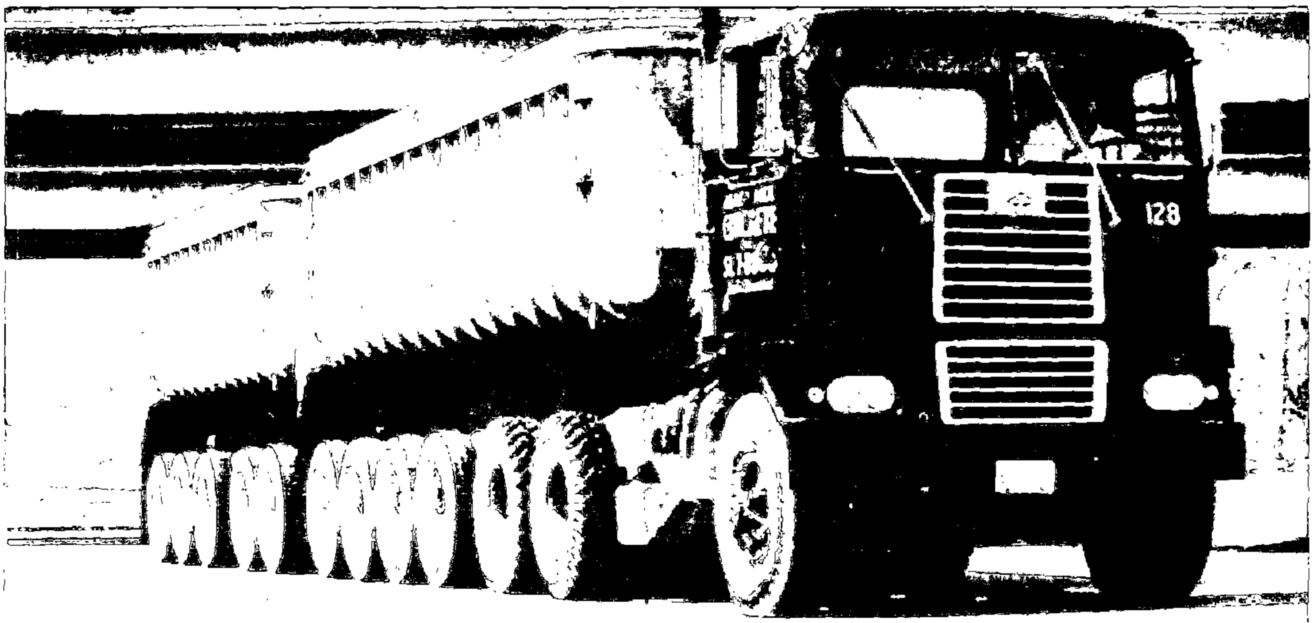
Impact of Truck Characteristics on Pavements: Truck Load Equivalency Factors

FHWA-RD-91-064

July 1992



Trailer train - This 12-axle tractor trailer train hits the road in Michigan

First 12-axle trailer train hauls aggregates

U.S. Department of Transportation
Federal Highway Administration

Office of Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

FOREWORD

This report (FHWA-RD-91-064) presents the findings of a research study concerning the development of axle level equivalency factors derived from the primary responses of flexible pavements.

In this study, a number of primary response equivalency factor methods were evaluated and several selected for further study. Deflection and strain pavement response measurements were measured at the Federal Highway Administration's test pavement facility at the Turner-Fairbank Highway Research Center in McLean, Virginia. These data were evaluated over an experimental factorial of axle type, axle load, tire pressure, speed, pavement thickness, and pavement temperature. Primary response load equivalencies were calculated using the selected methods and a number of statistical comparisons were made. Results of the study indicate that the concept of primary response truck load equivalency factors is viable and can be extremely useful for estimating load equivalency for pavement design and research purposes. Recommendations are also made for use of primary response load equivalencies and for further research into the subject.

This report will be of interest to researchers and engineers concerned with the relative frictional performance of various types of truck tires in a series of controlled tests.

Sufficient copies of this report are being distributed by FHWA memorandum to provide two copies to each FHWA Region, and three copies to each FHWA Division and State highway agency. Direct distribution is being made to the Division Offices. Additional copies for the public are available from the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, Virginia 22161.

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Research and Development

NOTICE

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16. Abstract

Truck load equivalency factors relate vehicle axle loads to pavement damage and life. They provide a method to convert the effects of mixed traffic to a standard loading condition. Equivalent loadings for most pavement design and evaluation procedures are currently predicted using the AASHTO method of equivalency factors. This study was undertaken to evaluate various types of primary pavement response derived load equivalency factors. These use pavement response measurements such as strain and deflection to estimate the equivalent damaging effect of any axle loading condition. These types of factors are also expressed in terms of a relative number of equivalent standard axle loads.

A number of primary response equivalency factor methods were evaluated and several selected for further study. Deflection and strain pavement response measurements were evaluated over an experimental factorial of axle type, axle load, tire pressure, speed, pavement thickness, and pavement temperature. Primary response load equivalencies were calculated using the selected methods and a number of statistical comparisons were made.

Results of the study indicate that the concept of primary response truck load equivalency factors is viable and can be extremely useful for estimating load equivalence for pavement design and research purposes. Results also indicate which of the vehicle and pavement factors studied most significantly effect the estimate of load equivalency. Recommendations are also made for use of primary response load equivalencies and for further research into the subject.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C
		Illumination		
fc	foot-candles	10.76	lux	lx
fL	foot-Lamberts	3.426	candela/m^2	cd/m^2

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
		AREA		
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
		VOLUME		
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		Illumination		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fL

* SI is the symbol for the International System of Measurement

(Revised July 1989)

TABLE OF CONTENTS

TABLE OF CONTENTS
(continued)

TABLE OF CONTENTS
(continued)

LIST OF FIGURES

LIST OF FIGURES
(continued)

LIST OF FIGURES
(continued)

LIST OF TABLES

LIST OF TABLES

LIST OF ABBREVIATIONS AND SYMBOLS

AASHO . . . American Association of State Highway Officials
AASHTO . . American Association of Highway & Transportation Officials
AC . . . . asphalt concrete
ANOVA . . . . . analysis of variance
ASTM . . . . . American Society for Testing and Materials

CF . . . . calibration factor

DCDT . . . . direct current linear variable differential transformer
df . . . . degree of freedom

ESAL . equivalent single axle load

FHWA . . . . Federal Highway Administration

GF . . . . . . strain gauge factor

HMAC . . . hot mix asphalt concrete

INST . . . . instrument

LEF . . . . load equivalency factor
LOD . . . . load
LVDT . . . . linear variable differential transformer

MF . . . . . multiplication factor
MS . . . . mean square

NS . . . . not significant

PCC . . . . portland cement concrete
PVMT . . . . pavement

REF . . response equivalency factor
Ref. . . . . reference
RS . . . . roadbed soil

SEF . . . stress equivalency factor
Sig . . . . significance
SLD . . . single layer deflectometer
SN . . . . structural number
SPD . . . . speed
SPSS . . . Statistical Package for the Social Sciences
SS . . . . sum of squares

TEMP . . . . temperature
TP . . . . tire pressure
TRK . . . . truck

VDC . . . . . volts - direct current

WL . . . . wheel load
WLEF . . . . wheel load equivalency factor

SECTION 1. INTRODUCTION

PROBLEM STATEMENT

The effects of vehicle size, weight, and configuration on pavement performance and maintenance requirements must ultimately be evaluated by means of an economic comparison. This evaluation is possible only if the effects of current vehicle parameters can be related to pavement stresses and strains, distresses, and finally overall pavement performance. This approach is the basis for evaluating the effects of various estimates of equivalent single axle loads (ESAL's) on pavement performance.

Equivalent loadings for most pavement design procedures are currently predicted using the American Association of State Highway and Transportation Officials (AASHTO) method of equivalency factors which were derived from the American Association of State Highway Officials (AASHO) Road Test which was performed in the late 1950's and early 1960's.[1] It has been shown recently in a number of studies that these factors may not accurately quantify the effects of many vehicle loading parameters. Vehicle characteristics such as tire pressure, suspension types, axle configurations, axle loads and gross vehicle weights have changed significantly since the AASHO Road Test. This may have the general effect of causing more relative damage to the pavement than the axle loads and overloads used at the AASHO Road Test. It is possible that the equivalency factors developed from the road test tend to underestimate the amount of damage caused to the pavement by modern vehicles.

Since the AASHO Road Test there have been a number of studies to derive new types of load equivalency factors (LEF's) which account for changes in vehicle characteristics and other factors not accounted for in the road test results. Many of these studies have produced new LEF's which are intended to supplement or replace the AASHTO equivalencies currently used by most agencies. These methods of LEF development range from empirical methods using observed loading and distress data to mechanistic models which incorporate pavement response parameters such as stress, strain or deflection to estimate pavement damage. These are known as primary pavement response LEF's. There are many forms of pavement performance and damage models that predict pavement life or distress at various loading conditions in a pavement's life. There is a need to distinguish the most accurate and most reliable sets of pavement performance models and/or LEF's available in the current literature.

The most fundamental problem addressed by the current research effort is to define the most accurate and reliable set of load equivalency relationships for modeling pavement behavior relative to applied loads. These must eventually be related to pavement performance or damage models which accurately predict pavement performance or distress. The specific problem addressed in the current study is the validation of load equivalency relationships developed in the Canadian "Weights and Dimensions Study" and other equivalency relationships which may prove to be more accurate or reliable than the Canadián results.[2]

PROJECT OBJECTIVES AND SCOPE

Project Objectives

The primary objective of this project is to evaluate the accuracy and reliability of various methods of primary pavement response LEF's. The research is evaluating the actual effects of and how the methods account for parameters such as axle load, axle configurations, gross vehicle load, tire pressure, pavement structure, material properties, environmental conditions, and other quantifiable factors. Individual objectives of the project include the following:

- Identify feasible and reasonable methods of determining primary pavement response equivalency factors.
- Develop the concept of using primary response load equivalencies for pavement design, evaluation, or research purposes.
- Verify the Canadian results and other promising methods as to their ability to produce precise, accurate, and reliable LEF's.[2]
- Quantify the effects of independent variables including load, axle configuration, speed, tire pressure, temperature, and pavement structure on load equivalencies from various methods.
- Establish whether vehicle classification is necessary or if axle load and type are adequate for describing traffic for equivalency factor purposes.
- Determine whether primary pavement response based LEF's are a viable concept and if some basic models for predicting primary response LEF's could be developed with additional testing and research.

Project Scope

The scope of the project involved two main aspects. First, a comprehensive review and evaluation was undertaken to identify equivalency relationships and select several promising methods for estimating primary response load equivalencies. The second aspect was to perform field testing of instrumented pavement sections to provide data to evaluate and verify the various selected methods for load equivalencies. This has resulted in recommendations for applying pavement response based load equivalencies to estimate actual pavement loading. The scope included the following specific activities:

- Perform a comprehensive literature search and review.
- Formulate and program a framework system to calculate equivalencies using the methods identified to be the most accurate and reliable.
- Perform field testing to obtain directly measured pavement responses to known loading conditions.

- Use the measured responses to estimate relative damage using the candidate methods of load equivalencies.

- Evaluate of the effects of various vehicle parameters such as axle type, axle load, speed, and tire pressure on estimates of primary response truck LEF's.

- Evaluate the results to identify the most accurate and reliable method of determining axle group primary response equivalency factors and number of equivalent single axle loads.

SCOPE OF REPORT

This report describes the approach and work undertaken to achieve the stated objectives. An exposition of the background of load equivalency relationships including descriptions of directly applicable work which has been performed and is presented in section 2 with supporting details in appendix A. A technical description of the details of the research performed to achieve the project objectives is presented. Section 3 describes the preliminary evaluation of candidate primary response equivalency methods and the final selection for further evaluation. Section 4 presents the instrumentation set-up and section 5 describes the experiment design for collecting the data necessary to evaluate the load equivalency methods. Data analysis from its raw voltage state into finalized LEF's is covered in section 6. Section 7 contains detailed discussions and interpretations of the results of the data analysis to produce the objectives of the project. Finally, section 8 presents recommendations for additional analysis of the existing data and collection of additional data to produce implementable primary response load equivalency models for use in practice.

This report is intended to provide a complete overview of the project by concisely describing the work performed and results obtained. It provides interpretation of those results towards achieving the objectives of the project. The appendix contains a literature review of many primary response equivalency factor methods.

SECTION 2. BACKGROUND

The concept of equivalent loads was introduced by AASHO and the Bureau of Public Roads soon after the AASHO Road Test was completed in 1961. Initial implementation of the concept was through the use of pavement performance equations that had been developed and reported by road test staff. The resulting equivalent single axle load (ESAL) factors were published in the AASHTO "Interim Guide for Design of Pavement Structures."[3]

Major outputs from the AASHO Road Test were large quantities of observed data, and empirical equations for relationships between (1) pavement structure and traffic factors and (2) pavement response (e.g. deflection), pavement distress (e.g. cracking), and pavement performance (e.g. present serviceability index history). A unique feature of the road test was that traffic for each test section was constrained to a single vehicle type whose loading parameters (axle loads, axle configuration, tire pressure, transverse placement, speed, etc.) were fixed throughout the two year period of load applications. Each loading condition was repeatedly applied to several pavement types (AC on granular base, AC on stabilized base, plain PCC, and reinforced PCC) and to several layer thickness combinations within each pavement type. Moreover, all combinations of pavement type and layer thickness were treated by at least two loading conditions, and some combinations were treated by as many as six different loading conditions. Thus, for fixed loading conditions, it was possible to observe the effects of certain structural factors on pavement response, distress, and performance. For fixed structural conditions, it was possible to observe the relative effects of different loading conditions.

The derived empirical equations thus expressed pavement response, distress, and performance for a given pavement type as functions of structural and loading factors. Although most of the equations contained terms for loading factors, only one loading condition could be entered for any particular application of the equation. Thus, the equations were for "fixed-loads." No effort was made by the road test staff to relate the findings to "mixed-load" conditions, particularly since mixed-loading effects were not observed at the road test.

UTILITY AND GENERAL DEFINITION OF EQUIVALENCE FACTORS

In the practical world of pavement design and highway operations, loading conditions are in the mixed state rather than the fixed state. Loading conditions for an inservice pavement are generally different from vehicle-to-vehicle, hour-to-hour, day-to-day, and year-to-year throughout any phase of the pavement's life cycle. On the other hand, a large fraction of research-based knowledge of loading effects on pavement response/distress/ performance is for fixed-load applications. A fundamental and important question is therefore how best to translate research knowledge about fixed-load applications into a rational basis for pavement design and performance evaluation under mixed-load conditions.

The most widely-used answer to this question has been through the use of LEF's. The following is a general definition of LEF.

Suppose that two different fixed-load conditions (Lx and Ly) are applied repeatedly to separate pavements that have the same structural design and the same environment. Suppose also that when a given distress variable (D) reaches a specified value, D*, some type of pavement maintenance or rehabilitation is required. Thus D* may be called a "failure" level for the distress mode represented by D. Finally, suppose the respective pavements reach the D* condition after Nx applications of loading condition Lx and Ny applications of loading condition Ly. Then, by definition, Ny and Nx are equivalent load applications relative to D*. The ratio Ny/Nx is the load equivalence factor for Lx relative to Ly, and the ratio Nx/Ny is the load equivalence factor for Ly relative to Lx. If Ly is a "standard" loading condition (e.g., 18,000-lb (8,172-kg) single axle load), then Ny/Nx is the factor for converting Nx to an equivalent number of standard Ny load applications. From this definition it is clear that the load equivalence concept is relative to a particular mode of distress (D), a particular level of the selected distress mode (D*), and to fixed structural and environmental conditions.

Throughout this report the term primary response is used as a generic description for the complete set of specific internal responses (stresses, strains and displacements) that exist in the pavement during an individual load application. It is assumed that the major determinants of the response include physical properties of the pavement structure and its roadbed as well as the loading factors. Thus two pavements with different physical properties will generally be in different response states for a fixed set of loading conditions. Alternatively, two different loading conditions might produce the same response state in two pavements whose physical properties differ. It is therefore important to distinguish between fixed-response applications and fixed-load applications, and between response equivalence factors (REF's) and load equivalence factors. It is for this reason that determinants are at fixed levels.

Another approach to the mixed-load question is through the use of Miner's criterion for damage ratios associated with individual fixed-load conditions.[4] It must be understood however, that the damage-ratio approach is a special case of the load equivalence and response equivalence approach. In this project we are examining response equivalence approaches that are assumed to relate to pavement damage. So, in reality we are investigating these slightly different approaches as a single concept and generically calling it load equivalency for consistency with other research in this area.

Virtually all reported load equivalence relationships and factors have been <u>derived</u> from pavement response/distress/performance relationships that represent fixed response/load conditions. From the general definition for LEF it can be seen that the required relationships are those which predict the number of applications, under a given loading condition, at which a particular distress/performance variable will reach a specified failure or terminal level. The ratio of the prediction for the standard loading condition to the prediction for loading condition X is then the load equivalency factor for loading condition X. It is quite clear that derived LEF's are not only dependent upon the distress variable and its failure level, but also upon the relationships (equations) that have been used for the derivations.

SUMMARY BACKGROUND

The concept of primary response truck load equivalencies can be quite complex. There are many factors and considerations that influence such factors. A number of studies have been undertaken to examine and estimate primary response truck LEF's by various methods. Appendix A presents a summarization of the methods which were uncovered in a thorough literature review on this project. Each of these methods handled the above considerations in various ways. Appendix B provides an in-depth, theoretical discussion of the concepts of load and response equivalency which were briefly touched on in this section.

The objective of this project was to examine each of the available load equivalency methods as presented in appendix A and select several for detailed analysis. The selected methods would then be analyzed and compared to determine the viability and usefulness of primary response LEF's for pavement evaluation, design, and research. It was also to recommend a promising method of primary response equivalencies for use in practical applications. The research is also aimed at determining which vehicle and pavement parameters influence the results of these primary response truck LEF's and the quantity of that influence. The following section describes how a large array of primary response equivalency methods presented in appendix A were evaluated and screened to select several methods for further study.

