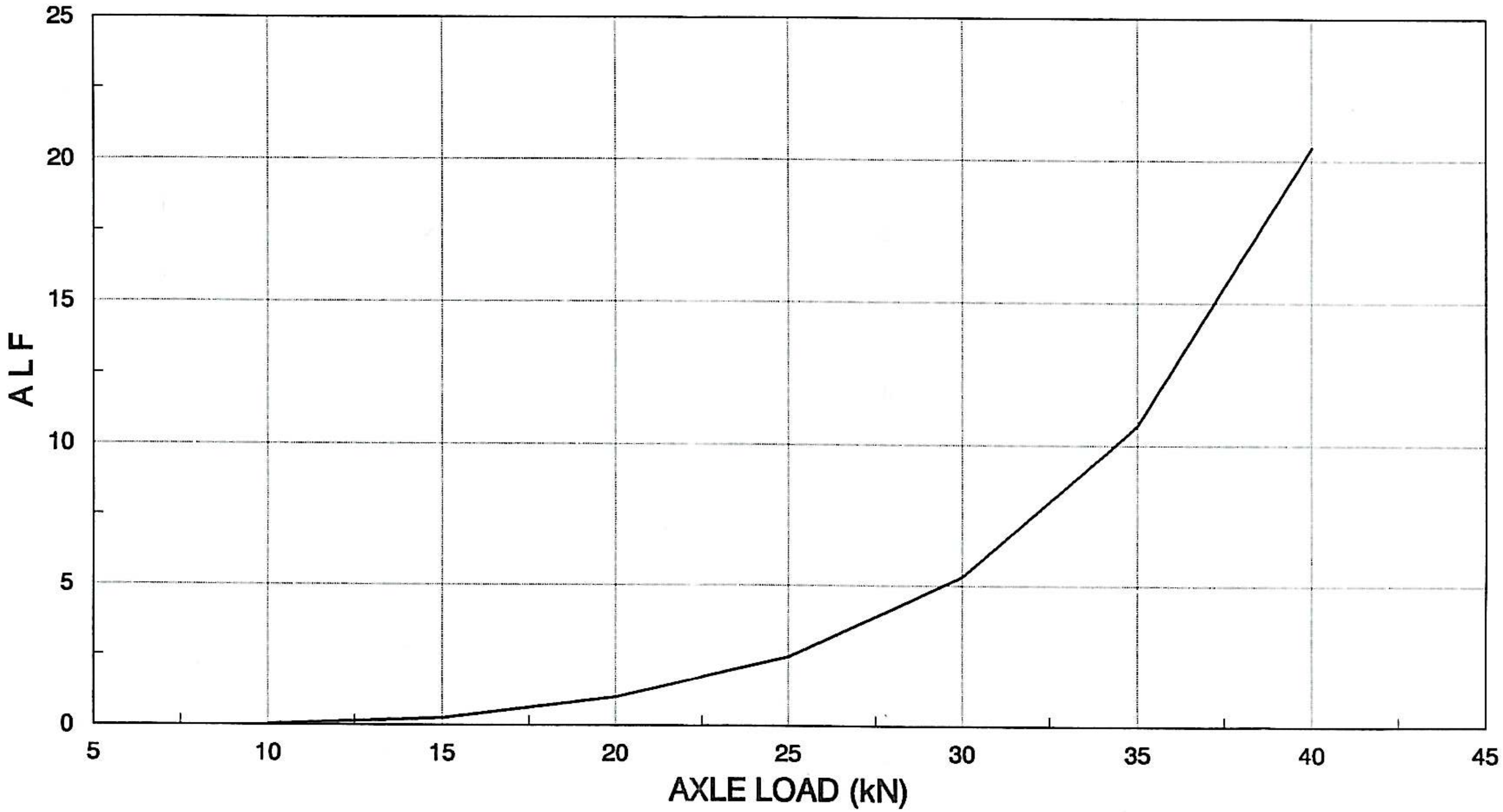


FIGURE C46: Calculated Axle Equivalent Factors (ALF) for different axle loads on Pavement F

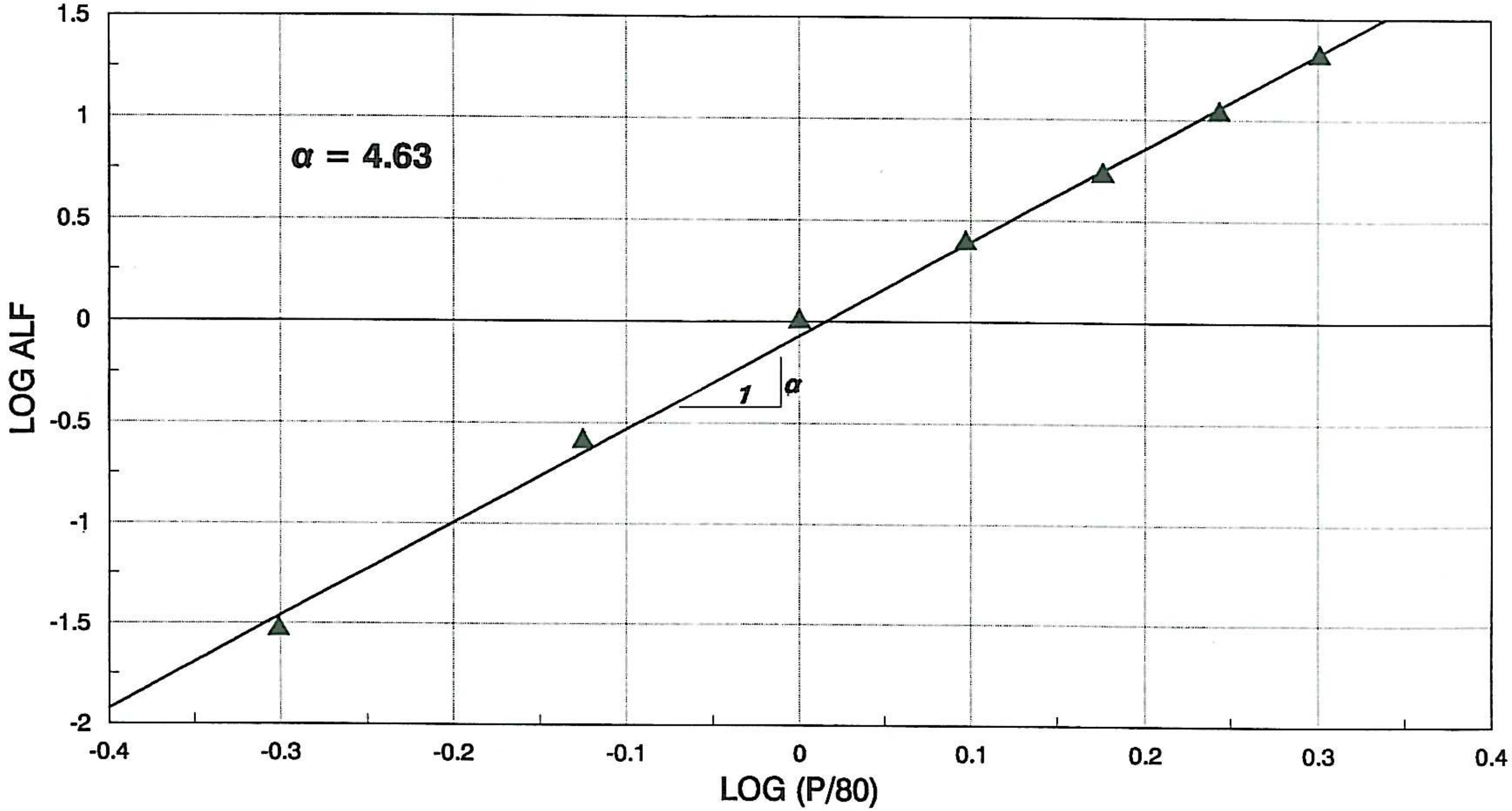


FIGURE C47: Load damage coefficient 'α' for Pavement F as determined by regression analysis of the calculated Axle Load Factors (ALF)

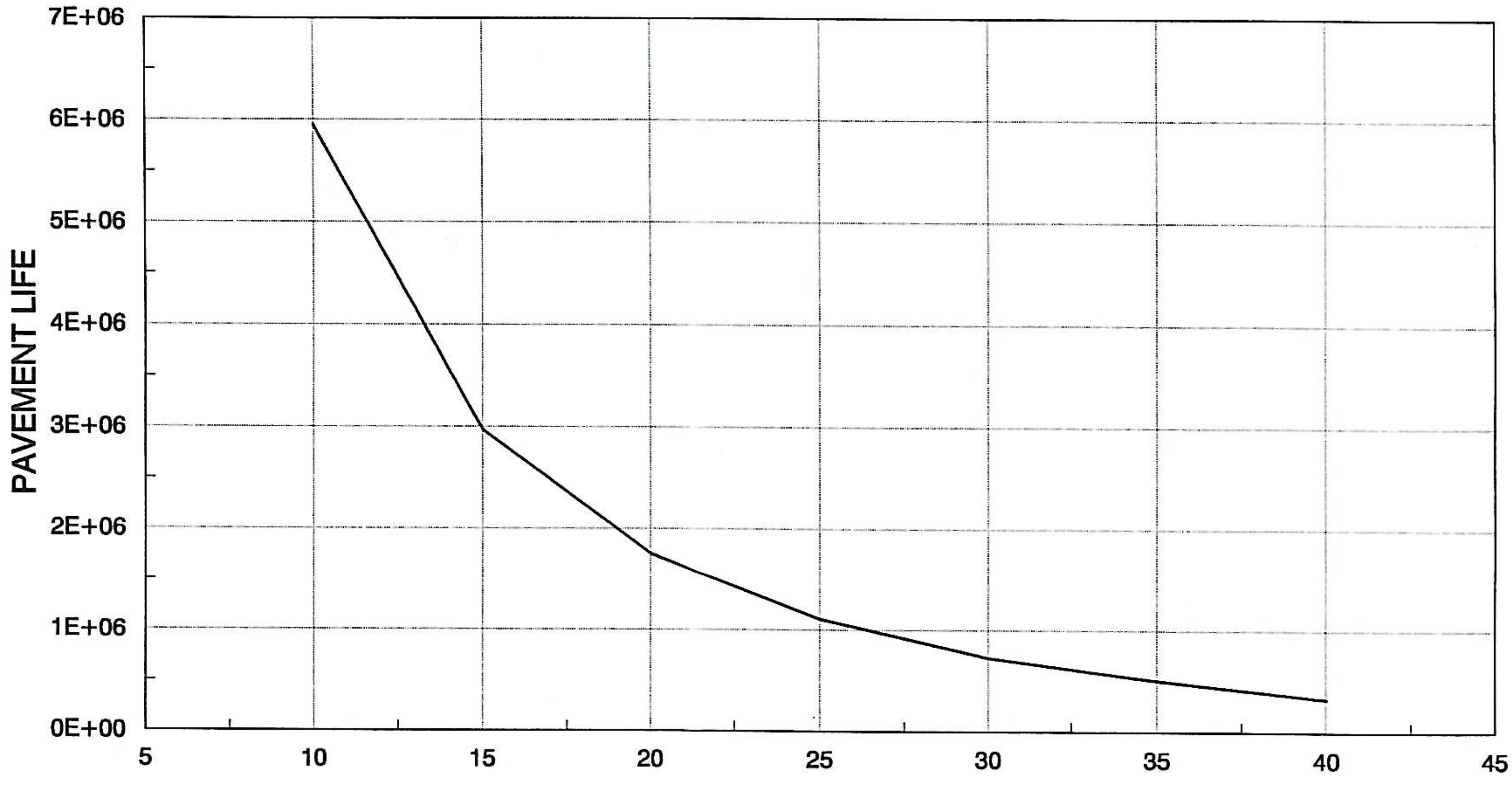


FIGURE C48: Calculated pavement life for a single wheel axle configuration on Pavement F