SECTION 3. CANDIDATE PRIMARY RESPONSE METHODS

The most relevant structural pavement response-based equivalency factor methods currently available and reviewed in appendix A were screened subjectively as shown in table 1. All methods with a rating of 5 or more were selected for an objective screening evaluation. These methods were:

- Jung et al.[5]
- Battiato et al.[6]
- Southgate et al.[7, 8]
- Hudson et al.[9]
- Christison et al.[2]
- Hutchinson et al.[10]

The evaluation matrix for the objective criteria consisted of two pavement structures, weak and strong, as shown in figures 1 and 2. Three single axle loads used of 12-kip (5,448-kg), 18-kip (8,172-kg), and 24-kip (10,896-kg) and three tandem axle loads of 24-kip (10,896-kg) 32-kip (14,528-kg), and 40-kip (18,160-kg) were used in the analysis.

The pavement responses (e.g., deflection, strains, stresses) were obtained using the ELSYM5 elastic layer theory computer model. The location of the loads was modeled as shown in figures 3 and 4. The tire pressure was assumed to be 90-psi (620-kPa) in all cases except for the standard 18,000-lb (8,172-kg) single-axle dual-tire in which case 80-psi (550-kPa) was used.

The LEF's found using the selected methods are shown in table 2. The results are graphed in figures 5 and 6 as bar charts, and in figures 7, 8, 9, and 10 as line graphs including AASHTO values.

For single-axle loads of 12-kip (5,448-kg) and 18-kip (8,172-kg), all methods predicted less relative damage on the weak pavement structure than on the strong pavement structure. The single-axle load of 24-kip (10,896-kg) causes in all cases less relative damage on the strong pavement structure than on the weak structure. This general behavior is less clearly defined in Jung's method.

Similar trends, as defined above, have also been shown for tandem axle loads. In all cases, tandem axle loads of 24-kip (10,896-kg) and 32-kip (14,528-kg) tend to produce less relative damage on the weak pavement structure than on the strong pavement structure. Tandem axle loads of 40-kip (18,160-kg) tend to do the same except in the case of the Battiato, Southgate, and Christison strain methods.

Hutchinson's method gave the same results as Christison's method based on deflections because the computer program is unable to model the deflections as measured in the field (i.e., a deflection profile with two humps of different magnitudes).

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Table 1. Subjective evaluation of LEF methods.

RATING CRITERIA	0 = EITHER 1 = BOTH		-1 = NO 1 = YES	0 = NO 1 = YES	0 = EITHER 1 = BOTH		0 = NO 1 = YES	-1 = NO +1 = YES	0 = NO 1 = YES	-1 = YES +1 = NO	-1 = NO +1 = YES	FINAL RATING
SCREENING CRITERIA / METHOD	Is it a theoretical method ?	Is the method based on testing results ?	Are there explicit LEF equations in terms of pavement responses ?	Are there load equivalency factors for tridems ?	Is the method applicable to flexible pavements ?	Is the method applicable to rigid pavements ?	Are variables or parameters relatively easy to calculate or measure ?	Is the method rigorous (i.e Does it consider the effect of subgrade types, temperature, etc.) ?	Do results have widespread applicability ?	Has the method been further developed since first published ?	Is there enough information to implement the method ?	7-9 = To be Used 4-6 = Possible 0-3 = Not to be used
ZUBE et al (Ref. 65.1)	No	Yes 0	No -1	No 0	Yes	No 0	Yes 1	Yes 1	No 0	Yes -1	Yes 1	1
DEACON (Ref. 69.3)	Yes	No 0	Yes 1	No 0	Yes	No 0	Yes 1	Yes 1	No 0	Yes -1	Yes 1	3
SCALA (Ref. 70.2)	Yes	No 0	No -1	Yes 1	Yes	No 0	Yes 1	No -1	No 0	Yes -1	Yes 1	0
GERARD et al (Ref. 70.3)	Yes	No 0	No -1	No 0	Yes	No 0	Yes 1	Yes 1	No 0	No 1	Yes/No 0	2
RAMSAMOOJ et al (Ref. 72.6)	Yes	Yes 1	Yes 1	No 0	Yes	No 0	No 0	No -1	Yes 1	No 1	Yes/No 0	3
JUNG et al (Ref. 74.5)	Yes	No 0	Yes 1	No 0	Yes	No 0	Yes 1	Yes 1	Yes 1	No 1	Yes 1	6
TERREL et al (Ref. 76.1)	Yes	No 0	No -1	No 0	Yes	No 0	Yes 1	Yes 1	Yes 1	No 1	No -1	2
TREYBIG et a (Ref. 76.3)	Yes	No 0	No -1	Yes 1	Yes	No 0	Yes 1	Yes/No 0	No 0	Yes -1	Yes 1	1
NORDIC COOPERATIVE RESEARCH GRP (Ref. 77.4)	Yes	No 0	No -1	No 0	Yes	No 0	Yes 1	Yes 1	No 0	Yes -1	Yes/No 0	0
KIRWAN et al (Ref. 77.5)	Yes	No 0	No -1	No 0	Yes	No 0	No 0	No -1	No 0	No 1	Yes 1	0
VON QUINTUS (Ref. 78.2)	Yes	No 0	Yes 1	Yes 1	Yes	No 0	Yes 1	Yes/No 0	No 0	Yes -1	Yes 1	3
CHRISTISON (Ref. 78.3)	No	Yes 0	Yes 1	No 0	Yes	No 0	Yes 1	No -1	Yes 1	Yes -1	Yes 1	2
CHRISTISON et al (Ref. 80.5)	No	Yes 0	Yes 1	No 0	Yes	No 0	Yes 1	No -1	Yes 1	Yes -1	Yes 1	2
WANG et al (Ref. 81.8)	Yes	No 0	No -1	Yes 1	Yes	No 0	Yes 1	Yes/No 0	Yes 1	No 1	Yes/No 0	3
TAYABJI et al (Ref. 83.9)	Yes	Yes 1	No -1	Yes 1	No	Yes 0	Yes 1	Yes 1	No 0	No 1	Yes/No 0	4
SOUTHGATE et al (Ref. 84.8)	Yes	No 0	Yes 1	Yes 1	Yes	No 0	Yes 1	Yes 1	Yes 1	Yes -1	Yes 1	5
GORGE (Ref. 84.9)	Yes	Yes 1	No -1	No 0	Yes	Yes 1	No 0	Yes 1	Yes 1	No 1	Yes/No 0	4
BATTIATO (Ref. 84.10)	No	Yes 0	Yes 1	Yes 1	Yes	No 0	Yes 1	Yes 1	No 0	No 1	Yes 1	6
SOUTHGATE et al (Ref. 85.5)	Yes	No 0	Yes 1	Yes 1	Yes	No 0	Yes 1	Yes 1	Yes 1	No 1	Yes 1	7
YAO (Ref. 85.16)	Yes	Yes 1	No -1	No 0	No	Yes 0	Yes 1	Yes 1	No 0	No 1	Yes/No 0	3
SHARP et al (Ref. 86.8)	No	Yes 0	No -1	Yes 1	Yes	No 0	Yes 1	No -1	No 0	No 1	No -1	0
HUDSON et al (Ref. 86.9)	Yes	No 0	Yes 1	Yes 1	Yes	No 0	Yes 1	Yes 1	Yes 1	No 1	Yes 1	7
CHRISTISON et al (Ref. 86.19)	No	Yes 0	Yes 1	Yes 1	Yes	No 0	Yes 1	Yes 1	Yes 1	No 1	Yes 1	7
HUTCHINSON et al (Ref. 87.10)	No	Yes 0	Yes 1	Yes 1	Yes	No 0	Yes 1	Yes 1	Yes 1	No 1	Yes 1	7
MAJIDZADEH et al (Ref. 88.10)	Yes	No 0	Yes 1	No 0	Yes	No 0	Yes 1	Yes 1	No 0	No 1	No -1	3

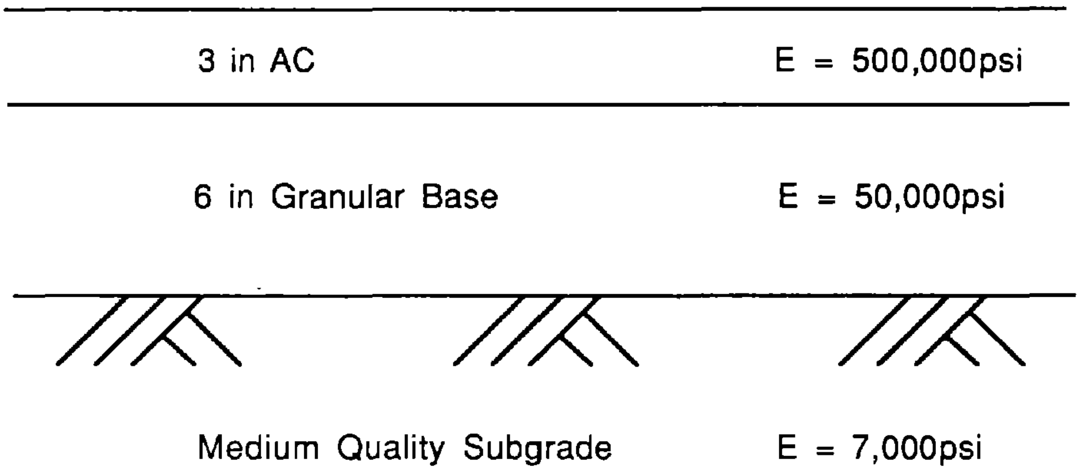


Figure 1. Weak pavement structure.

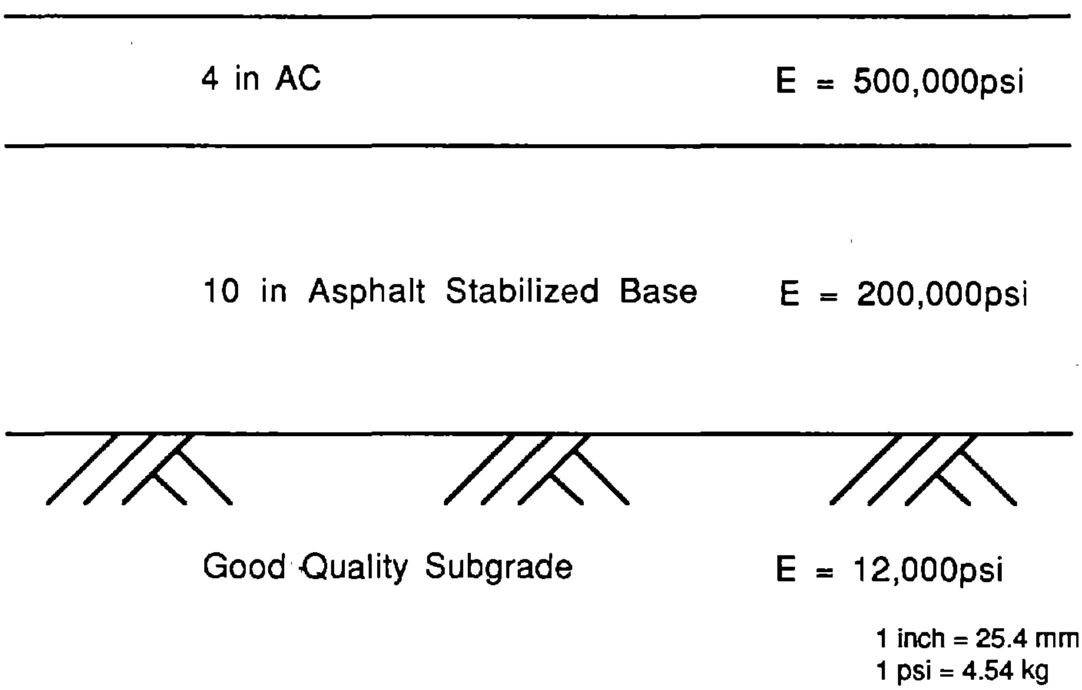


Figure 2. Strong pavement structure.

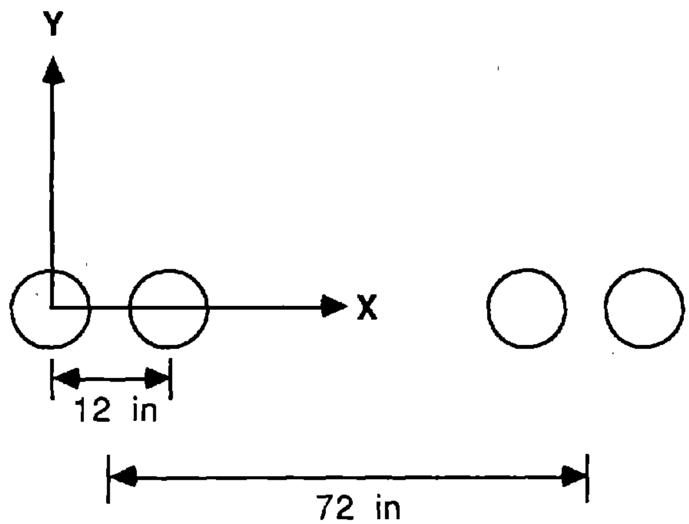


Figure 3. Location of single axle loads.

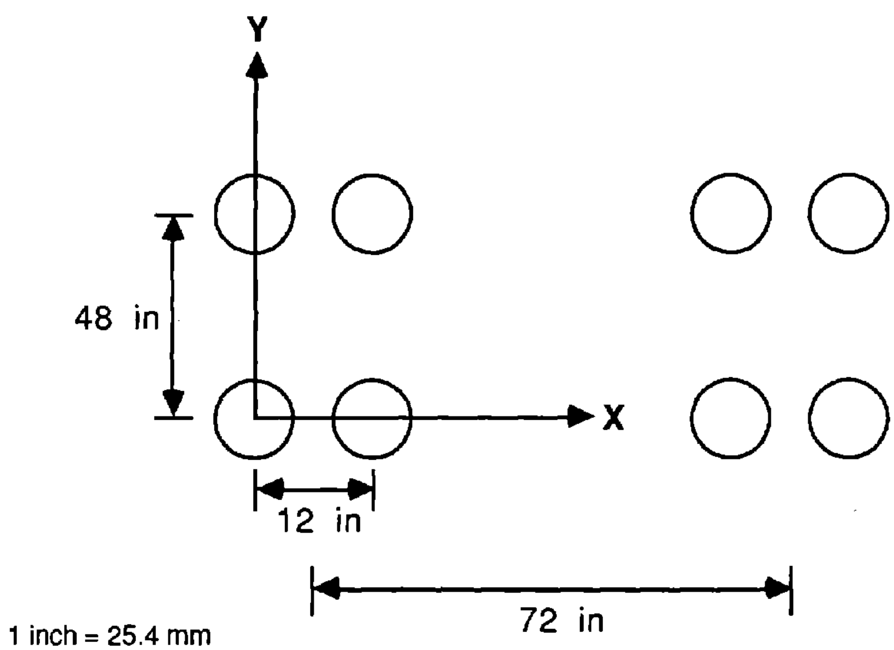


Figure 4. Location of tandem axle loads.

Table 2. Load equivalency factor results.

LEF METHOD		Single Axle 12k-Load	Single Axle 18k-Load	Single Axle 24k-Load	Tandem Axle 24k-Load	Tandem Axle 32k-Load	Tandem Axle 40k-Load
Jung Strain	Weak Pavement	0.169	1.001	2.971	0.269	0.652	1.072
Jung Strain	Strong Pavement	0.185	1.022	2.931	0.632	1.475	2.380
Battiato Strain	Weak Pavement	0.526	1.296	2.360	0.979	1.848	2.931
Battiato Strain	Strong Pavement	0.861	1.492	1.833	1.522	2.251	2.732
Southgate Strain	Weak Pavement	0.469	1.560	3.305	0.450	1.061	1.907
Southgate Strain	Strong Pavement	0.701	1.761	2.687	0.626	1.225	1.794
Hudson Tensile Strain	Weak Pavement	0.324	1.576	4.514	2.604	6.526	12.711
Hudson Tensile Strain	Strong Pavement	0.771	2.020	2.900	34.74	61.19	80.95
Hudson Shear Strain	Weak Pavement	0.381	1.634	4.431	21.71	47.49	83.27
Hudson Shear Strain	Strong Pavement	0.592	1.927	3.357	68.35	127.01	184.34
Hudson Shear Stress	Weak Pavement	0.346	1.624	4.326	23.45	49.46	84.55
Hudson Shear Stress	Strong Pavement	0.597	1.906	3.302	78.22	140.62	200.38
Christison Deflection	Weak Pavement	0.230	1.012	2.963	0.632	1.825	4.153
Christison Deflection	Strong Pavement	0.254	1.036	2.855	1.036	2.887	6.437
Christison Strain	Weak Pavement	0.565	1.259	2.146	1.060	1.865	2.810
Christison Strain	Strong Pavement	0.876	1.427	1.714	1.569	2.222	2.639
Hutchinson Deflection	Weak Pavement	0.230	1.012	2.963	0.632	1.825	4.153
Hutchinson Deflection	Strong Pavement	0.254	1.036	2.855	1.036	2.887	6.437

1 kip=454 kg

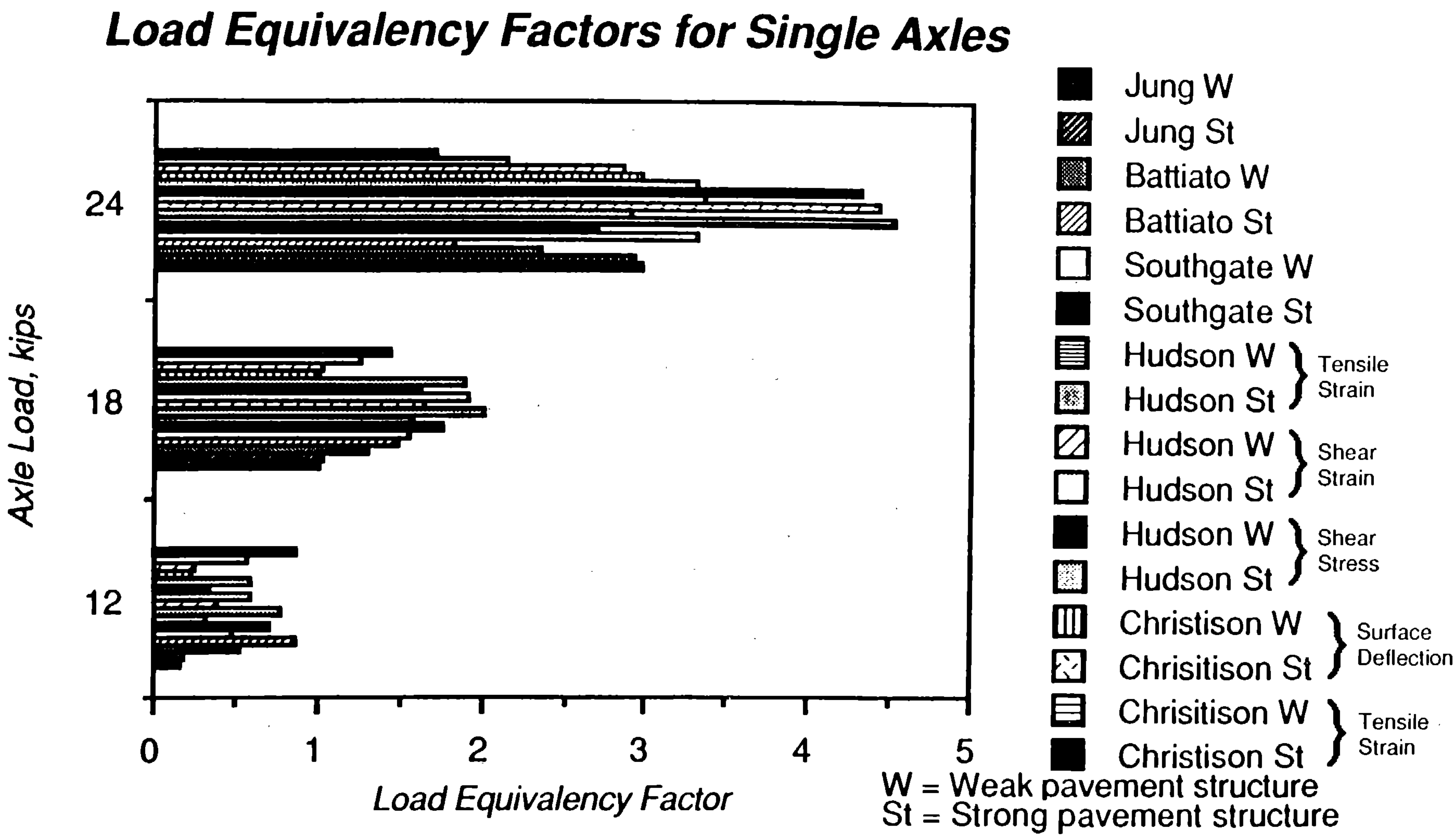


Figure 5. Load equivalency factors for single axles.