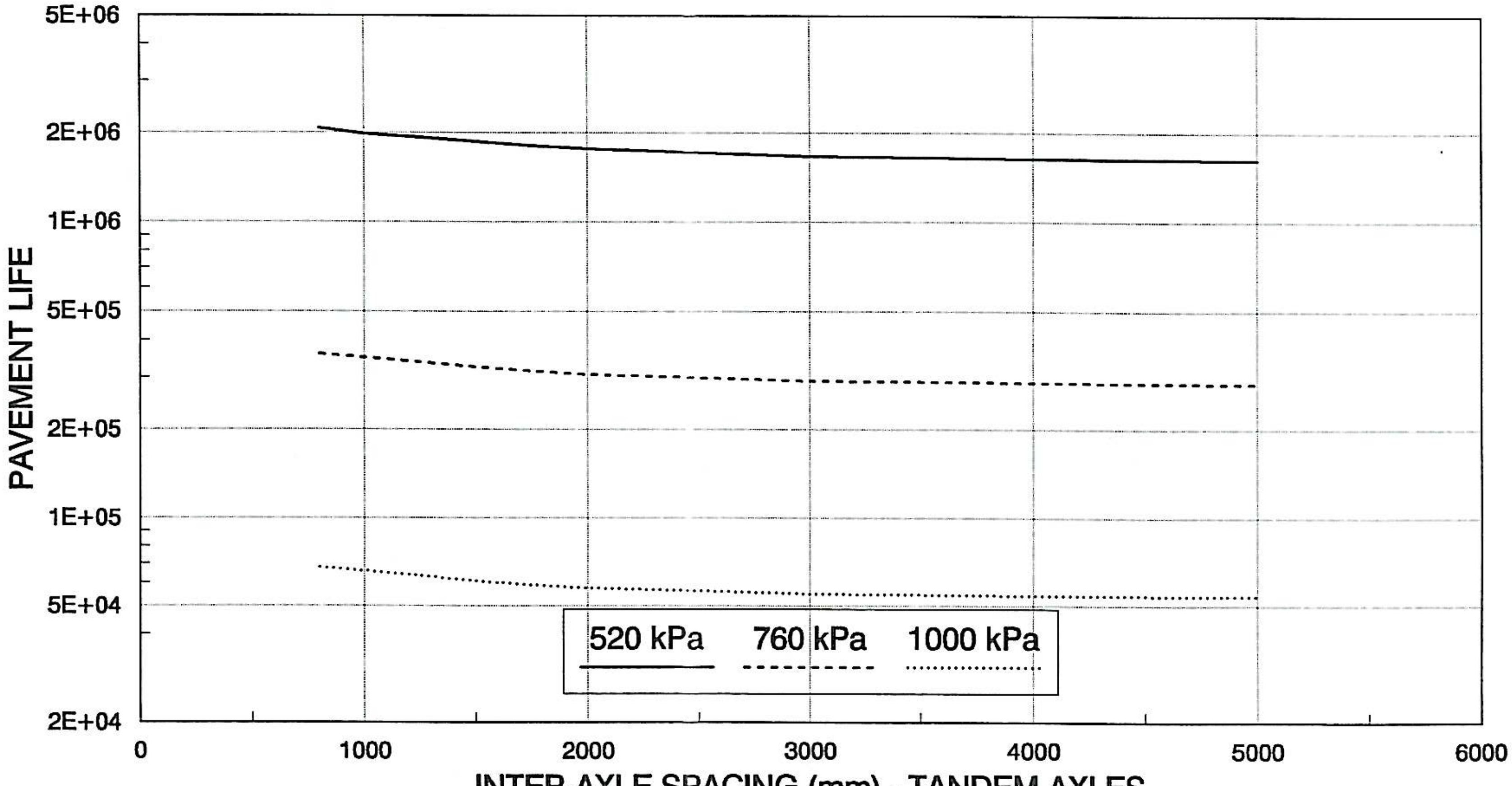


FIGURE C49: Calculated pavement life for different inter-axle spacing for tandem axles on Pavement G (cemented base)

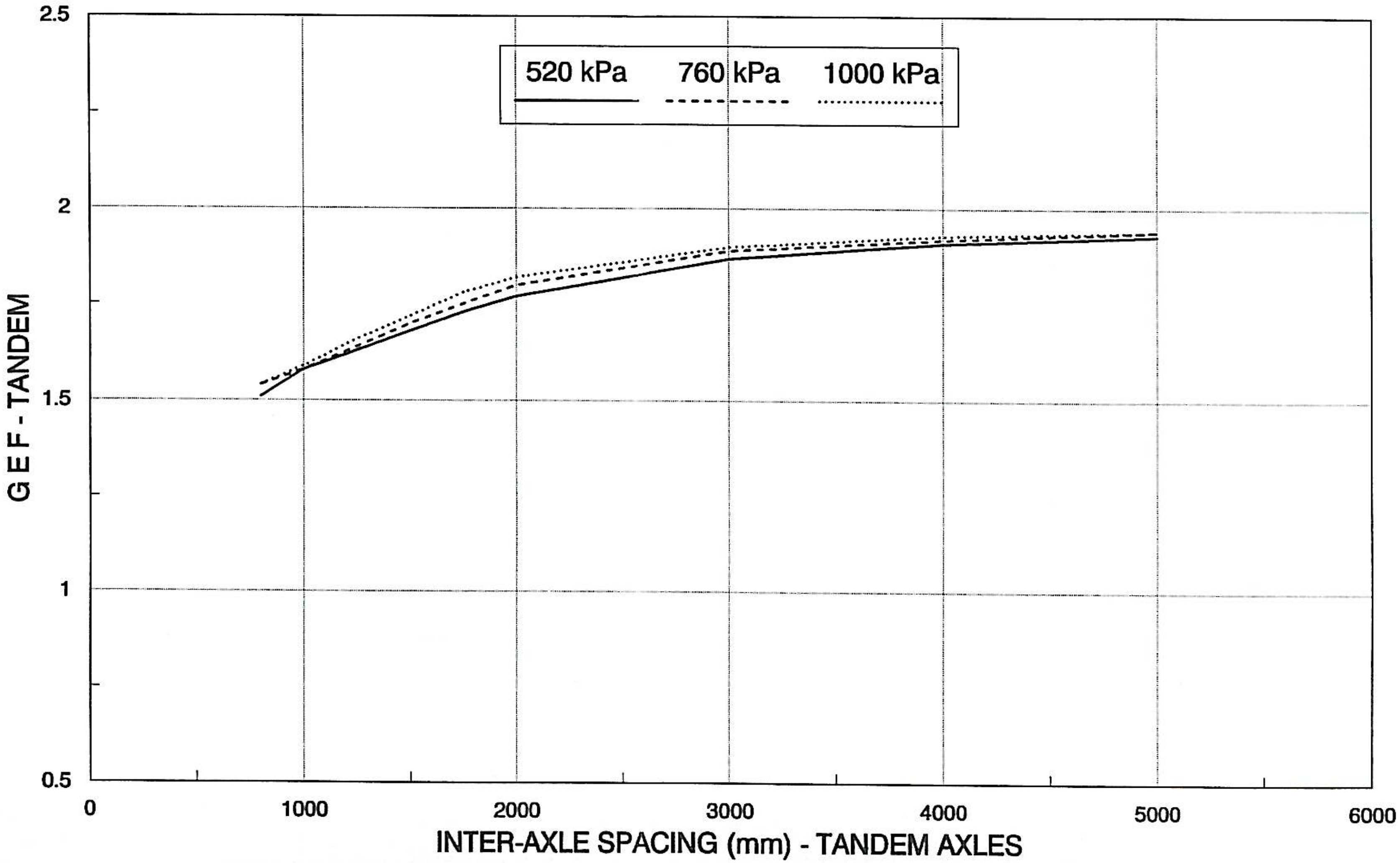


FIGURE C50: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tandem axles on Pavement G

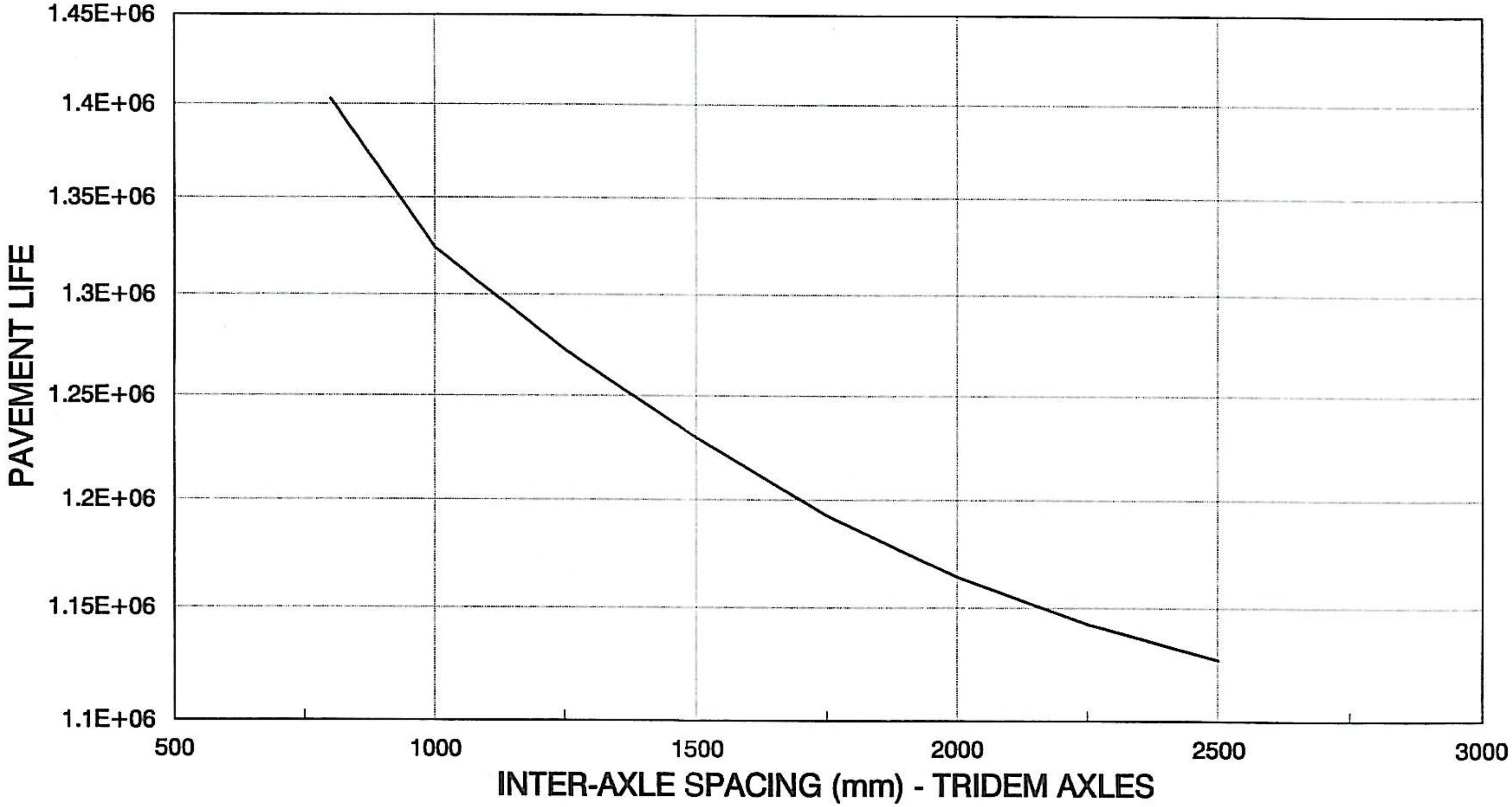


FIGURE C51: Calculated pavement life for different inter-axle spacing for tridem axles on Pavement G (cemented base)