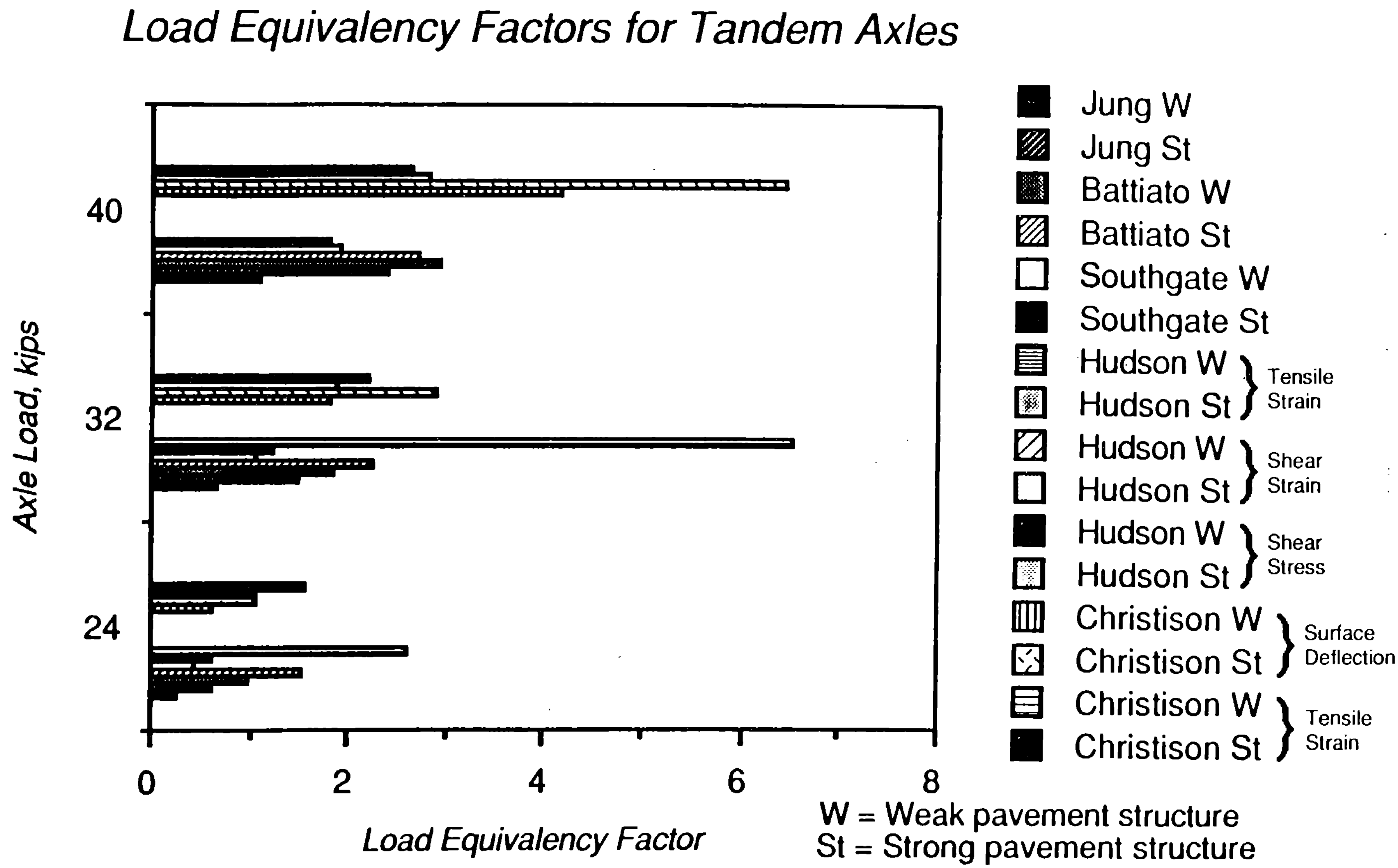


Figure 6. Load equivalency factors for tandem axles.

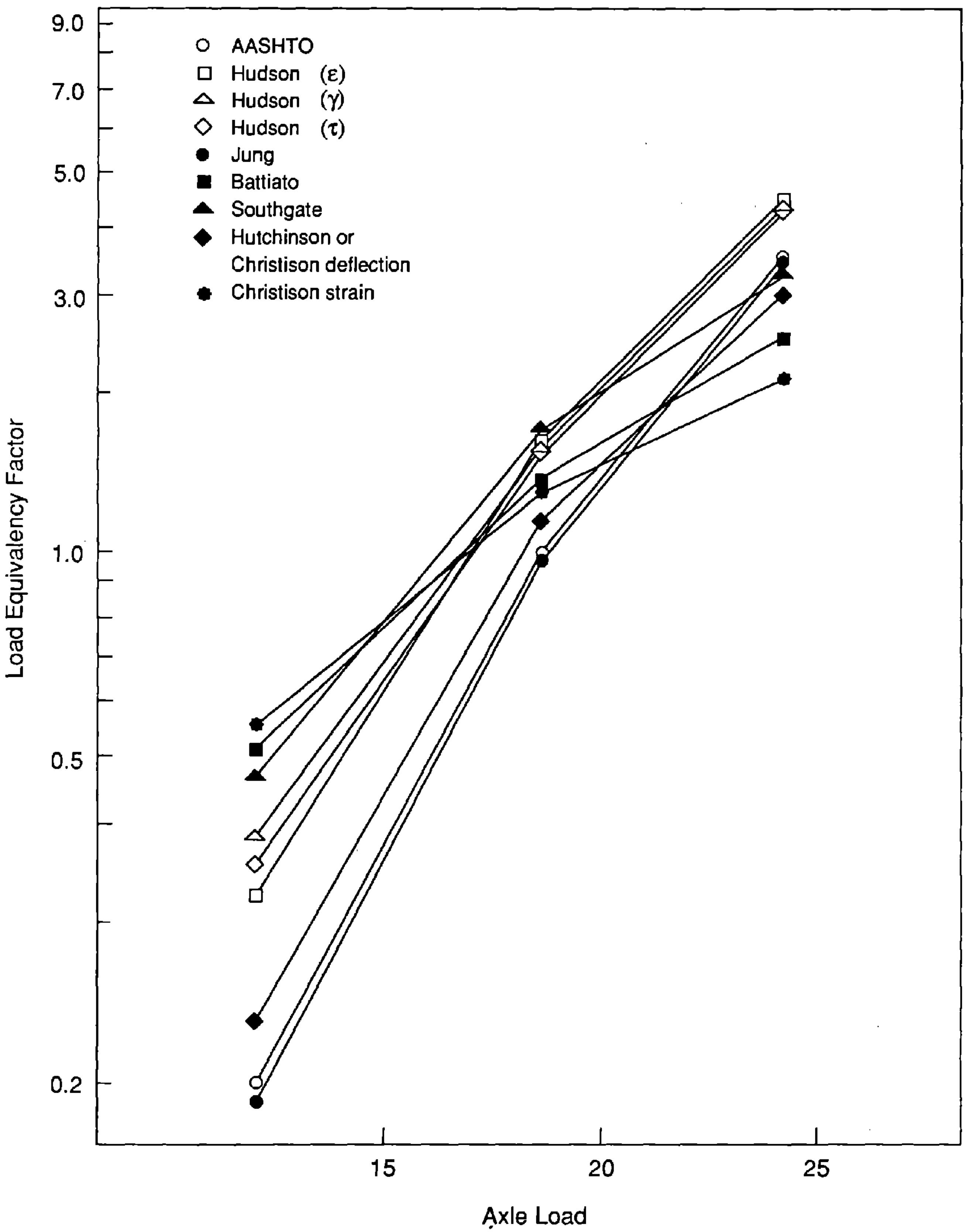


Figure 7. Comparison of LEF's for single axle on weak pavement.

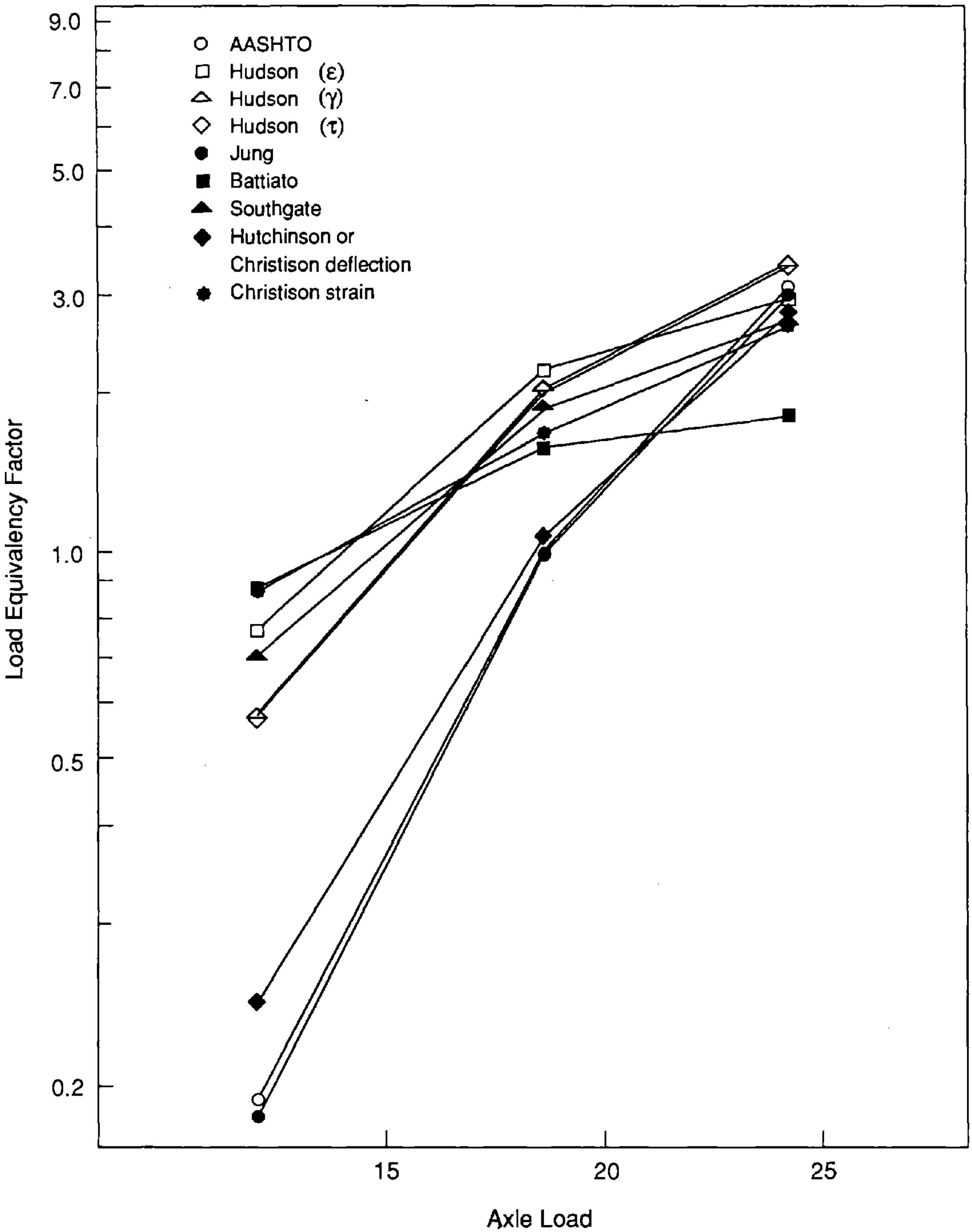


Figure 8. Comparison of LEF's for single axle on strong pavement.

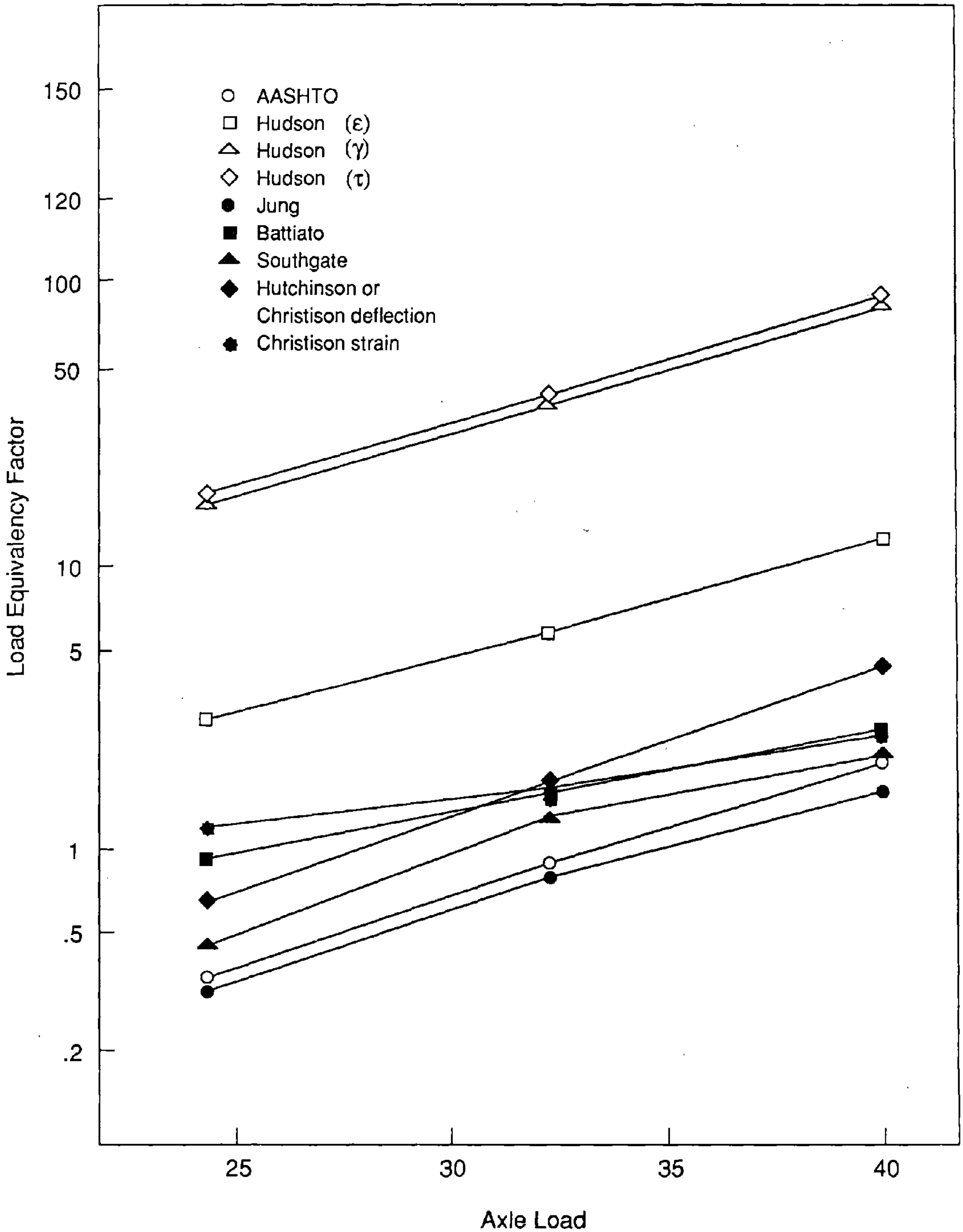


Figure 9. Comparison of LEF's for tandem axle on the weak pavement.