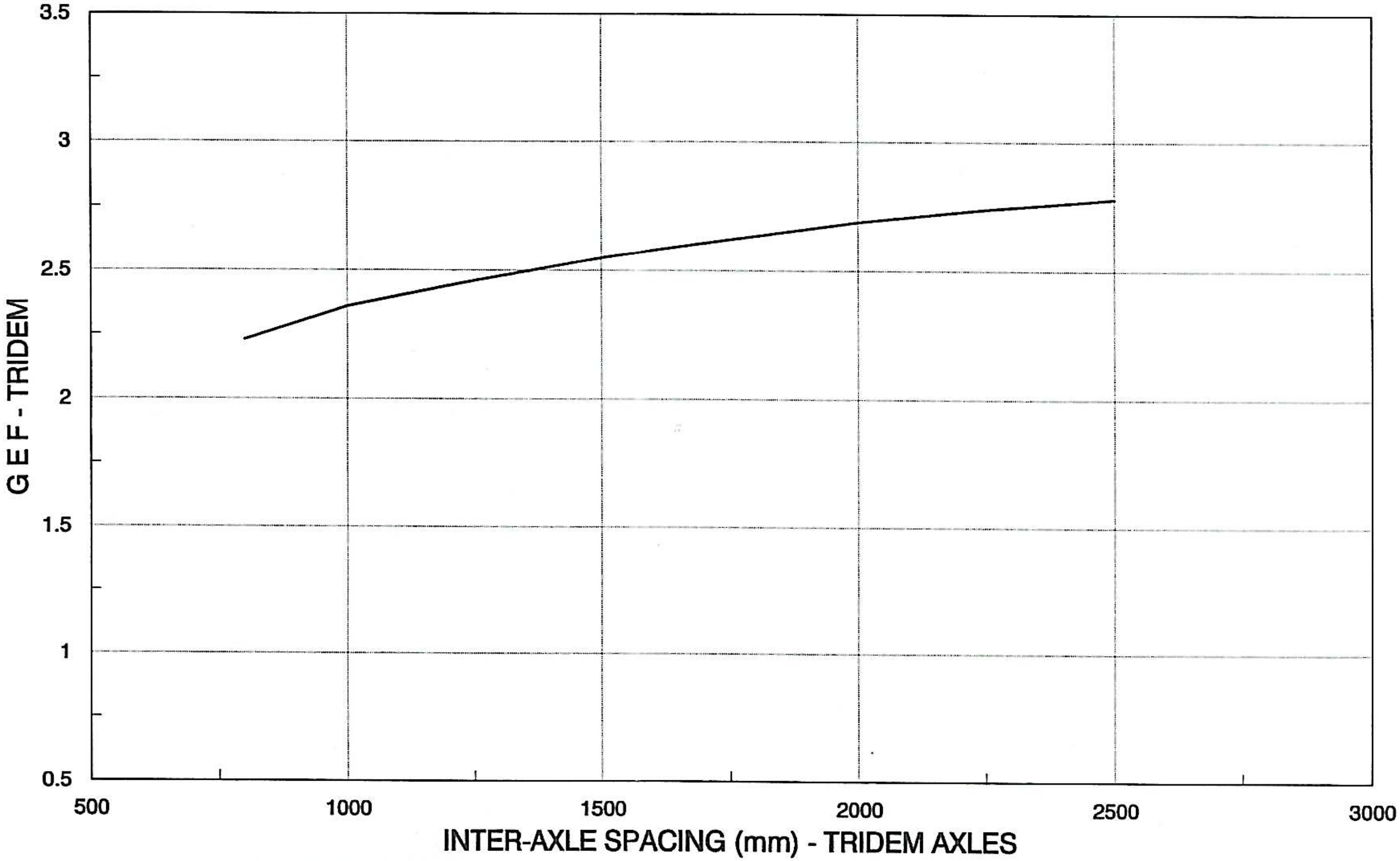


FIGURE C52: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tridem axles on Pavement G

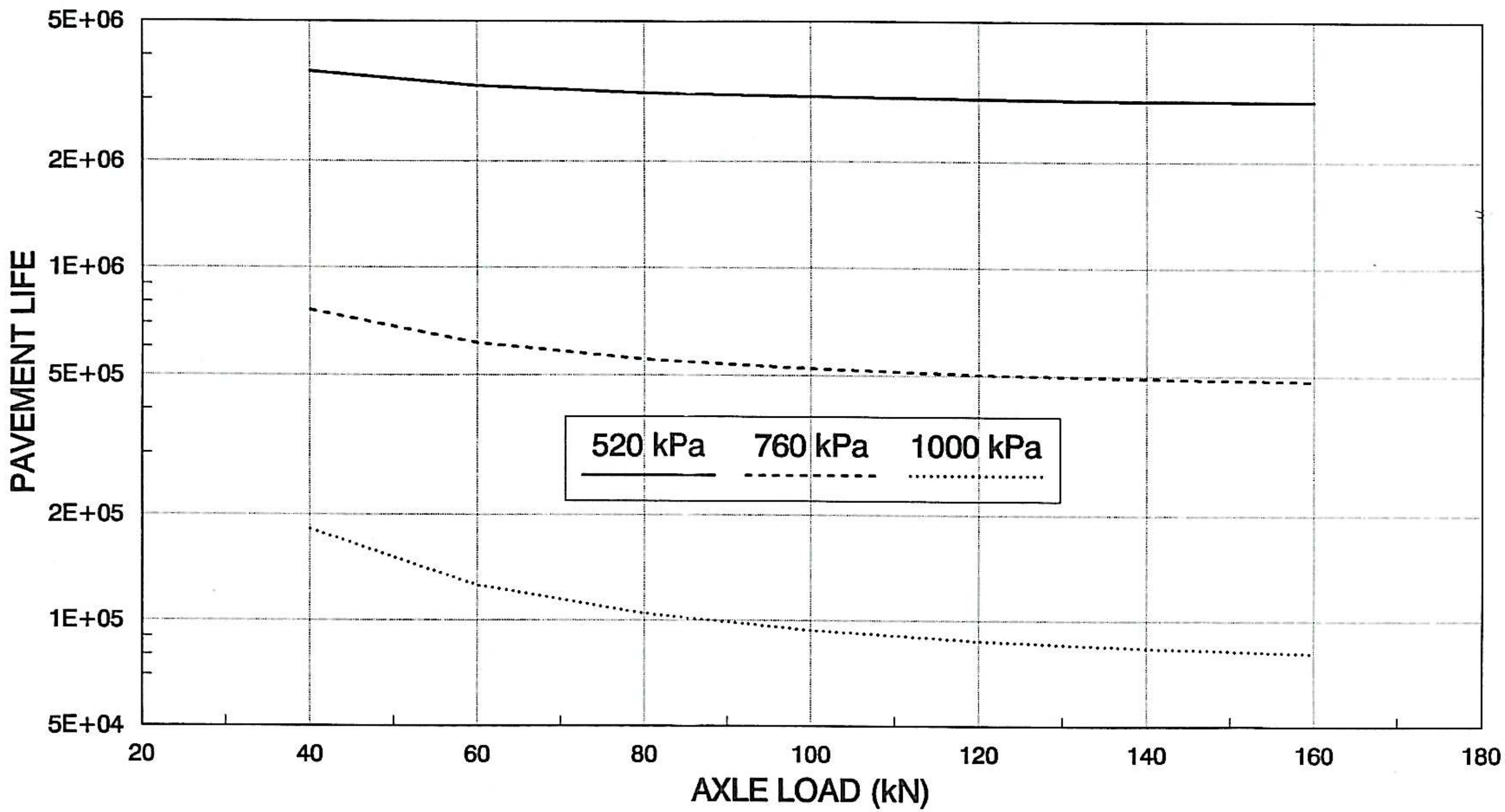


FIGURE C53: Calculated pavement life under a standard dual-wheel axle for different axle loads on Pavement G

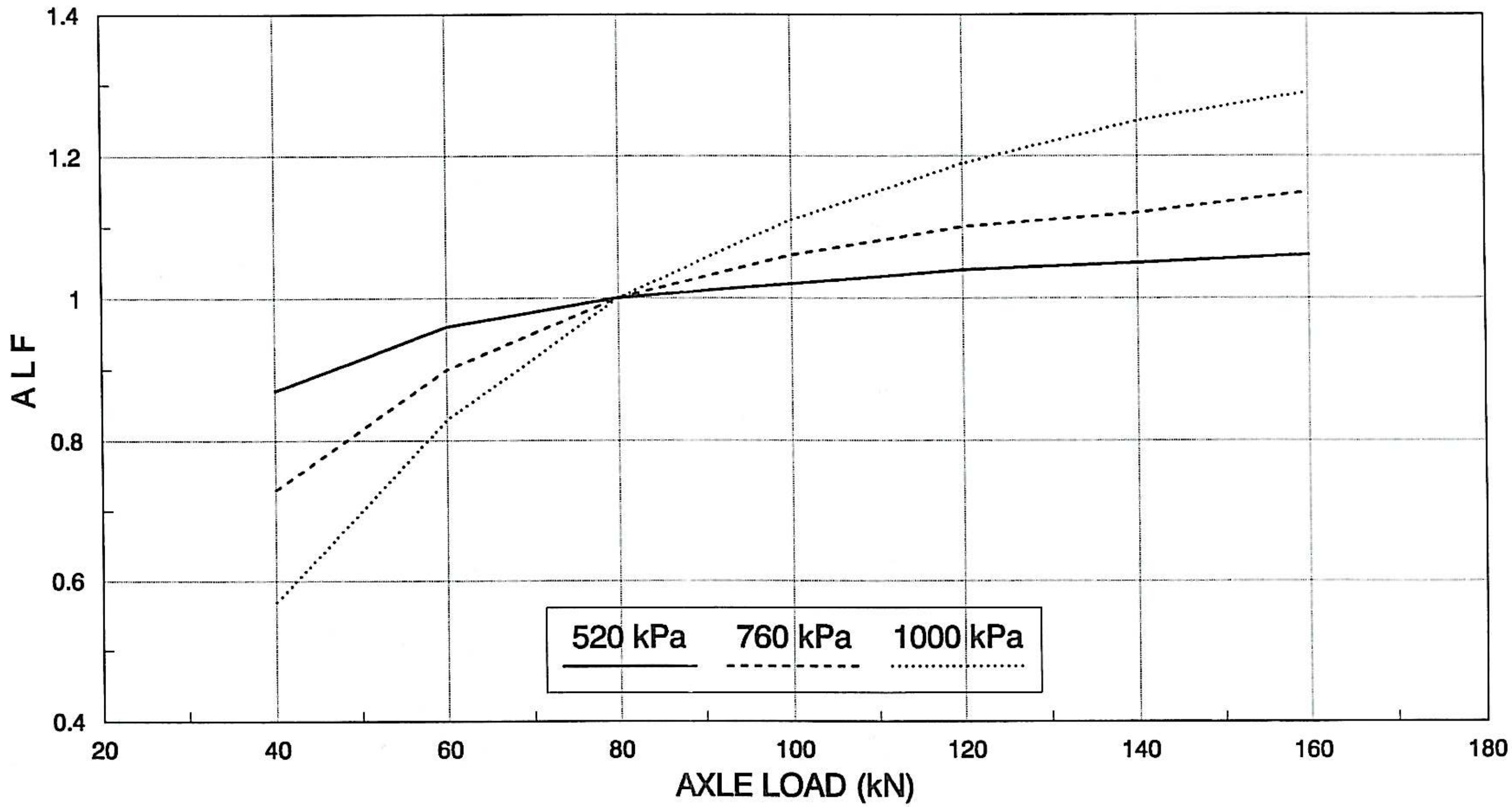


FIGURE C54: Calculated Axle Equivalent Factors (ALF) for different axle loads on Pavement G