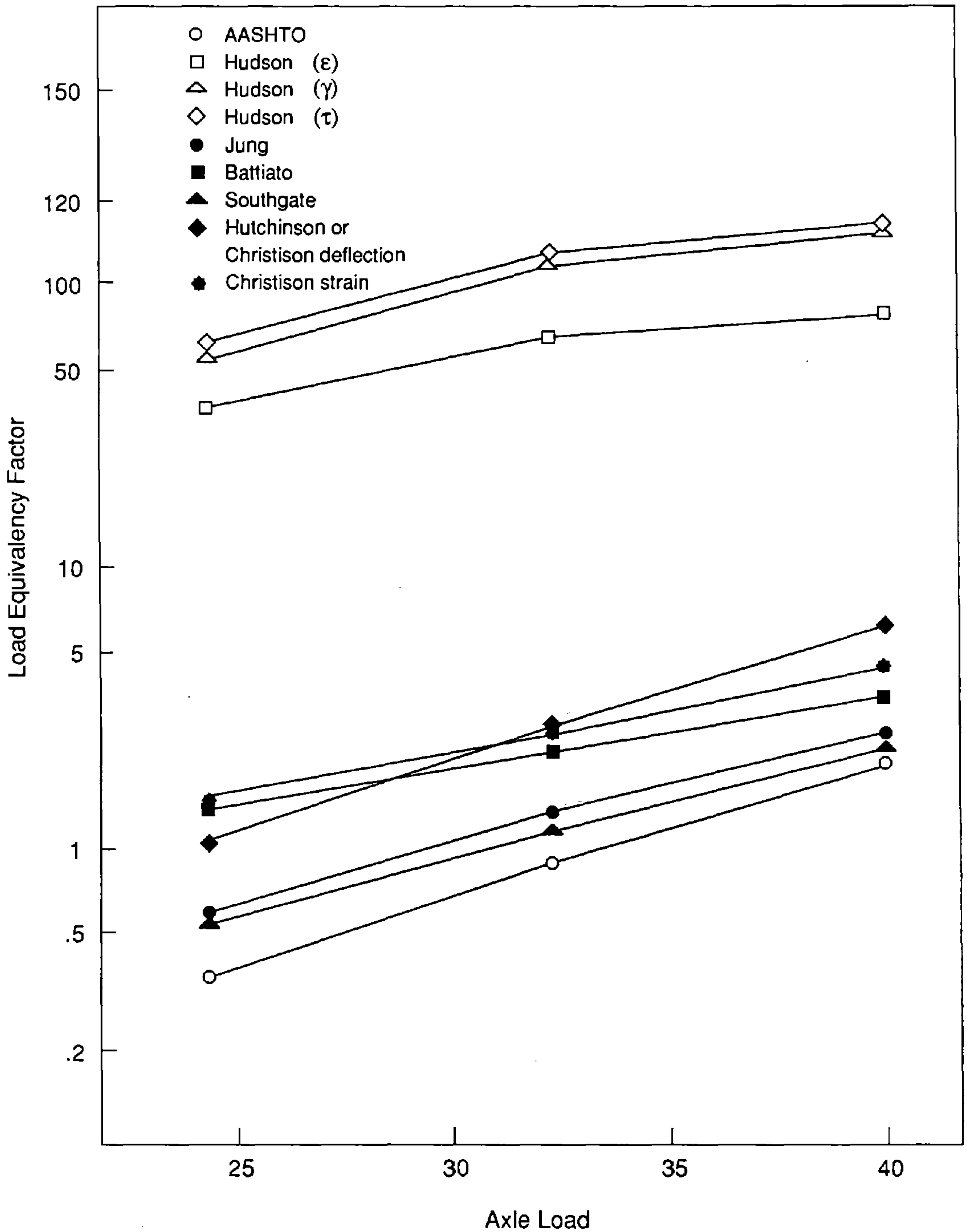


Figure 10. Comparison of LEF's for tandem axle on the strong pavement.

Two of the methods proposed by Hudson use pavement response measures that are not directly measurable with the instrumentation on this project. Also, their values of LEF's were relatively high compared to the AASHTO equivalency values. Hudson's strain based method produced much more reasonable values; however, these were somewhat higher than the AASHTO values and the values from Christison's strain and deflection method and Hutchinson's deflection method. Jung's method also uses a pavement response measurement that was not directly measurable from the instrumentation installed on this project. The Southgate strain method produced results that are relatively comparable to the AASHTO factors. The Battiato method produced somewhat higher equivalency factors in general and also could not be directly used with the pavement response measurements obtained on this project.

Based on the examination of these results, the following methods were recommended for further study:

- Christison's methods (strain and deflection).
- Southgate's method (strain only).
- Hutchinson's method (deflection only).

These four methods are used in evaluating primary response LEF's for the investigative portion of the study. Field measurements of strain and deflection will be used in each of the two strain methods and each of the two deflection methods mentioned above. Resulting equivalency factors will be analyzed over a large experimental factorial to determine the effects of axle type, axle load, tire pressure, speed, pavement type, and instrumentation variation on the estimation of primary response LEF's. We will also provide a more detailed analysis of the differences between these final four LEF methods.

The next section describes the data collection procedure and set-up plan for collecting the strain and deflection data necessary for further investigation of these primary response LEF methods.

SECTION 4. PAVEMENT RESPONSE MEASUREMENTS

A test facility was constructed and instrumented at the FHWA Turner-Fairbank Highway Research Center (TFHRC) in McLean, Virginia. Descriptions of the vehicles used for load testing and the instrumentation used to collect the primary pavement responses for the basic load equivalency factor experiments are presented.

INSTRUMENTED TEST SECTIONS

Test Section Layout

The first consideration in the design of the test sections is the overall layout. The accommodation of the test vehicle is important to adequately provide for efficient movement. The approximate length of each pavement section is 100-ft (30.5-m). Lengths have been selected to allow the test vehicle to fully load each pavement type independently, yet minimize construction costs. The sections were built on the Route 193 access road to the TFHRC as shown in figure 11.

Each access road test section is 12-ft (3.7-m) wide and 100-ft (30.5-m) long. Appropriate transition sections were provided at the ends of each section to insure the design thickness, compaction, and smoothness of each section. The existing access road pavement was removed by saw cutting the asphalt concrete and removing asphalt concrete, aggregate, aggregate base and soil to a depth of two feet (0.6-m) below the design subgrade elevation for each test section.

Reconstruction of the access road was performed in continuous operations for the entire 225-ft (68.6-m)± length. Each element of construction (i.e., pavement removal; placing, compacting and grading the soil subgrade; placing, compacting and grading of the crushed aggregate base and asphalt concrete paving) was completed prior to starting the next element. Approximately one week access to the finished crushed aggregate base prior to the asphalt concrete paving was required for gauge and cable installation.

Pavement Structure Characteristics

Two cross-sections characterized as weak and strong were constructed as shown generally in figures 12 and 13. The weak pavement consists of 3½-in (88-mm) of hot-mix asphalt concrete over a 12-in (403-mm) crushed aggregate base on a select subgrade soil. The strong pavement has 7-in (177-mm) of hot-mix asphalt concrete over the same 12-in (403-mm) crushed aggregate base and select subgrade soil. Detailed cross-sections of the two pavement structures are shown in figures 14 and 15.

PAVEMENT INSTRUMENTATION DESCRIPTION AND LAYOUT

Figure 16 illustrates the instrumentation layout for the equivalency factor experiments. Dimensions and details of one set of deflection and strain gauges is shown in figure 17. Strain measurements are accomplished by gauges encapsulated in asphalt plate strain carriers developed by the Alberta Research Council. The gauges are placed at the asphalt

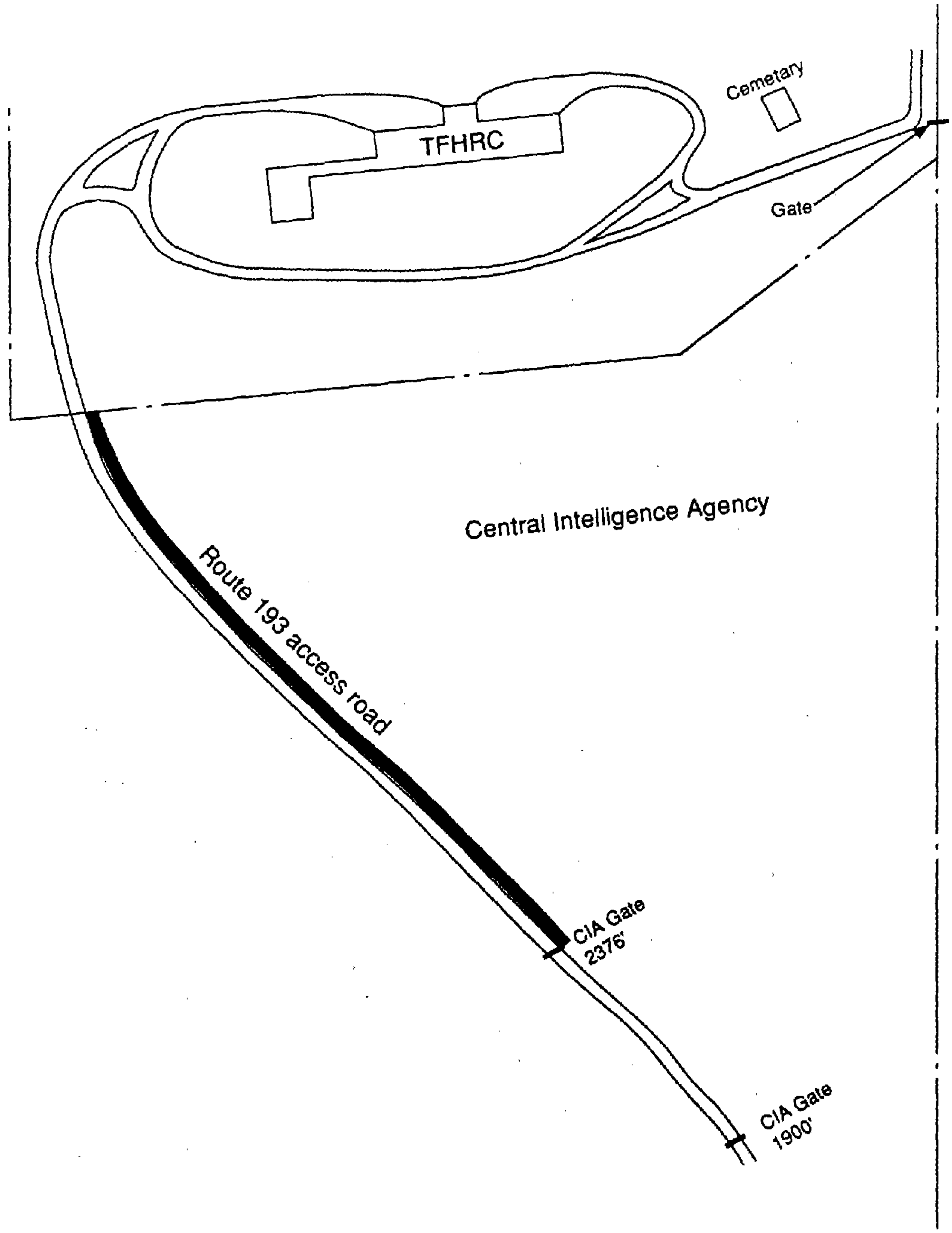


Figure 11. General layout of test sections.

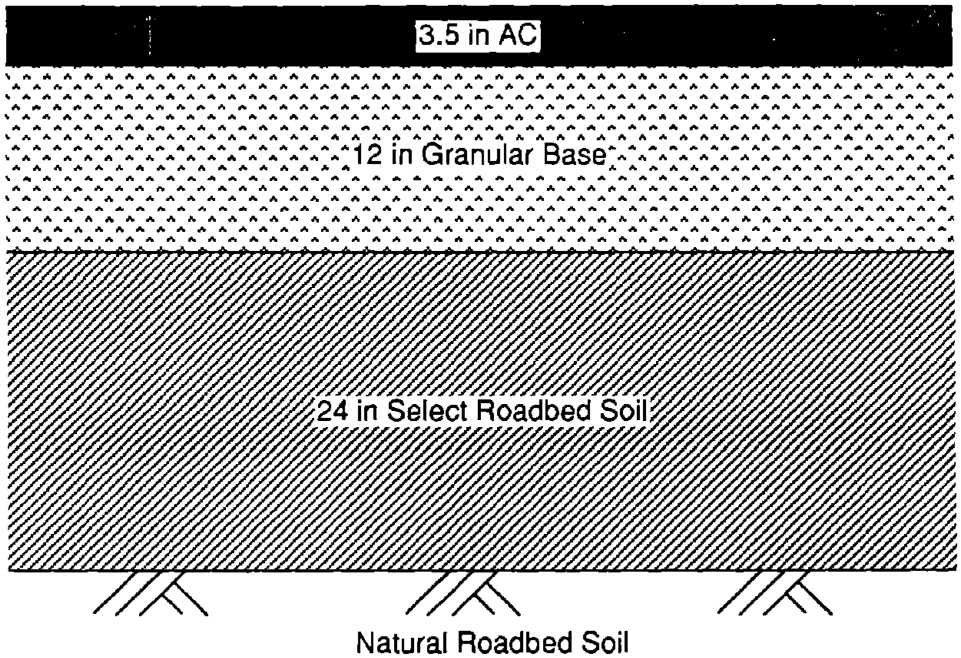


Figure 12. Planned cross section of the weak pavement structure.

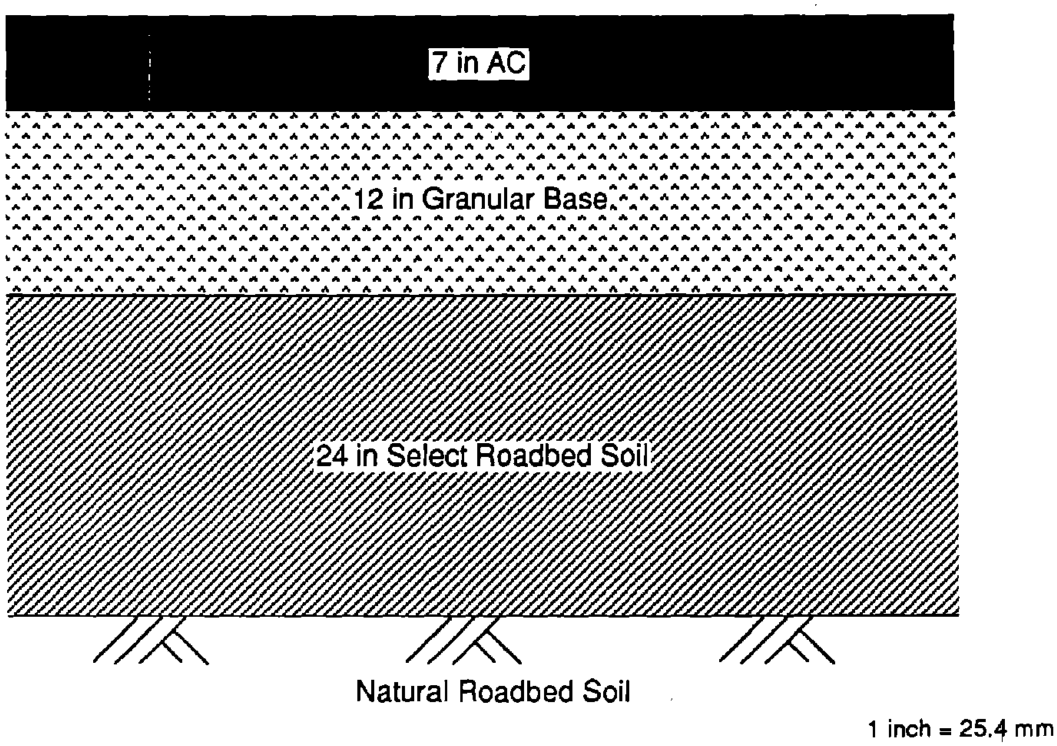


Figure 13. Planned cross section of the strong pavement structure.

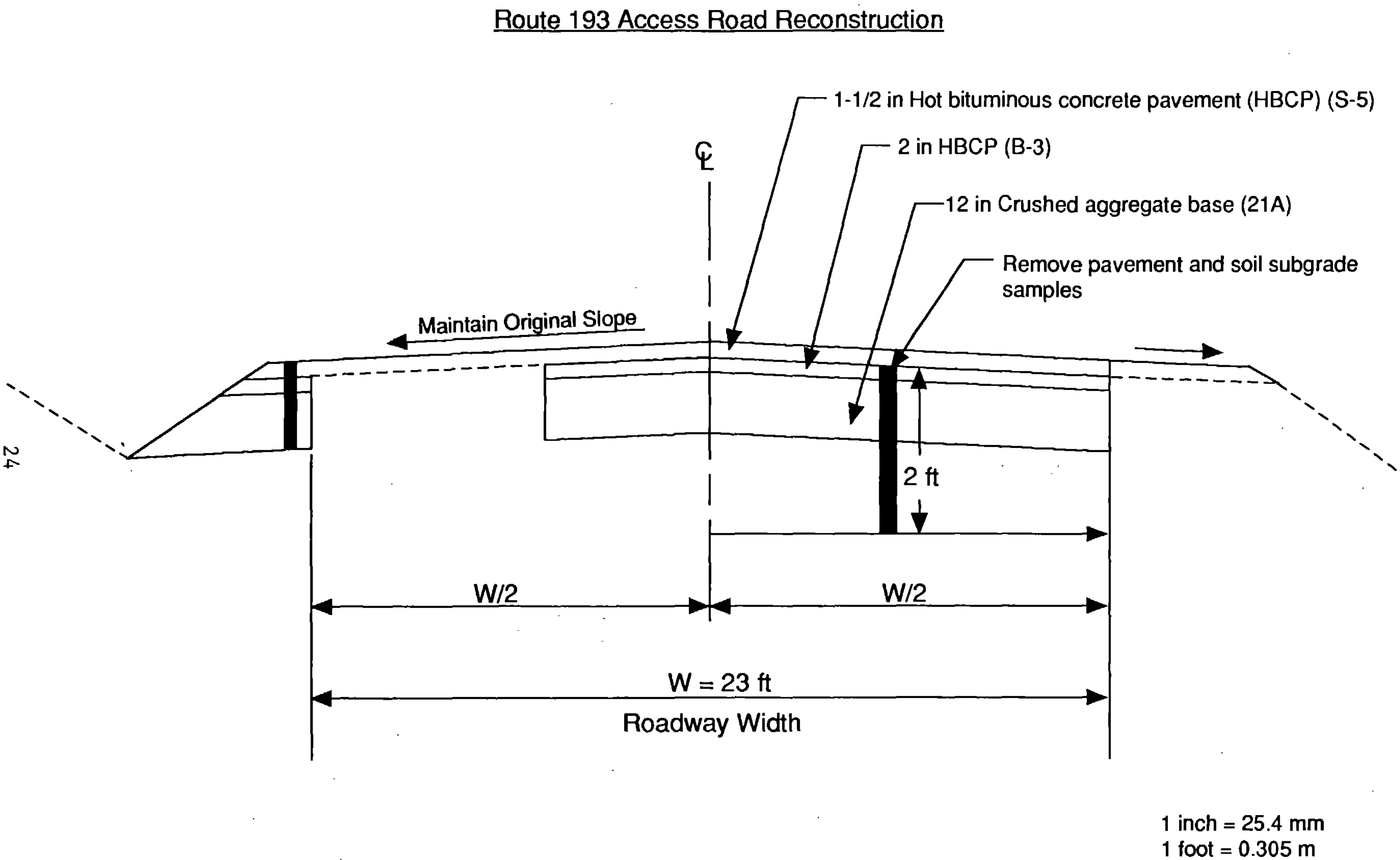


Figure 14. Cross section of thin (3 1/2-in) pavement structure.

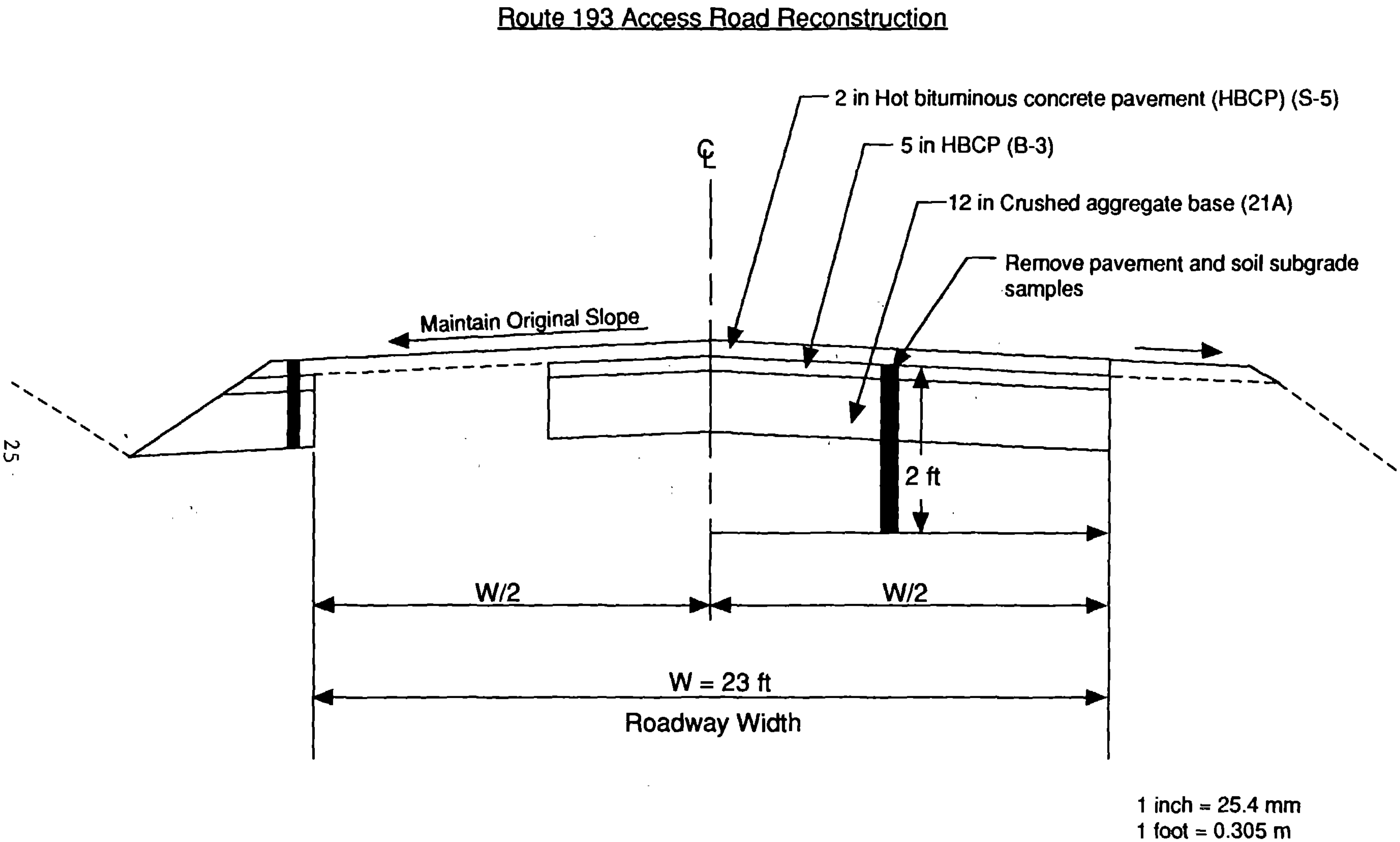


Figure 15. Cross section of thick (7-in) pavement structure.

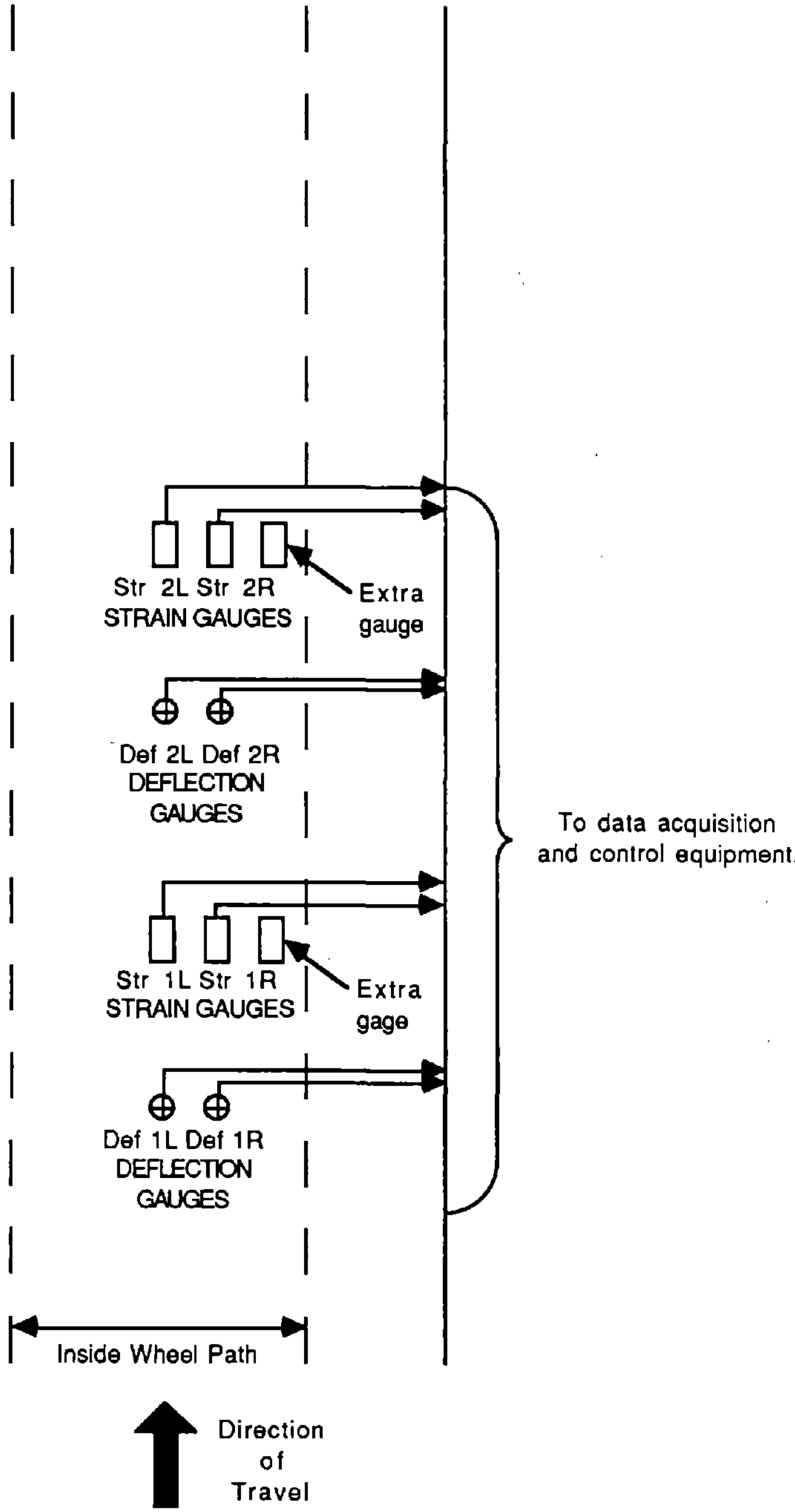
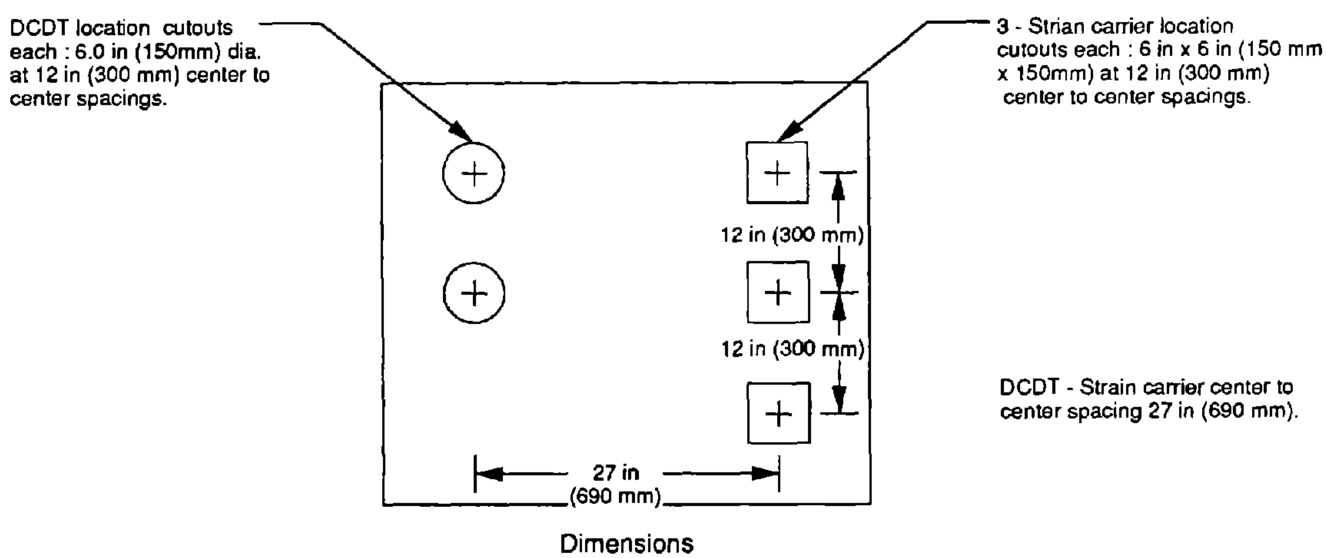


Figure 16. Instrumentation layout and numbering for one pavement section.



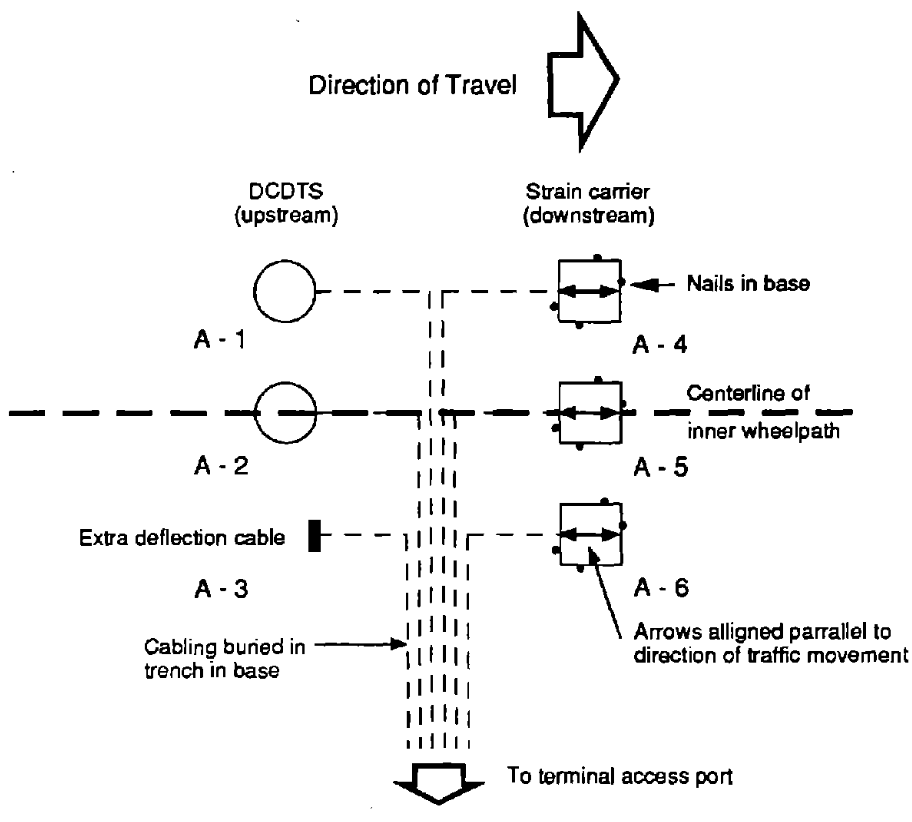


Figure 17. Transducer layout dimensions and details.

concrete-base layer interface to measure longitudinal interfacial tensile strains as shown in figure 18. Deflection will be measured using linear variable differential transformers (LVDT's) mounted in subsurface referencing assemblies, placed transversely across the outer wheelpath as shown in figure 19. Thermocouples will be placed near the outer edge of the test section for pavement temperature measurements. Underground cables from the strain and deflection transducers in the pavement are routed to a junction box located centrally adjacent to the sections. Figure 20 illustrates the correspondence between the transducers and the connections inside the junction box. Following are technical descriptions of some of the specific instruments installed.

Lateral Position Measurement

Method Used: Ultrasonic

Transducer Description / Specifications

Manufacturer:	Amerace Corporation
Supplier:	Newark Electronics
Model:	AGASTAT PCUA30M30AV
Dimensions:	1.17-in (30-mm) dia, 2.73-in (70-mm) long
Input Voltage:	10 to 30 VDC
Output:	0 to 5 VDC, proportional to location of detected target
Beam Width:	Approx. 10 degrees
Min. Distance:	4-in (101-mm)
Max. Distance:	30-in (762-mm)
Response Speed:	Approx. 50 milliseconds
Operating Temp.:	-20 to +50°C
Humidity:	0 to 95 percent, noncondensing
Approx. Cost:	$240

The intended application for this device is to accurately locate the test vehicle footprint as it passes over other transducers such as the deflection and strain gauges. The sensor provides a 0 to 5 VDC analog signal which is proportional to the position of a sensed target. This simplifies the data acquisition task since the device can be connected directly to the acquisition system without any additional signal conditioning or interfacing. The standard sensing window covers a range of 4- to 30- in (101- to 762-mm) but is easily adjusted to as little as 10 percent of this range if desired. This device is protected from damage by a stray test vehicle by a steel housing.

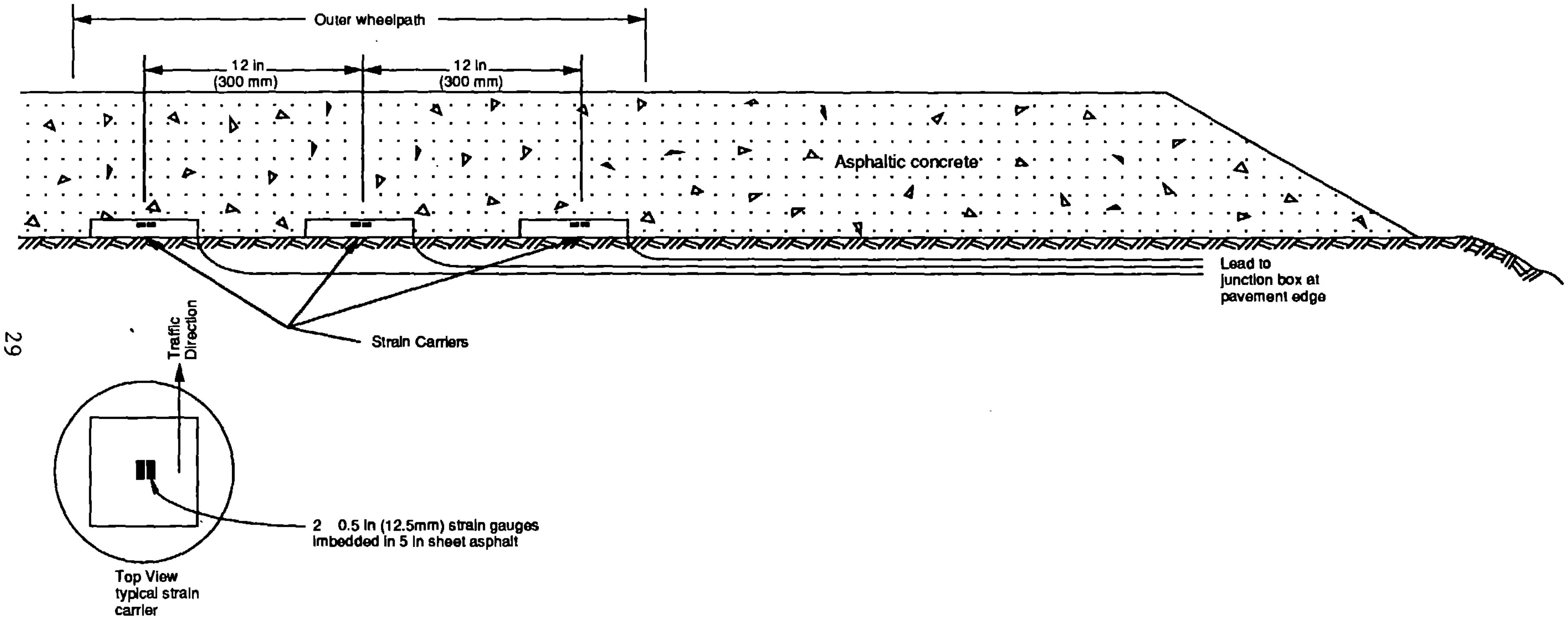


Figure 18. Strain transducer installation.