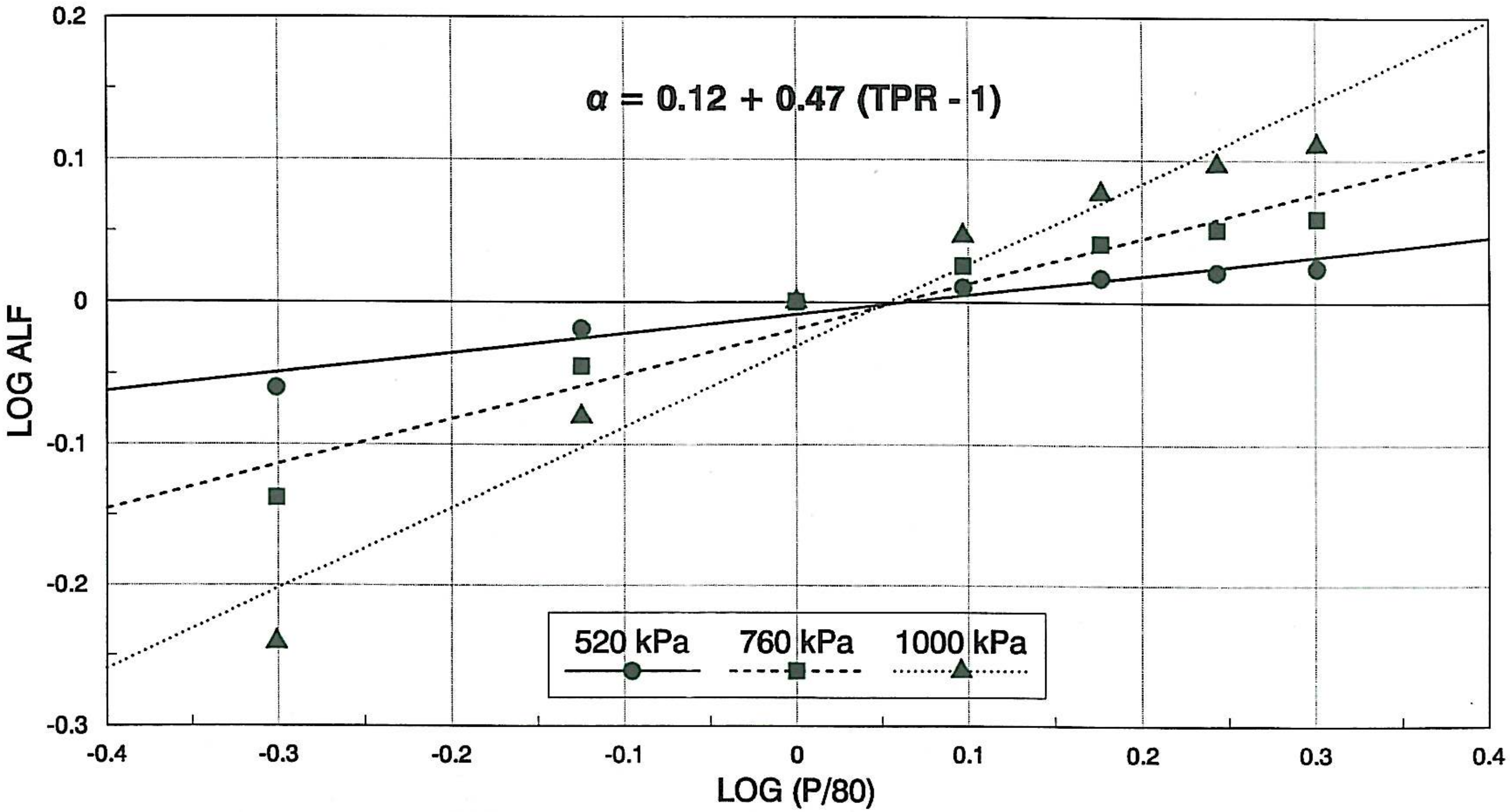


FIGURE C55: Load damage coefficient 'α' for Pavement G as determined by regression analysis of the calculated Axle Load Factors (ALF)

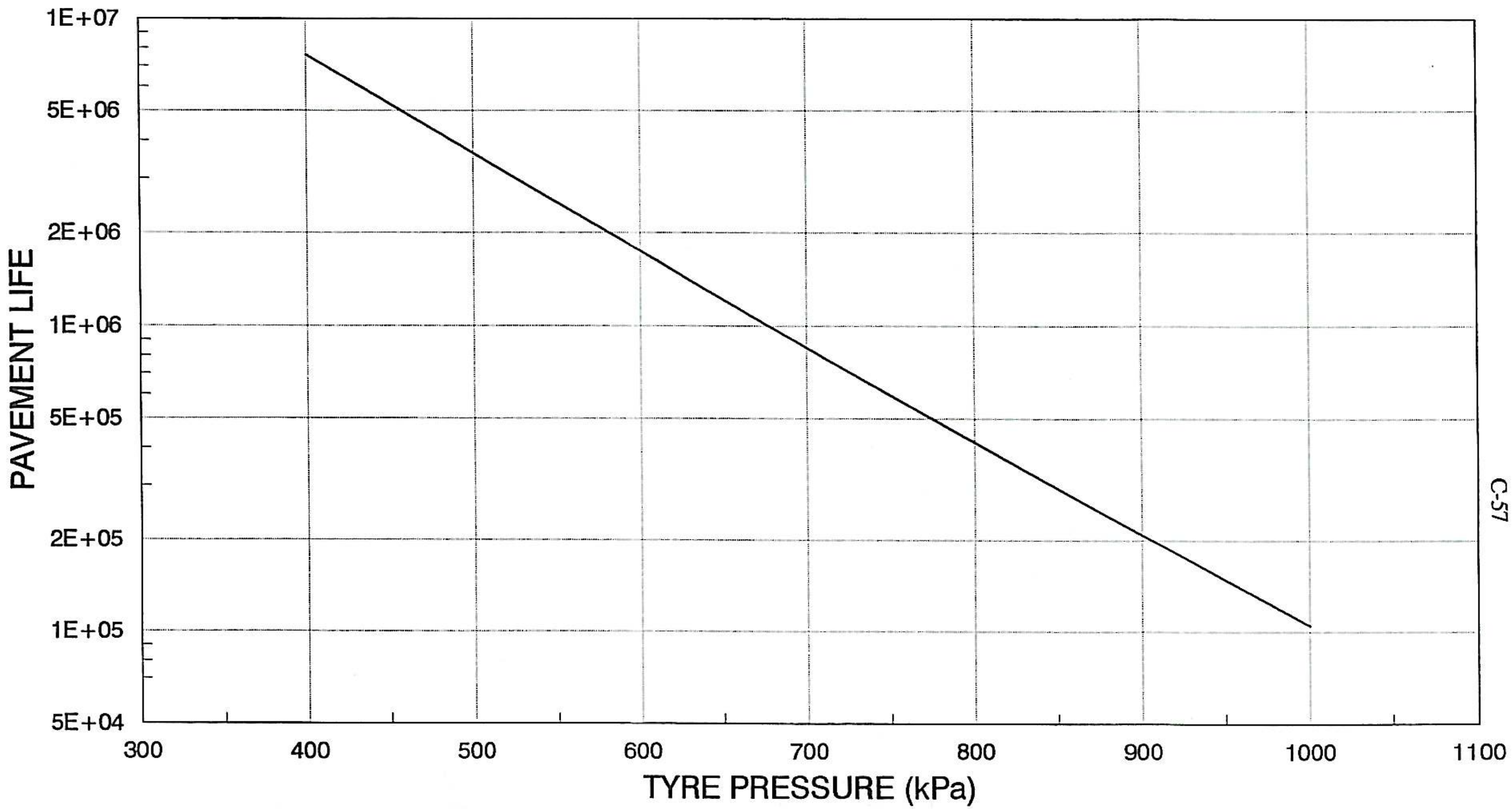


FIGURE C56: Calculated pavement life under a standard 80 kN dual-wheel axle with different tyre pressures