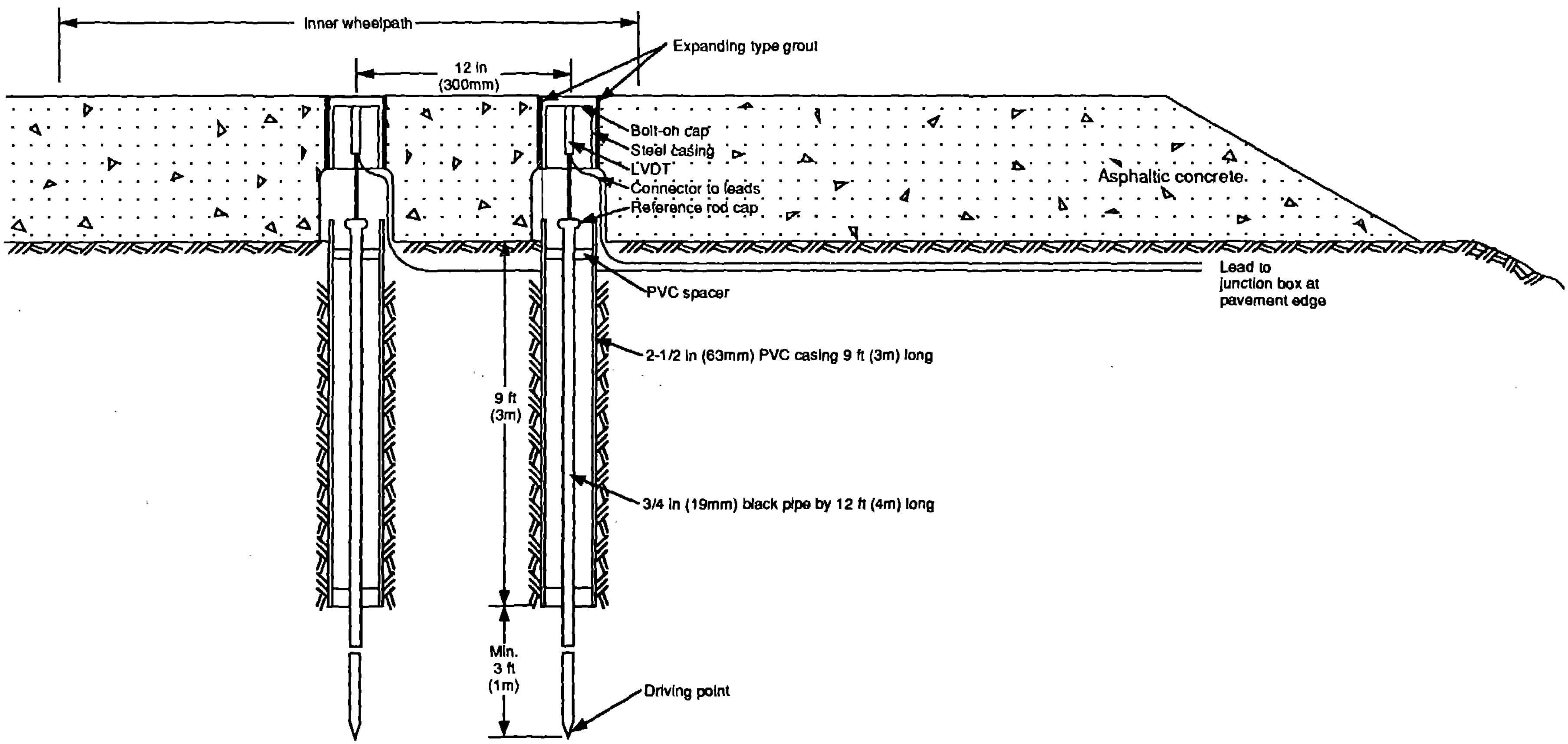
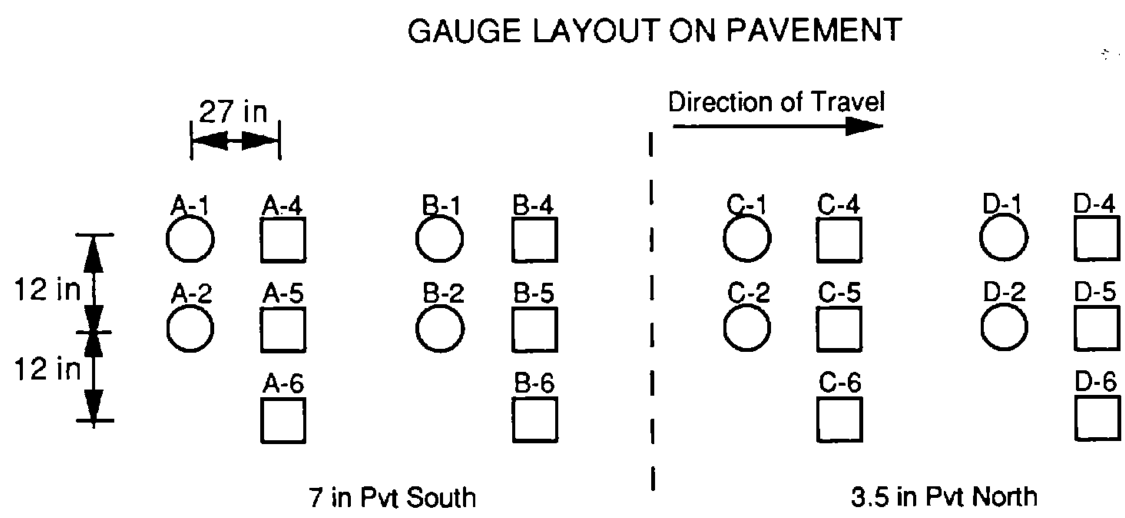
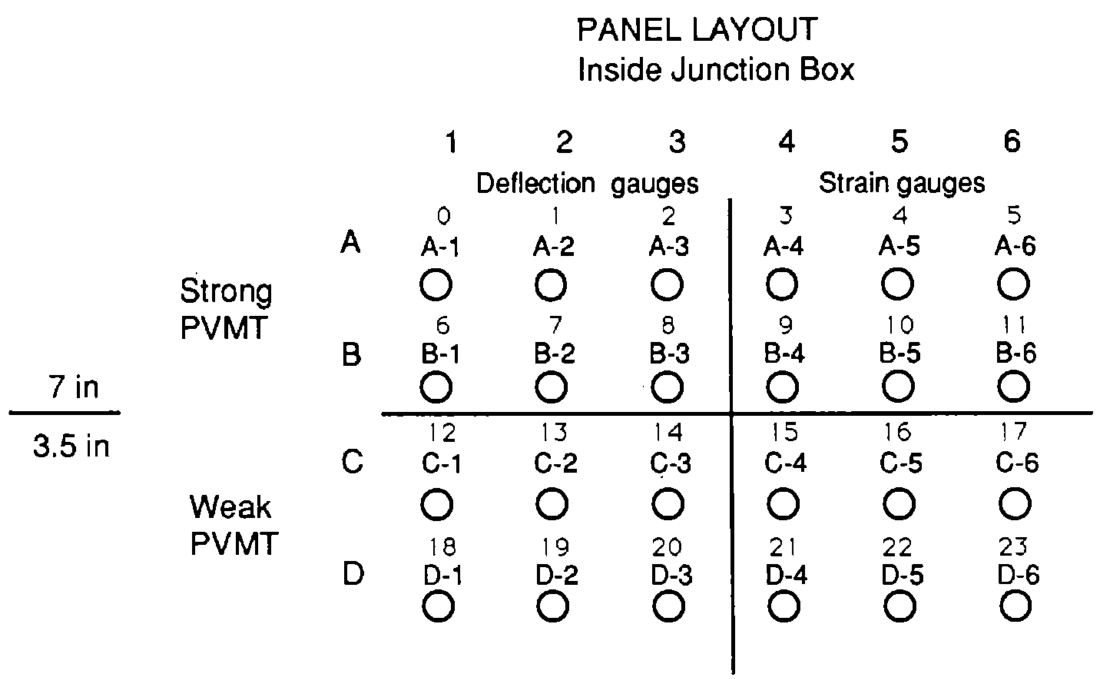


Figure 19. Deflection transducer installation.





NOTE: Deflection gauges A-3, B-3, C-3, D-3 were never installed although cables are in the pavement.

1 inch = 25.4 mm

Figure 20. Cable organization and junction box layout.

Deflection Measurement

Method Proposed: DCDT (DC-DC LVDT)

Transducer Description / Specifications

Manufacturer:	Trans-Tek Inc.
Model:	0201-000
Range:	+/- 0.100-in (2.54-mm)
Input Voltage:	5 to 7 VDC
Output Voltage:	+/- 2.8 VDC, full scale, open circuit
Scale Factor:	28 volts/in
Linearity:	+/- 0.5 percent Full Scale
Temp. Range:	-65 to 140°F
Approx. Cost:	$200 ea

This device is specified by the Alberta Research Council for use in their single layer deflectometers. DC-DC LVDT's differ from conventional LVDT's in that in addition to a precision LVDT they also contain a solid state oscillator and phase sensitive demodulator within a single compact package. This allows for simple DC in, DC out operation without the need for additional signal conditioners.

Single Layer Deflectometer

Manufacturer:	Alberta Research Council
Transducer:	Trans-Tek DC-DC LVDT (0201-0000)
Approx. Cost:	$1500 (less DC-DC LVDT)

The type of Single Layer Deflectometer (SLD) used for deflection measurement offers several advantages, including relatively simple installation, easy DC adjustment/replacement, and modest cost. This SLD like most other designs uses a central rod anchored at a point beneath the influence of surface loading for a fixed reference point.

Installation of a SLD occurs in two stages. After the basecourse has been completed, and prior to paving, the SLD locations are tied in by accurate survey, and the cables are installed. To protect the cables from the paving process, they are buried in shallow trenches in the top of the basecourse and then carefully covered. At each SLD location, the cable end is placed in a 4- to 6-in (101- to 152-mm) depression in the basecourse and then backfilled with fine sand.

The second stage of the installation process occurs after the asphalt concrete (AC) is in place. Each SLD site is located using the survey information from phase one. A 6-in (152-mm) core is cut through the AC at each SLD site, the sand carefully removed and the cable end recovered. A 4-in (101-mm) hole is augered to the anchor point 8- to 10-ft (2.4- to 3.1-m) and is lined with PVC pipe. The reference rods are then driven to refusal using spacers as necessary to center them in the bore. The DC carrier is then installed in the 6-in (152-mm) AC bore using an expanding type grout. Care must be taken throughout this process to protect the cables from damage. A diagram of the DCDT carrier showing the details of the installation is presented in figure 21.

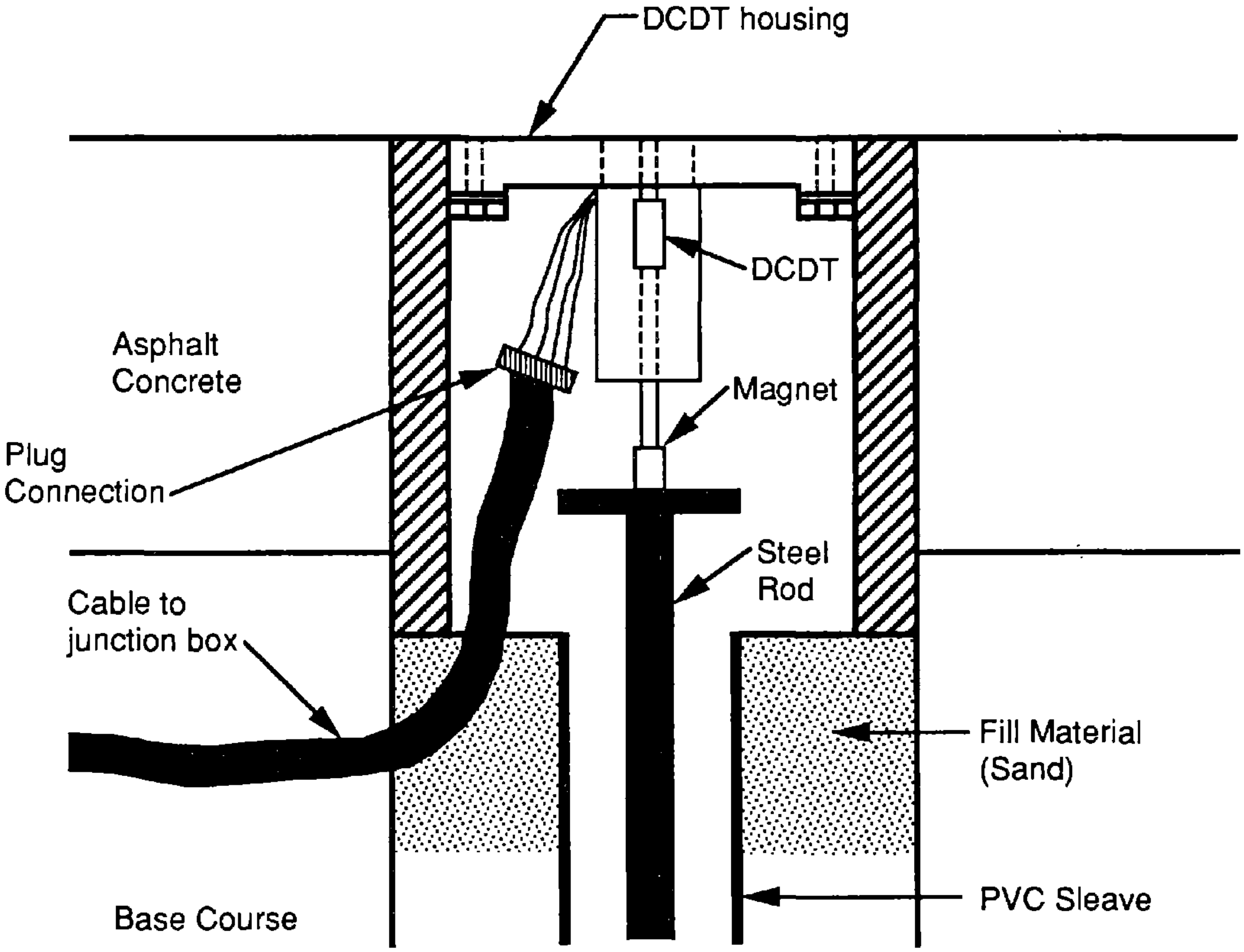


Figure 21. Details of DC-DC LVDT carrier which is the main portion of the single layer deflectometer.

Strain Gauges

Tensile Strains HMAC - Basecourse interface

Method Proposed - strain gauges encapsulated within asphalt mastic (Alberta Research Council type).

Transducer description / specifications

Gauge Type : Bonded Metallic Foil
Resistance: 120 ohms
Carrier: Asphalt mastic
No. Gauges: 2 per carrier, 1 active, 1 spare
Configuration: Quarter Bridge
Dimensions: Approx. 6-in by 6-in by .75-in (152-mm by 152-mm by 19-mm)
Approx. Cost: $300 per carrier

Installation occurs after the completion of the basecourse and prior to paving. Immediately prior to paving the carriers should be covered with approximately 1-in (25.4-mm) of HMAC with the coarse aggregate removed in an effort to protect the gauges from damage. The wheels of the paver must not pass over the carriers during paving.

The installation process can be divided into three phases:

1. Gauge Placement.
2. Paving.
3. Installation Completion.

Paving is the most critical phase, for it is the paver along with associated trucks with personnel which present the greatest threat to the safety of the gauges. To minimize this threat, the gauges must be placed in a manner so that no wheels of any kind pass over them.

Gauge Placement. Two major considerations controlled where the gauges were placed. First is where the wheels of the paver and other construction traffic passed during paving (they can destroy the gauges), and second, the gauges were positioned in a manner to collect data relevant to the study. Figure 22 illustrates the paver path and the remaining areas of the pavement which were available for gauge placement. The gauges were placed on 12-in (305-mm) centers to allow gauges A1 and A2 to be under the approximate center of the outside and inside dual on the test vehicles. The strain gauges can be damaged by prolonged exposure to heat from sunlight and were placed just prior to paving.