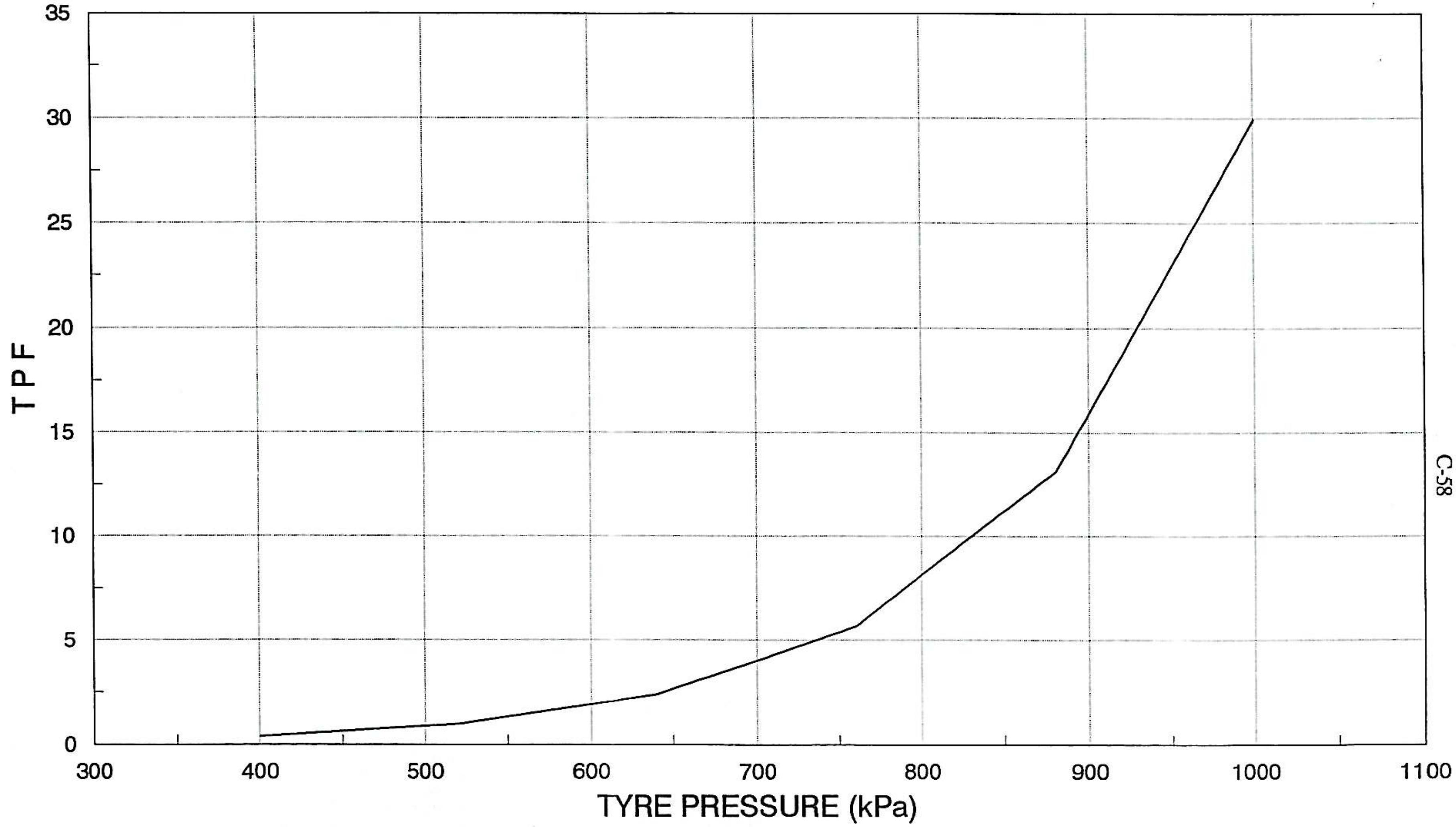


FIGURE C57: Calculated Tyre Pressure Factors (TPF) for different tyre pressures on Pavement G

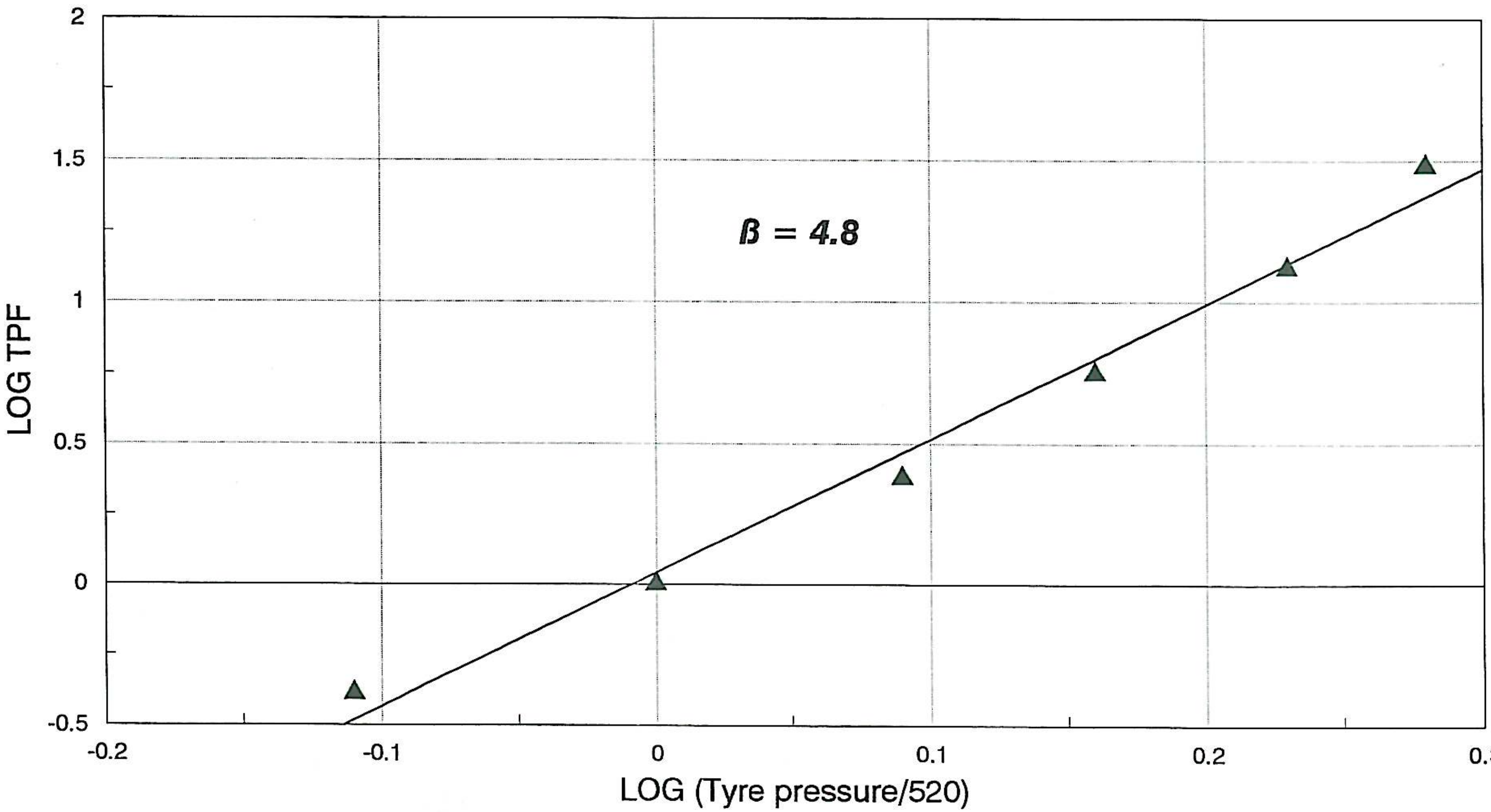


FIGURE C58: Pressure damage coefficient 'ß' for Pavement G as determined by regression analysis of the calculated Tyre Pressure Factors (TPF)

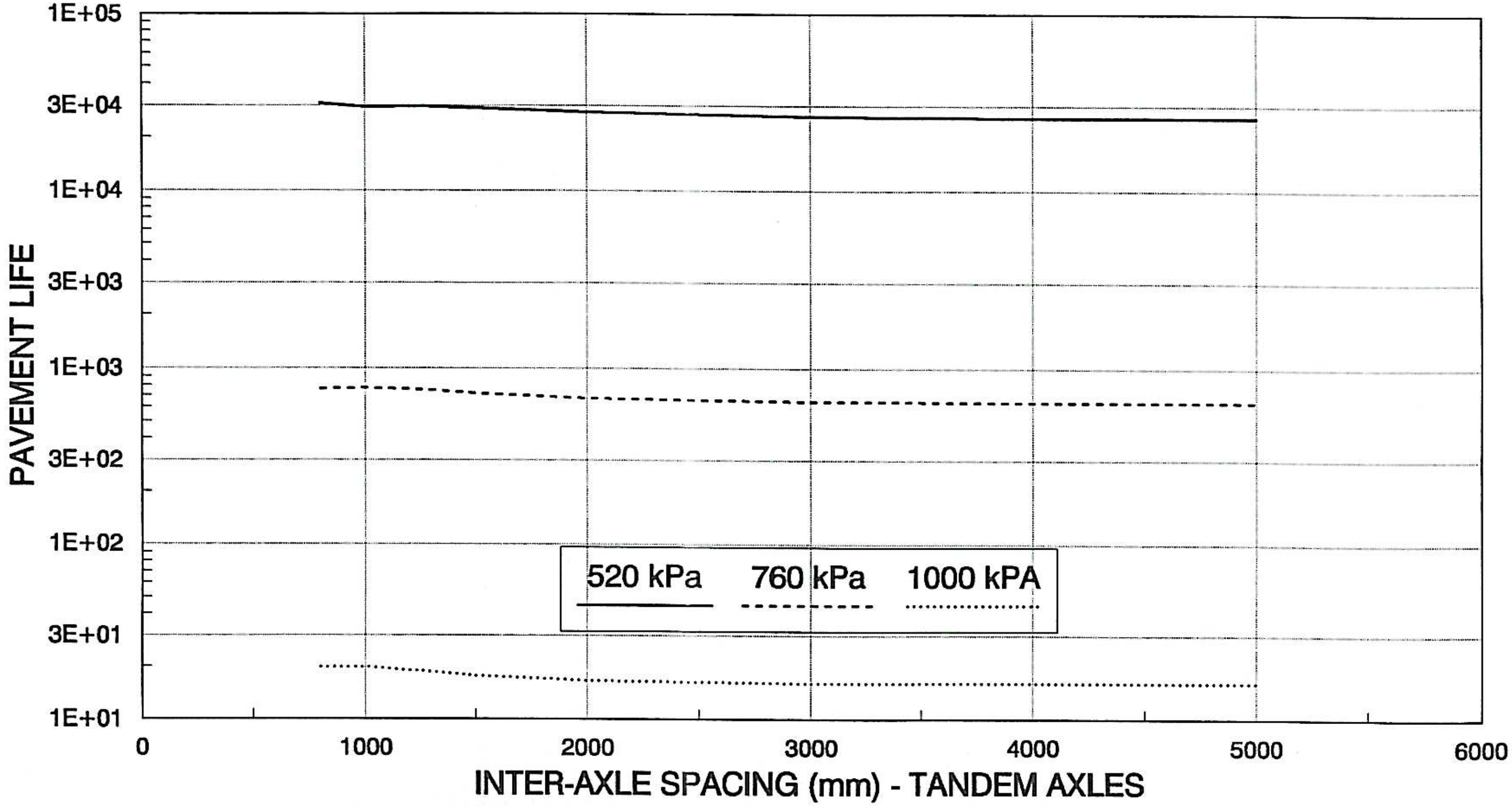


FIGURE C59: Calculated pavement life for different inter-axle spacing for tandem axles on Pavement H (cemented base)