Paving. This is the process that ultimately determines success or failure of the gauge installation. The paver was positioned so that it straddled the gauges. The trucks providing mix to the paver also straddled the gauges. They can and will crush the delicate gauges if given the opportunity. Rolling is an especially critical operation. Vibration was not used on the first course. Vibration would most likely break the gauges or the attached leads. The direction of rolling is also important, tension between the strain carriers and their lead wires could cause a break and subsequent gauge loss. Rolling was performed so the

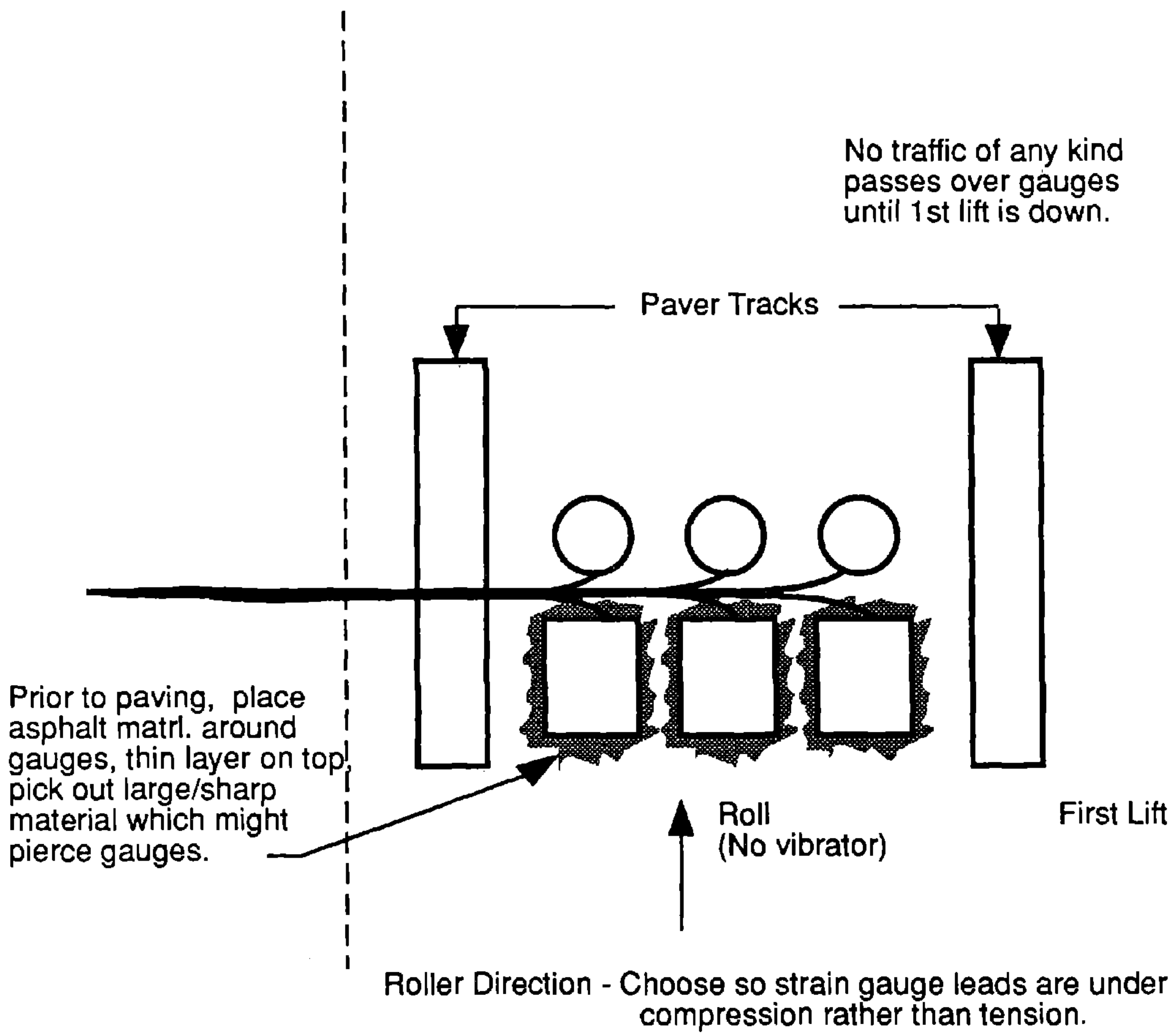


Figure 22. Diagram depicting how the paver must straddle the gauges to avoid damage.

connection between the cables and strain carriers was always in compression.

Just before the paver passed over the gauges, some of the mix was placed around and over the strain carriers. Care was taken to remove any large stones or sharp objects which might pierce a gauge. The installation went well and the result was a strain gauge installation which is a permanent part of the pavement structure. Once the first asphalt layer was down and cool and the gauges had survived, the remaining paving was completed normally. Vibration was used while rolling the second and third layers.

Temperature Measurement

Method proposed: Thermocouple - Copper/Constantan (Type "T")

Transducer description / specifications

Type:	Thermocouple Type "T"
Supplier:	Omega Engineering
Temp. Range:	-75 to +350°F typical
Accuracy:	Approx. +/- 1.0°C. Individual thermocouples should be calibrated prior to installation.
Approx. Cost:	Varies with packaging and specified accuracy. Typical cost is $60 for 5 thermocouples (self adhesive type).

Thermocouples are without a doubt the most widely used temperature measurement transducer. Their use is greatly simplified when combined with data acquisition systems featuring reference junctions and automatic voltage to temperature conversion. Figure 23 illustrates the locations of the thermocouple string installed in the two pavement sections. Note that a subbase layer was present in this area of the existing pavement.

VEHICLES

Three classifications of vehicles were used for the equivalency factor experiments. For the pilot study, a two axle - single unit truck was used to apply all loads. This truck also provided the standard loading condition of 18,000-lb (8,172-kg) single axle - dual tire load. During the primary testing program three general categories of vehicle classification were used. These consist of:

- Single axle vehicle.
- Tandem axle vehicle.
- Tridem axle vehicle.

The three classifications used in the testing are represented in figure 24. The single axle vehicle is the same one used for all pilot studies and to apply the standard 18-kip (8,172-kg) load. It is a single unit two-axle truck capable of being loaded to about 30,000-lb (13,620-kg) on the rear axle. The tandem axle vehicle is the common 3S-2 configuration. The tridem vehicle was a tractor semi-trailer with a set of rear tridem axles. The values of the individual axle loads for the low (3), medium (2), and high (1) levels of load for the three truck types is shown in table 3.

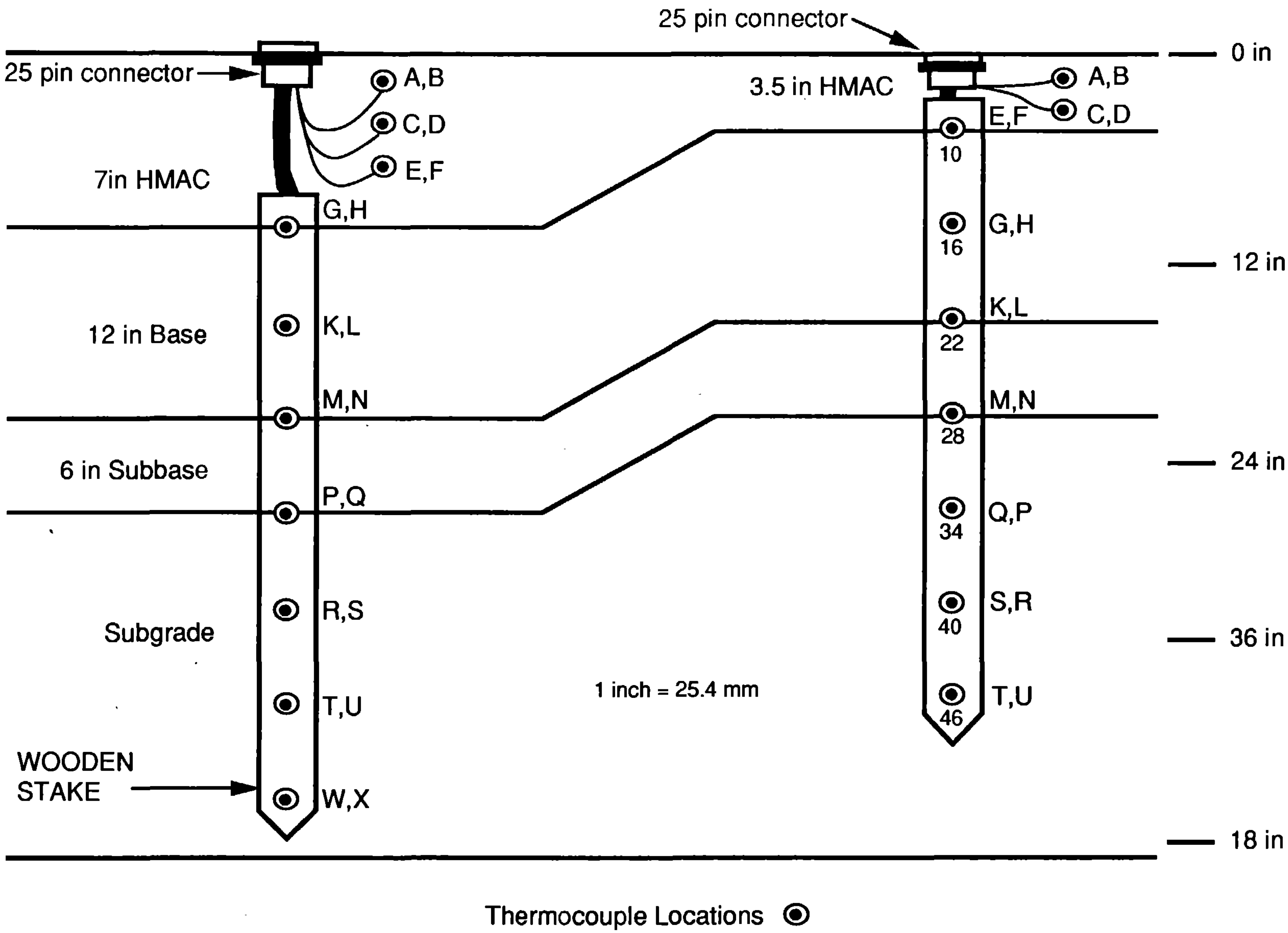


Figure 23. Illustration of thermocouple string layout in the two test setions.

DATA COLLECTION EQUIPMENT

Data collection on the study was accomplished with various types of electronic data collection equipment. Several key components make up the data collection and storage system used to interpret and store the raw data signals sent by the strain gauges and deflection transducers installed on the instrumented pavement test sections. The data handling system consists of a set of data conditioning equipment manufactured by the FHWA and a portable 80386 based microcomputer. The connection of these devices is shown schematically in figure 25. Strain and deflection signal conditioning and amplification was accomplished using a system manufactured by the FHWA for this project. These devices have the capability of sampling 16 gauges at high rates to allow for data collection at highway speeds. Data from each strain and deflection gauge are read and transferred to a microcomputer for permanent storage and analysis.

Signal Conditioning Equipment

The signal conditioning equipment consists of three major components as indicated in the top layer of figure 25. The strain signal conditioner is the largest of the boxes and can handle 16 strain gauges. This unit transforms the voltage signals from the strain gauges to measurements of strain and amplifies them by a factor of 276.25, according to the Electronics Laboratory at the TFHRC of the FHWA. They were the manufacturers of this equipment specifically for use on the project. The deflection signal conditioner simply provides a 6-volt power supply to the DCDT's in the pavement and amplifies the return signal by 1.784 to produce an output voltage ranging from -5 to +5 volts. The third component of the signal conditioning setup is the variable amplifier. This device can amplify the signals from the deflection and strain signal conditioning units by the following factors: 0.5, 1.0, 2.0, 4.0, 8.0, 16.0, and 32.0. The device then transfers the signals to an interface with the CODAS hardware and software for the data collection and storage computer.

Computer and CODAS Software

An IBM PS2 Model P70/386 was used for data collection, storage, and analysis on the project. CODAS hardware and software was installed on the IBM computer to transfer the analog signals from the signal conditioning equipment to digital signals for use by the computer. These digital signals are stored in the form of computer files with the values of voltage ranging from -5 to +5 volts as output by the signal conditioning equipment. The CODAS software stores the extremely large amount of data in binary computer files. CODAS interactively transfers these files into graphical profiles to interface with the computer user. The user can examine the data readily in the format of graphical profiles of the strain and deflection signals received from the instruments. More details about the data format is presented in section 6 on data analysis.

SUMMARY OF PAVEMENT RESPONSE MEASUREMENTS

The basic data for the purposes of this study were the strain and deflection measurements from instrumented pavement test sections. This section describes the details of the pavement test sections, instruments,

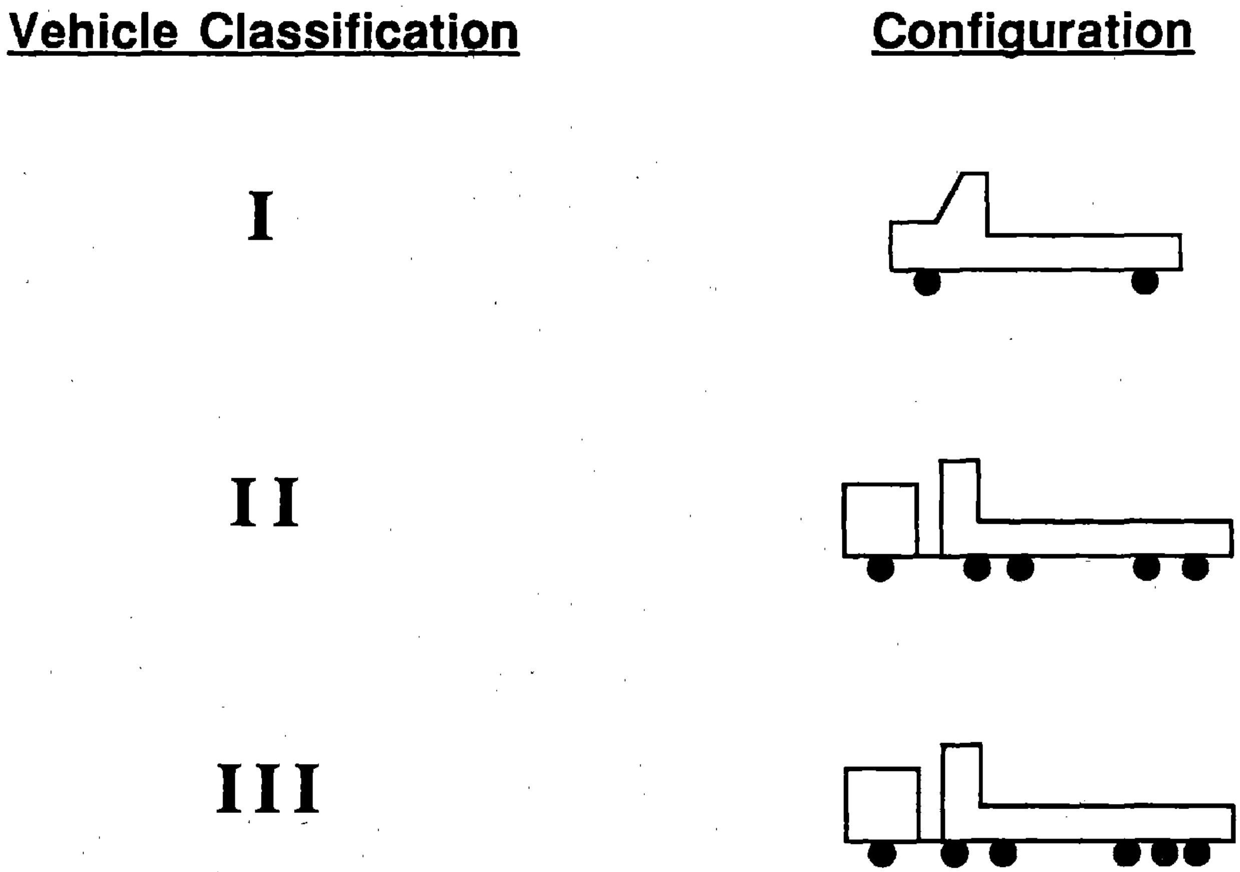


Figure 24. Definition of vehicle classifications for the primary experiment.

Table 3. Wheel weights for three load levels and three truck types used in the experiment.

VEHICLE 1 — Single Axle Truck

	FRONT	REAR	Shooting for	
Load 2 (lbs)	4000	9000	(9,000)	← Conducted test with Load 2 first.
Load 1 (lbs)	2900	13,500	(13,500)	
Load 3 (lbs)	3600	4500	(4,500)	

VEHICLE 2 — Tandem Axle Tractor Trailer

	FRONT			REAR		Shooting for
Load 1 (lbs)	4200	10,800	10,700	11,000	10,800	(11,000)
Load 2 (lbs)	4200	8100	8100	7000	8600	(8,000)
Load 3 (lbs)	4200	5000	5000	4500	5500	(5,000)

VEHICLE 3 — Tridem Axle Trailer

	FRONT			REAR			Shooting for
Load 1 (lbs)	4200	5600	5200	11,000	10,000	10,000	(10,000)
Load 2 (lbs)	4200	4800	4600	7500	7000	7000	(7,000)
Load 3 (lbs)	4200	4500	4500	4400	4200	4000	(4,000)

1 lbs. = 0.454 kg

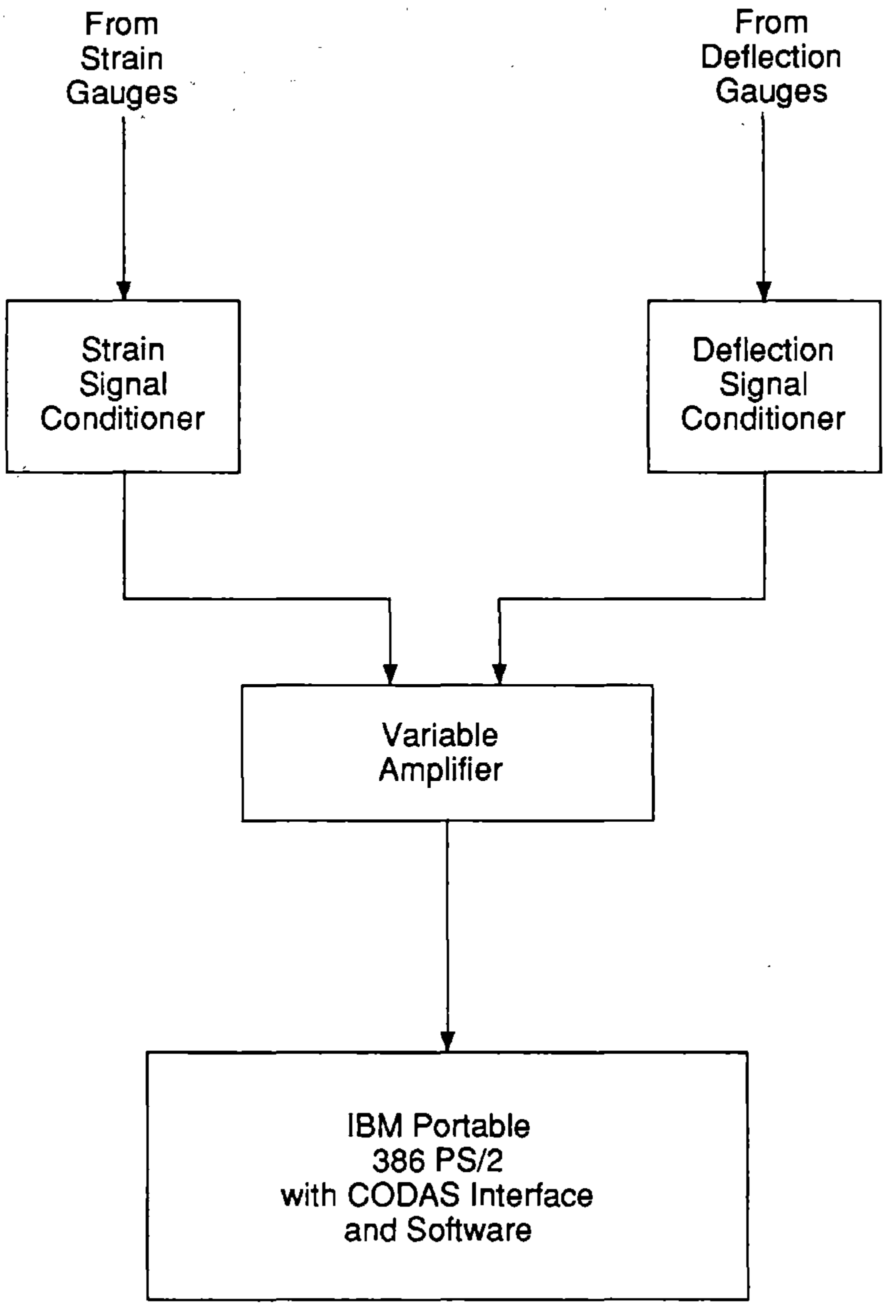


Figure 25. Schematic of signal conditioning and data collection equipment supplied by the FHWA.

data collection equipment, and vehicles which were used to collect the pavement response measurements. The details were presented to provide a clear record of how the measurements were collected. It also allows an evaluation of the level of detail and sophistication of the experimental testing equipment setup.