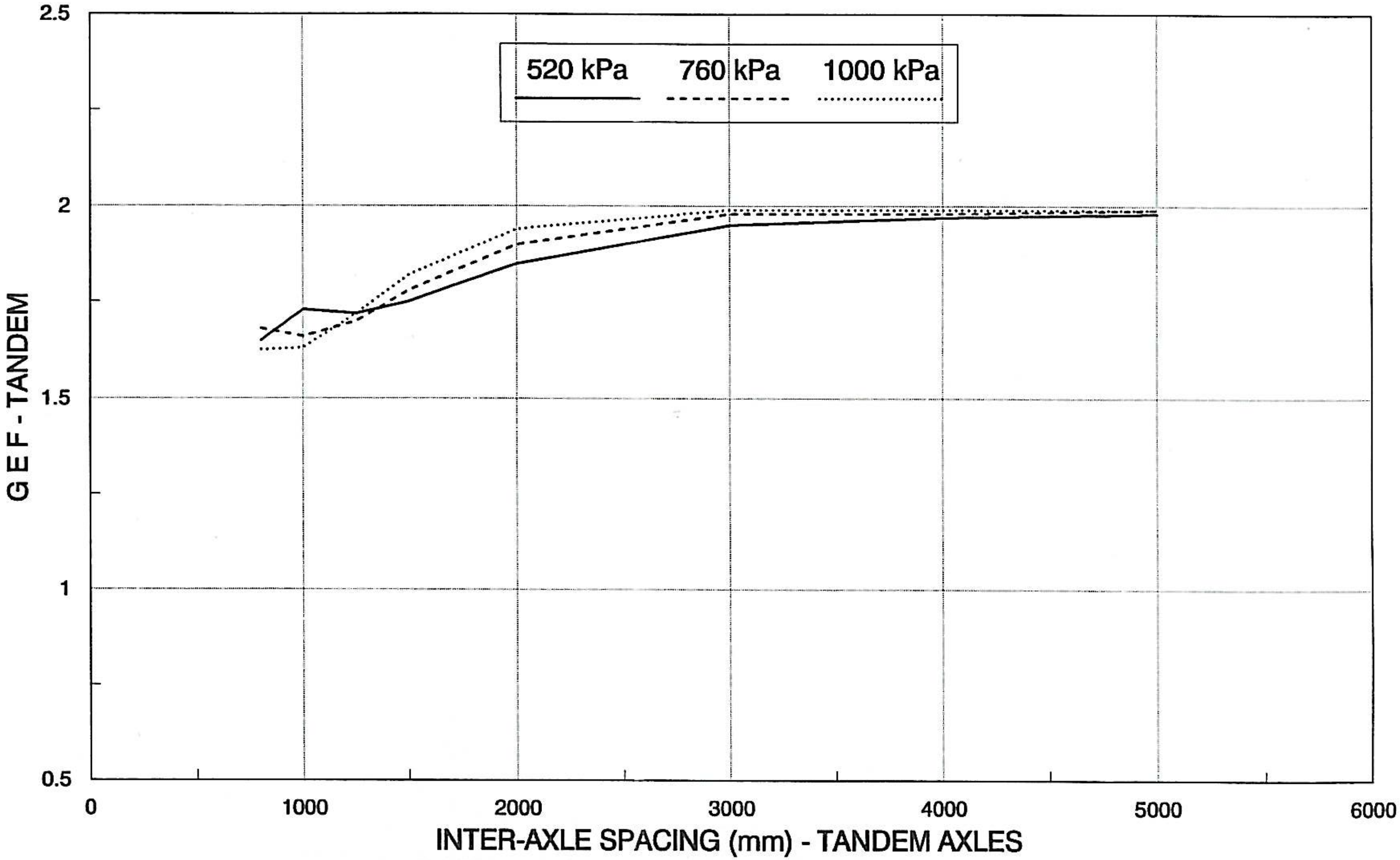


FIGURE C60: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tandem axles on Pavement H

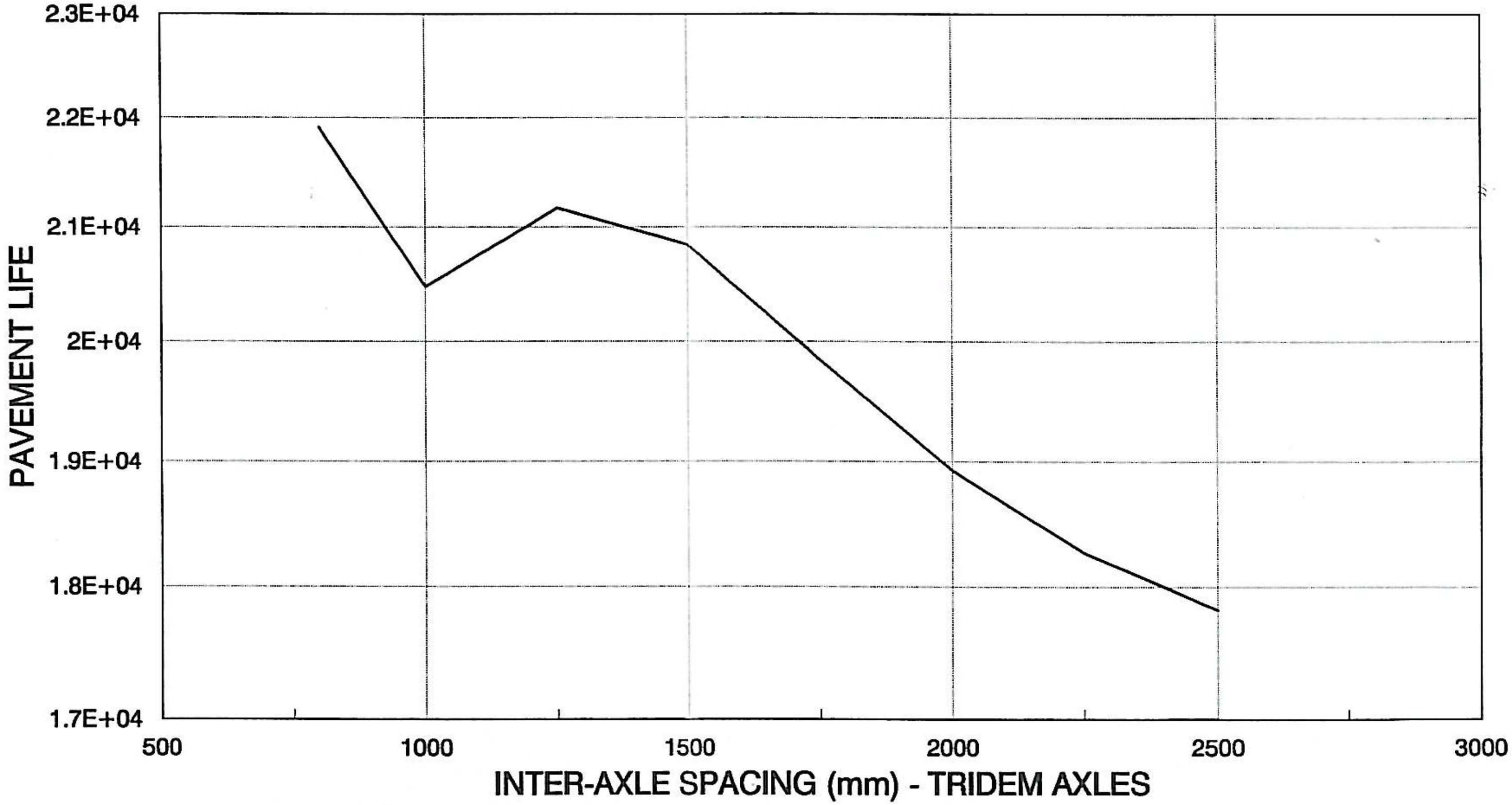


FIGURE C61: Calculated pavement life for different inter-axle spacing for tridem axles on Pavement H (cemented base)

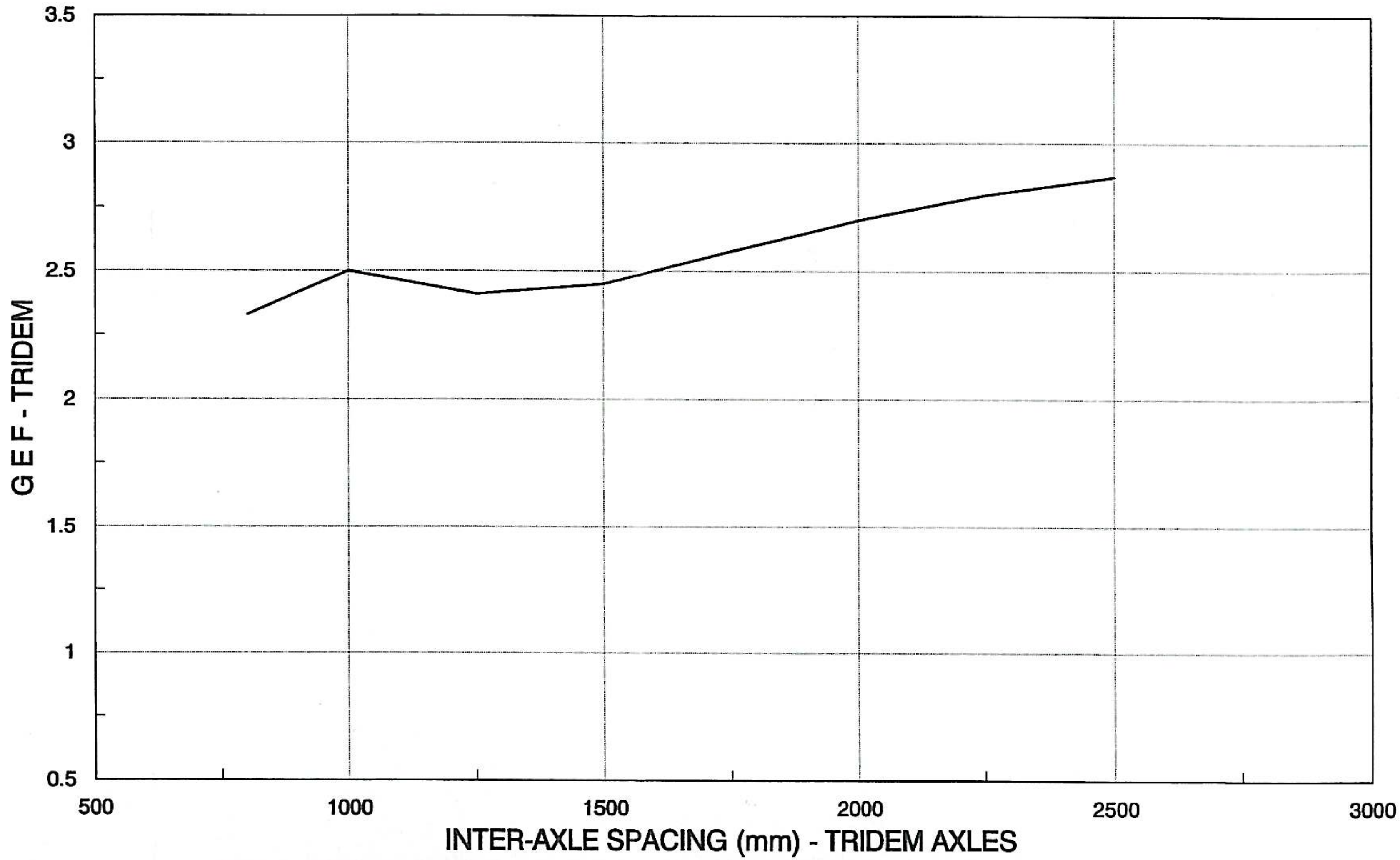


FIGURE C62: Calculated Group Equivalency Factors (GEF) for different inter-axle spacing for tridem axles on Pavement H

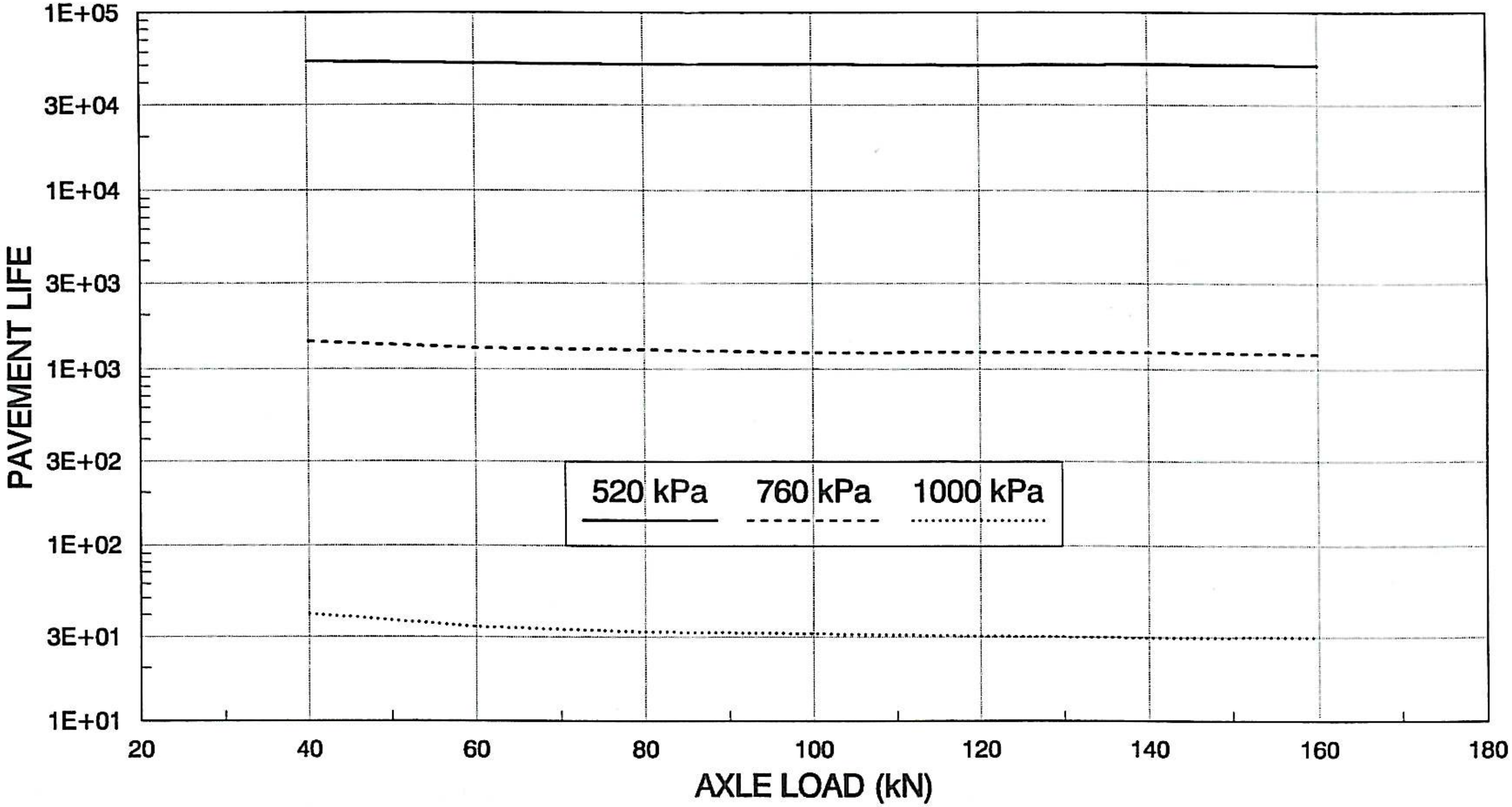


FIGURE C63: Calculated pavement life under a standard dual-wheel axle for different axle loads on Pavement H

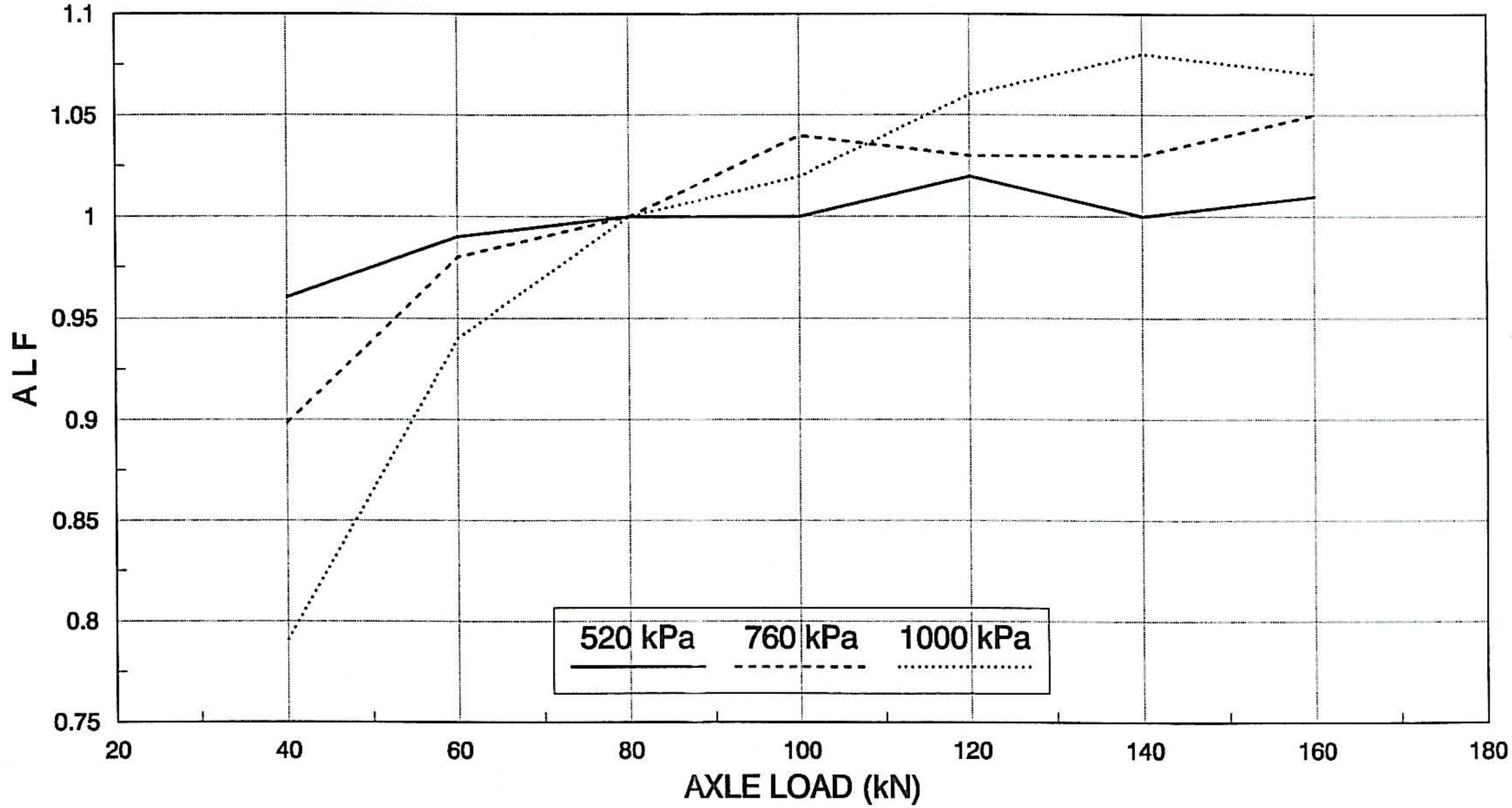


FIGURE C64: Calculated Axle Equivalent Factors (ALF) for different axle loads on Pavement H

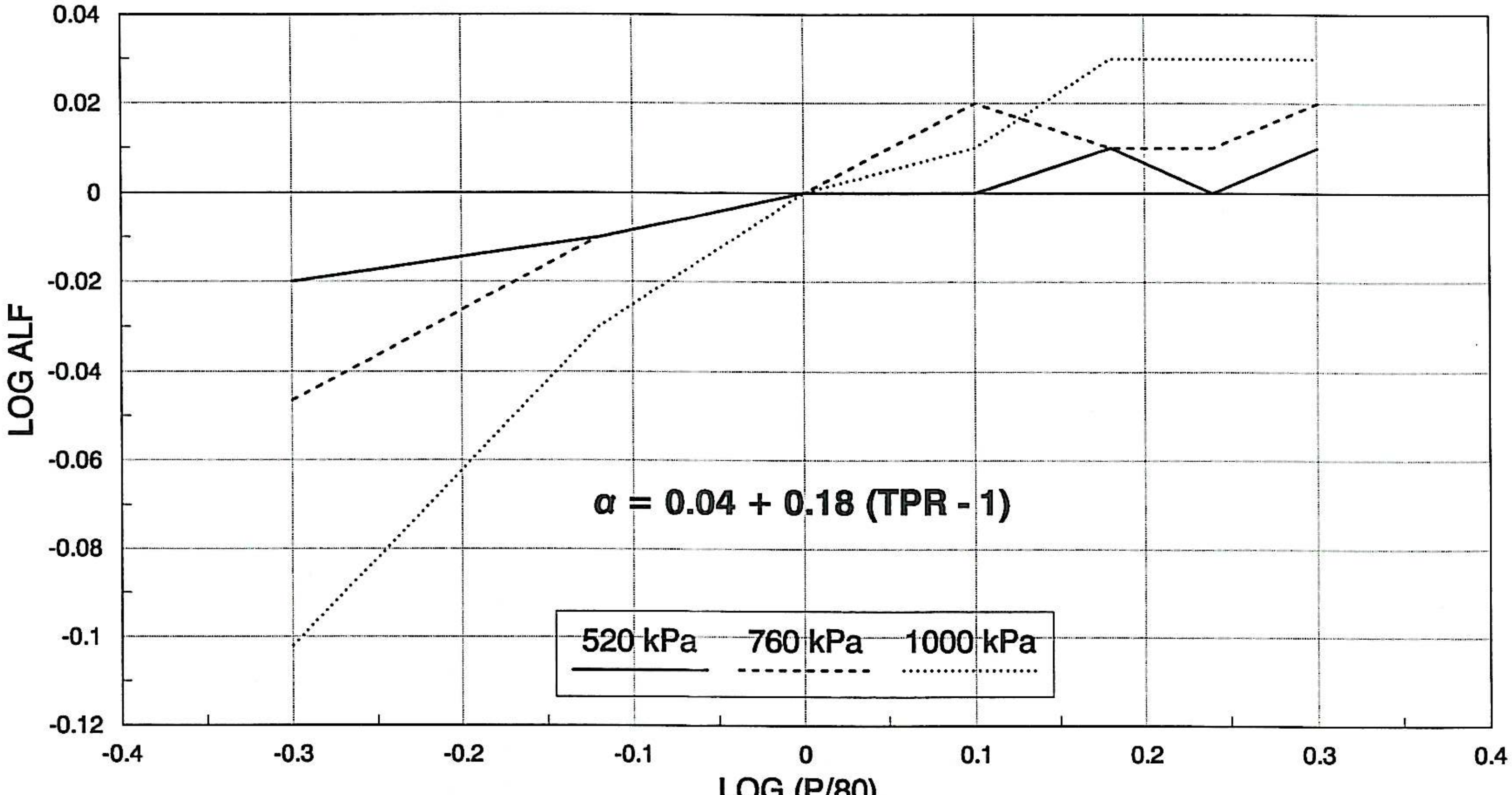


FIGURE C65: Load damage coefficient 'α' for Pavement H as determined by regression analysis of the calculated Axle Load Factors (ALF)

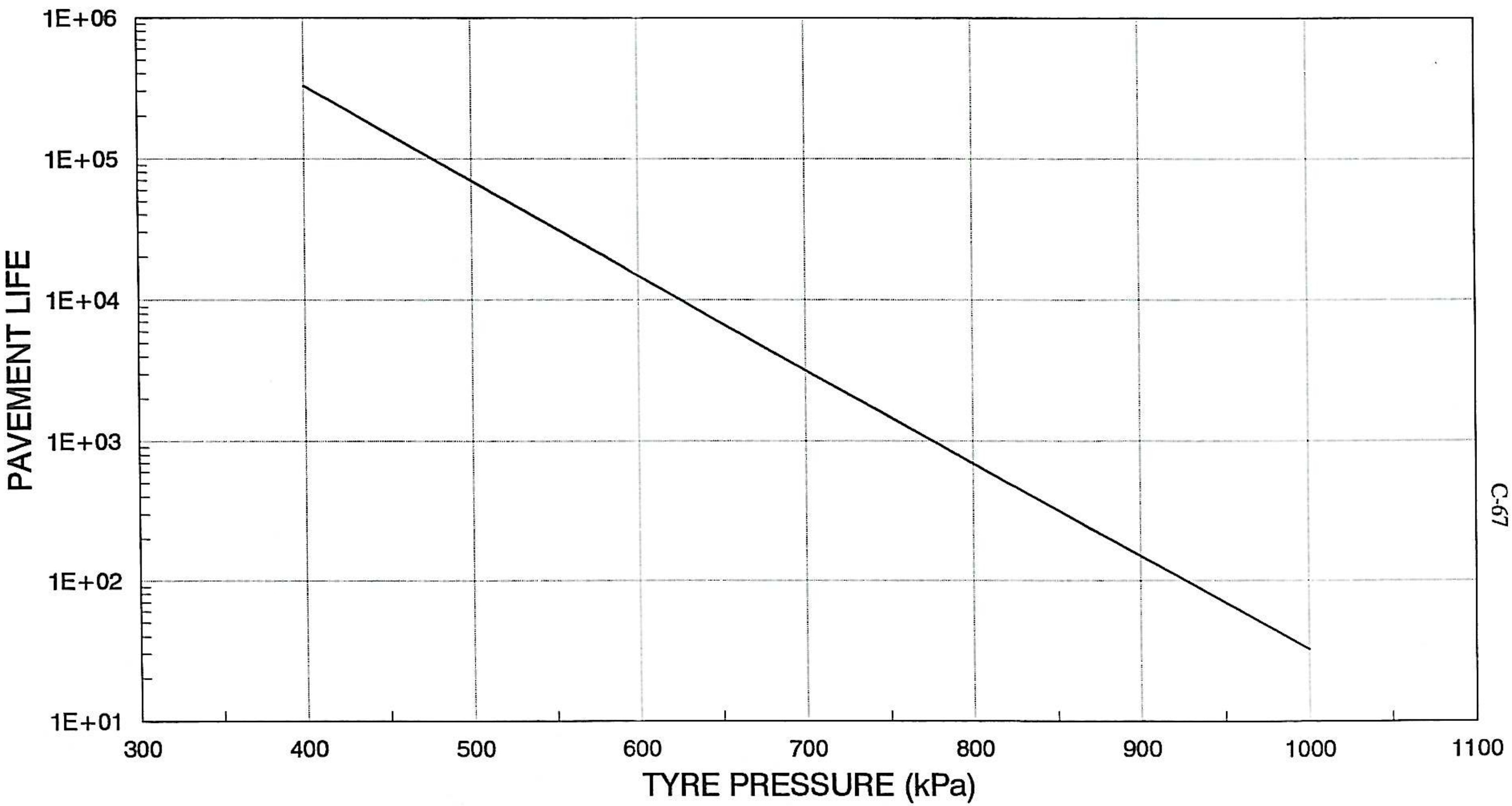


FIGURE C66: Calculated pavement life under a standard 80 kN dual-wheel axle with different tyre pressures

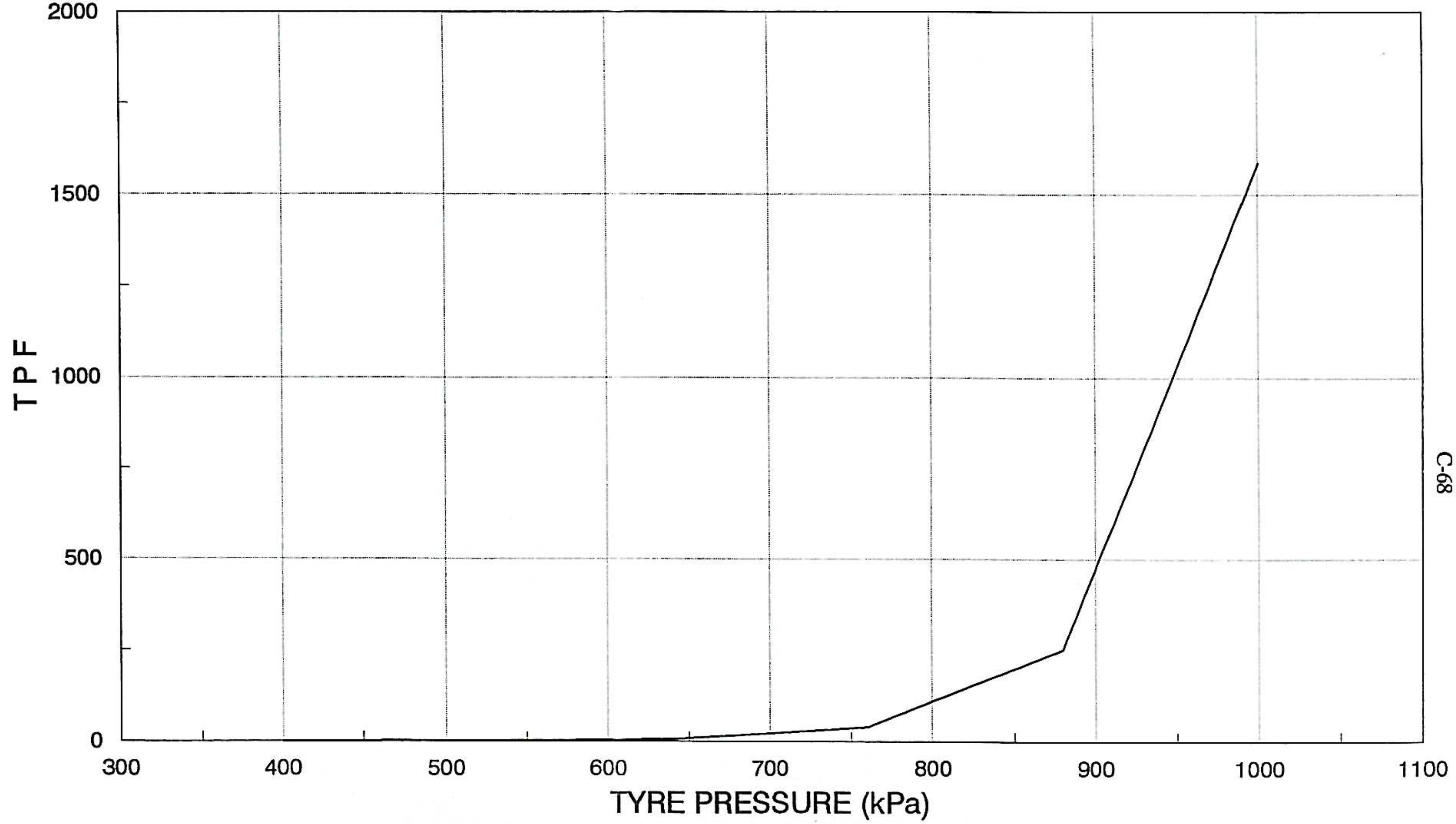


FIGURE C67: Calculated Tyre Pressure Factors (TPF) for different tyre pressures on Pavement H

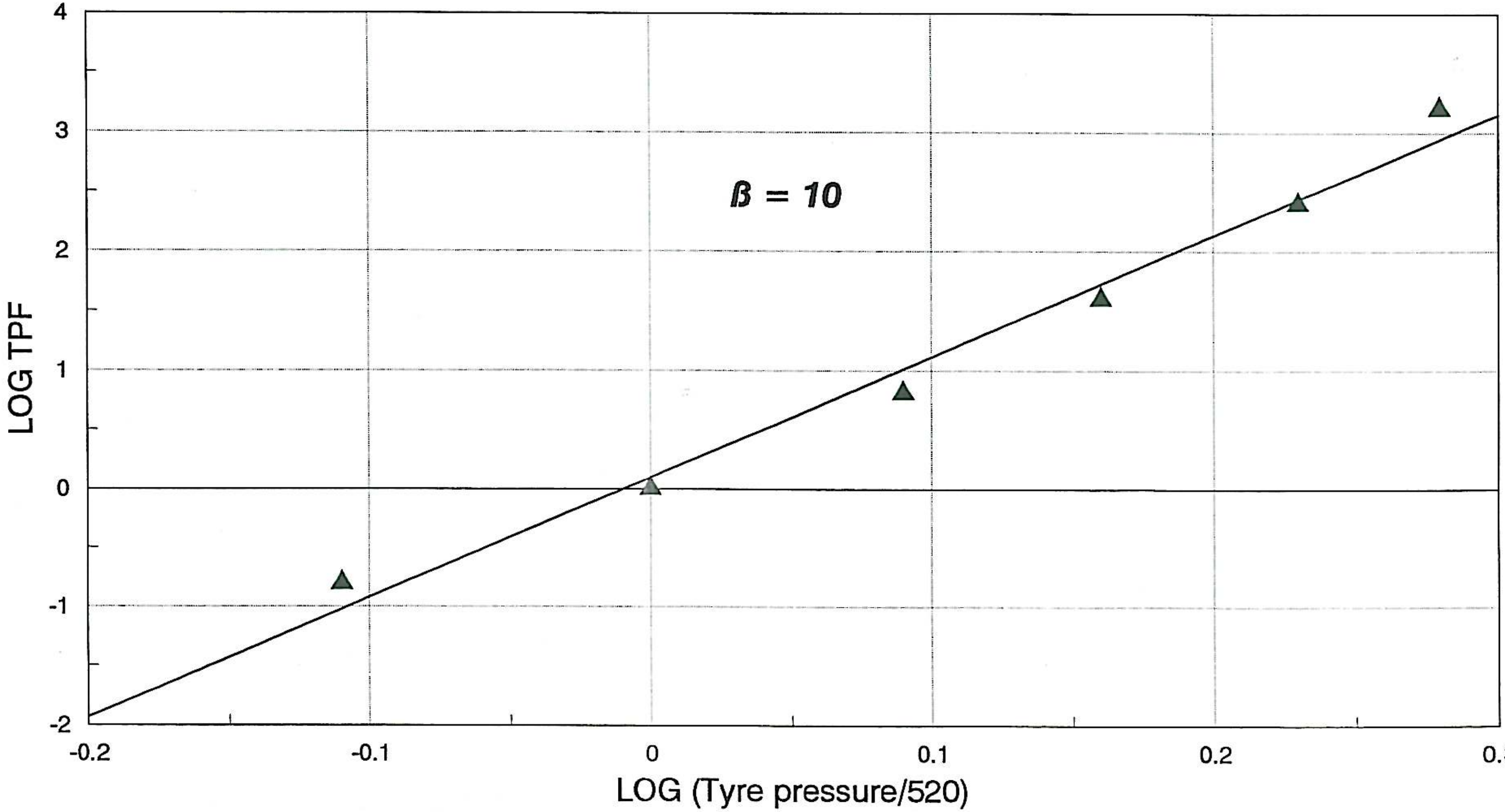


FIGURE C68: Pressure damage coefficient 'ß' for Pavement H as determined by regression analysis of the calculated Tyre Pressure Factors (TPF)