SECTION 5. EXPERIMENT DESIGN

Extensive work was performed to plan the collection of the field data necessary for this study. A large number of details were considered in planning for field tests of this magnitude. Detailed experiment designs were prepared to define exactly what data should be collected to allow for statistically valid, yet efficient experiments. Detailed planning and preparations were also required as discussed in section 4 to develop pavement instrumentation and data collection equipment setups that would be adequate for the needs of the project and provide efficient data collection and transfer for analysis. In order to achieve the goals of the project several designed experiments were performed. This section describes the experiment design concepts followed by a specific discussion of the objectives and scope of each experiment design.

EXPERIMENT DESIGN CONCEPTS

The research involved pavement response based load equivalency factors. The available methods for calculating these equivalency factors were reviewed and several methods were selected for further study. The methods were evaluated and screened using pavement response predictions from mathematical models. All of the equivalency factor methods are applicable for flexible pavements only. The philosophical question now exists - Which one is best? Therefore, each of the equivalency factor methods selected for further study were evaluated to recommend a set of response based equivalency factors which produce accurate and reliable results.

Figure 26 conceptually explains the interaction between the previous investigative studies and the experimental field studies described in this section. The figure is a simplified system diagram of input (independent factors), model, and output (dependent factors). For the investigative study, the output factors are the primary response factors of stress, strain, and deflections which are applied to calculate response based equivalency factors with the selected methods. In the investigative phase of the study, the 'model' is a mathematical algorithm which predicts primary pavement responses using the input of pavement structure geometry, material properties, environmental considerations (moisture and temperature), and traffic factors. For the field studies, the pavement structure(s) where the instrumentation exists becomes the model. Thus, the pavement structure geometry and material properties become fixed factors and the variable input factors are traffic and environment as shown in the lower part of the figure.

As shown in the lower part of figure 26, field data consisting of primary pavement responses and truck loads were gathered to verify and develop the concept of primary pavement response truck load equivalencies. The data consisted of truck characteristics and load, pavement properties, and the measured pavement response to the applied load in terms of strain and deflection.

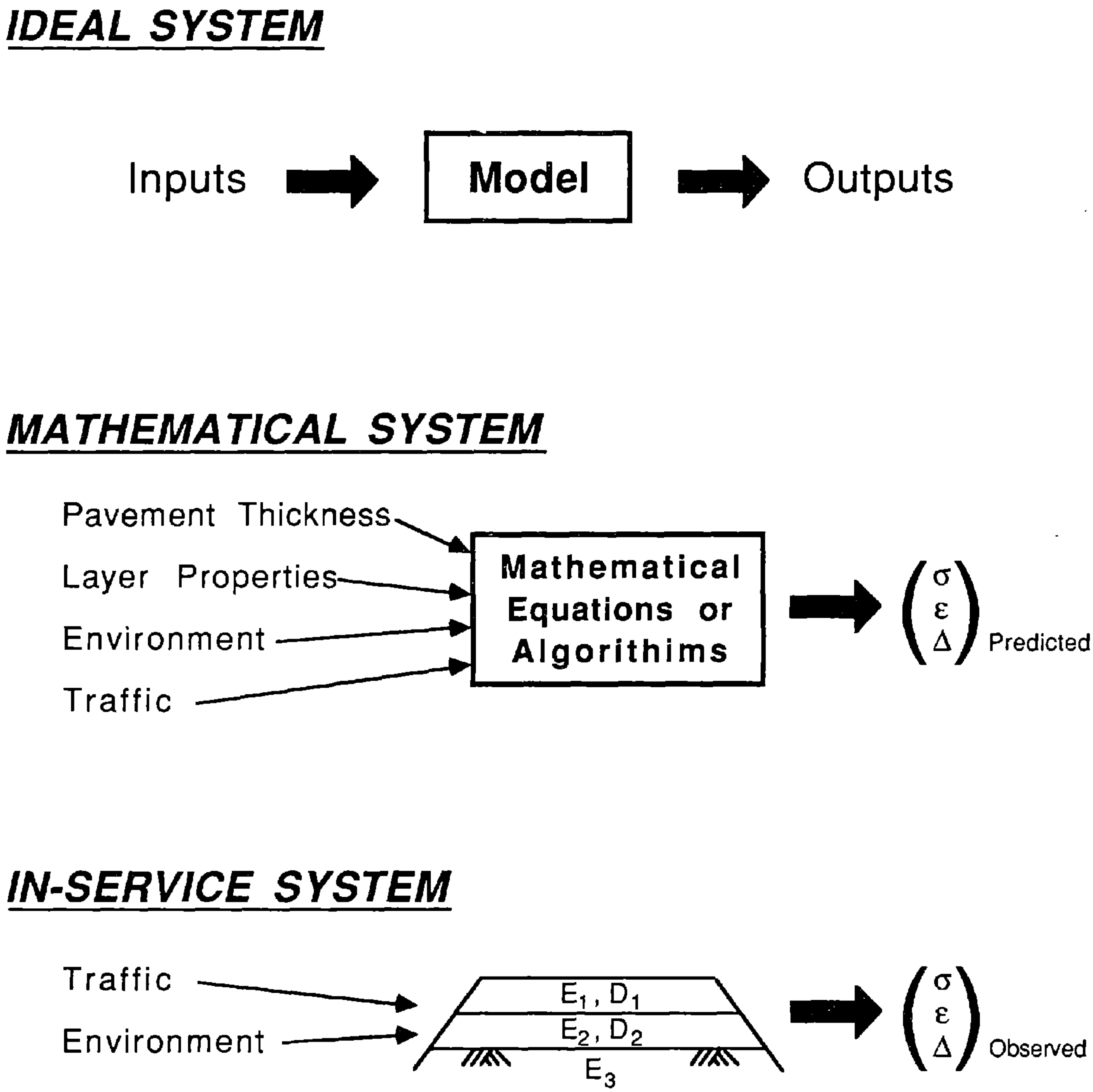


Figure 26. Conceptual diagram illustrating interaction of investigative and testing phase of the project.

OBJECTIVES OF THE EXPERIMENTS

The field experiments for the equivalency factor study consisted of designed experiments, which were set up to maximize efficiency and achieve the project objectives. The general objective of these experiments is to obtain pavement response data to validate and compare the primary response load equivalency relationships selected for further study on this project. The experiments were developed to collect field test data to verify the use of primary responses of strain and deflection for predicting load equivalency factors (LEF's) for various axle configurations and weights.

Several specific objectives accomplished by the experiment design are as follows:

- Develop and debug state-of-the-art pavement instrumentation and data collection capability for measuring pavement response variables.
- Measure variability associated with controlled and uncontrolled dependent and independent variables.
- Measure the effects of controlled independent variables including load, axle configuration, speed, tire pressure, temperature, and pavement structure on the dependent response variables of strain and deflection.
- Establish if vehicle classification is necessary or if axle load and type are adequate for describing traffic for equivalency factor purposes.
- Determine if primary pavement response based load equivalency factors are a viable concept and if some basic models for predicting primary response LEF's could be developed with testing over a much larger factorial.

The basic equivalency factor experiments consisted of a pilot study followed by the primary study.

Pilot Study. The pilot study was a small controlled experiment on the instrumented pavement test sections. The purposes of the pilot study were debugging the measurement systems; quantifying the inherent variations in the measurement processes; and providing a rough estimate of the effects of load, pavement structure, and speed on the primary responses of the pavement. Some tests were run at only 5-mi/h (8-km/h) in order to minimize any dynamic effects and provide a clear estimate of the 'static' pavement response. A higher speed was also used in order to get a basic indication of the effect of speed and vehicle dynamics on the instrumentation and measurement sensitivity.

Primary Experiment. The primary experiment was a larger controlled experiment on the instrumented test sections. The results from this experiment were the primary means by which to determine the effects of load, tire pressure, speed, temperature, and axle configuration on load equivalency factors for two pavement structures. The instrumentation and measurement variability determined in the pilot study were used to accurately quantify the effects of the important factors in the experiment listed above.

PILOT TEST PROGRAM

The pilot studies served to implement and debug the measurement instrumentation and data collection devices, and provided useful results concerning the effects of several important independent factors.

In general, the pavement instrumentation and data collection devices tested and debugged on the pavement sections during the pilot study consisted of the following:

- In-pavement strain gauges.
- In-pavement deflection transducers.
- In-pavement thermocouple.
- Vehicle lateral position indicators.
- Data acquisition and control systems.
- Data handling and storage computer.

The variables (factors) collected by these instruments are as follows:

Parameter	Transducer
Strain	Strain Gauge
Deflection	Linear Variable Differential Transformer
Temperature	Thermocouple
Lateral location within lane	Sonar Distance Detector

It was necessary to quantify the variation of each component in the measurement process so that further test results for identifying the vehicle and pavement factor effects could be interpreted.

The small pilot study was performed quickly and economically yet produced significant results. Only one vehicle (two axle-single unit truck) was used. The experiment design for the pilot study is shown in figure 27. The following factors were used to test the observed variations in a nested factorial experiment:

- Pavement structure (P).
- Instruments nested within pavement structure (I(P)).
- Load (L).
- Speed (S).

The steering axle load was held constant throughout the experiment. Only the drive axle load was varied and observed in the experiment. The two sets of instruments in each section allowed duplicate measurements in order to quantify instrument variations.

There were three main objectives to be accomplished with the pilot study:

1. Instrument shakedown - The instrumentation for measuring strains and deflections and recording the data to the computer was new and required a thorough shakedown and test out process

Speed (S)	Load (L), kip	Pavement Structure (P): 7 in (1), Instruments (I(P)) 1	7 in (1), Instruments (I(P)) 2	3.5 in (2), Instruments (I(P)) 3 (1)	3.5 in (2), Instruments (I(P)) 4 (2)
5 mi/h (1)	High (1)	1111	1211	2111	2211
5 mi/h (1)	Low (2)	1112	1212	2112	2212
15 mi/h (2)	High (1)	1121	1221	2121	2221
15 mi/h (2)	Low (2)	1122	1222	2122	2222
30 mi/h (3)	High (1)	1131	1231	2131	2231
30 mi/h (3)	Low (2)	1132	1232	2132	2232
45 mi/h (4)	High (1)	1141	1241	2141	2241
45 mi/h (4)	Low (2)	1142	1242	2142	2242

1 inch = 25.4 mm
mi/h = 1.61 km/h
Kip = 4.54 Kg

Figure 27. Factorial layout for the pilot study.

to make sure they were working properly and efficiently. This included check out of not only the gauges which were installed in the pavement, but also the signal conditioning units constructed by Federal Highway Administration and the CODAS data collection software and computer provided by the FHWA.

2. Determine instrument variability - The pilot study was also to give an initial indication of the variability of the strain and deflection gauges and measurement equipment. This includes an estimate of the repeatability and reproducibility of the measurement setup.

3. Initial determination of effects of various measurement parameters - The pilot tests are also designed to provide a broad indication of which of the main variables have the most influence over the strain and deflection readings. The main variables examined in the pilot test include pavement structure, load, and speed. The variable axle type was also included in the experiment; however, load was confounded with axle type and, thus, each axle type was analyzed separately.

PRIMARY EXPERIMENT PROGRAM

The primary experiment served to quantify the effects of a number of vehicle and pavement factors in influencing pavement response and load equivalency factors. The tests were performed on the same two instrumented test sections used in the pilot study experiment. These are described in detail in section 4.

The factors and levels included in the experiment are as shown in table 4.

The experiment covered all treatment combinations with full replication provided by a repeat of the entire set of instrumentation on each section. The three vehicle classifications were described previously. The factors axle type and axle load are interdependent and are actually a fixed set of values for each vehicle run. Thus, the vehicle load factors can be combined by each vehicle run into one-way load classifications as shown in figure 28. This produces a single factor of load with nine fixed levels. The overall factorial experiment layout is shown in figure 29 and each combination from figure 29 is repeated for each cell in the overall experiment. For each of the 16 cells in the overall factorial, all 9 loads were applied.

SUMMARY OF EXPERIMENT DESIGNS

The experiment design phase was important to plan the field experiments to be accomplished. This planning was to ensure that statistically valid data would be collected which would result in analyses to achieve the intended objectives of the project. The pilot study was important to quantify the errors in the measurement process and to understand the instrumentation variations. The main experiment was the primary objective of the field test and is the subject of all of the data analyses and interpretation discussed in the following sections. Table 5 shows a summary of all the variable level values in the experiment design.

Table 4. Factors and levels in primary experiment.

Factors	Type	Levels
Pavement Structure - (PVMT)	F	weak, strong
Instruments nested in Pavements - (INST)	R	1,2,3,4
Axle Type (TRK)*	F	1, 2, 3
Axle Load (LOD)	F	Low, Medium, High
Tire Pressure - (TP)	F	75-psi (515-kPa) 110-psi (760-kPa)
Speed - (SPD)	F	5-mi/h (8-km/h) 45-mi/h (72-km/h)

* 1 - single axle dual tire
  2 - tandem axle group
  3 - tridem axle group

AXLE TYPE (TRK)								
LEVEL 1 = SINGLE AXLE VEHICLE			LEVEL 2 = TANDEM AXLE VEHICLE			LEVEL 3 = TRIDEM AXLE VEHICLE		
AXLE LOAD (LOD)			AXLE LOAD (LOD)			AXLE LOAD (LOD)		
9 Kips	18 Kips	27 Kips	20 Kips	32 Kips	44 Kips	24 Kips	42 Kips	60 Kips
LEVEL 3 = LOW LOAD	LEVEL 2 = MEDIUM LOAD	LEVEL 1 = HIGH LOAD	LEVEL 3 = LOW LOAD	LEVEL 2 = MEDIUM LOAD	LEVEL 1 = HIGH LOAD	LEVEL 3 = LOW LOAD	LEVEL 2= MEDIUM LOAD	LEVEL 1 = HIGH LOAD

Kips = 4.54 Kg

Figure 28. One-way classification of axle type and weight into nine levels of load.

			PAVEMENT STRUCTURE (PVMT)			
			Level 1= Strong = 7 in		Level 2 = Weak = 3.5 in	
			INSTRUMENTS (INST)		INSTRUMENTS (INST)	
			1	2	3	4
TIRE PRESSURE (TP)	Level 1= 75 PSI	SPEED (SPD) Level 1 = 5 mi/h	9 LOADS	9 LOADS	9 LOADS	9 LOADS
		SPEED (SPD) Level 2 = 45 mi/h	9 LOADS	9 LOADS	9 LOADS	9 LOADS
	Level 2 = 110 PSI	SPEED (SPD) Level 1 = 5 mi/h	9 LOADS	9 LOADS	9 LOADS	9 LOADS
		SPEED (SPD) Level 2 = 45 mi/h	9 LOADS	9 LOADS	9 LOADS	9 LOADS

1 inch = 25.4 mm
psi = 6.9 KPa
mi/h = 1.61 Km/h

Figure 29. Overall experimental layout of the main experiment.

Table 5. Experiment design factor levels.

AXLE TYPE (TRK)	AXLE LOAD (LOD), kip LEVEL 1 = High	LEVEL 2 = Medium	LEVEL 3 = Low
Level 1 = Single Axle Vehicle	27	18	9
Level 2 = Tandem Axle Vehicle	44	32	20
Level 3 = Tridem Axle Vehicle	60	42	24

		LEVEL 1 =	LEVEL 2 =
TIRE PRESSURE	=	75-psi (515-kPa)	110-psi (760-kPa)
SPEED	=	5-mi/h (8-km/h)	45-mi/h (72-km/h)
PAVEMENT	=	Strong 7-in (177-m)	Weak 3½-in (88-m)

SECTION 6. DATA ANALYSIS

The data collected following the planned experiment described in the previous section went through a multi-step analysis procedure to (1) transform the raw voltage signals from the instruments into a useable format, (2) convert signals to strains and deflections, (3) organize strains and deflections relative to the test factorial, (4) sort out errors and poor measurements, and (5) summarize the data in a concise manner for accurate statistical analysis. The entire process used for data reduction and analysis is described and a summary of the results of each step is presented.

The measured primary pavement responses of strain and deflection were measured and the relative sensitivity of these factors to the variables being studied in the experiments was established. The basic concept in the data analysis procedures is to quantify the inherent errors or measurement variations that exist in the experimental process. These include uncontrollable errors in the equipment and testing procedures and variations associated with changes in the experimental factors being studied.

RAW CODAS DATA

The raw data collected from the strain and deflection gauges using the CODAS data collection software is stored in binary disk files. These files do not allow direct access to the numerical data without a computer transformation process. Due to the rapid sampling rate of the data collection equipment, literally millions of data points are stored in the binary data files. The CODAS software allows the user to examine these files readily in graphical format. These graphs show the strain or deflection trace as the instrument is sitting unloaded in the pavement and as a load approaches and passes the instrument. An example of a typical strain and deflection profile is shown in figure 30. A comprehensive set of all raw data collected in the main experiments in the form of strain and deflection profiles is available but were considered too voluminous to include in this report. These profiles can be shown in various levels of horizontal compression in order to reduce the pages required to present the data. The vertical scale of the plots is -5 volts to +5 volts in all cases. It is not feasible to print tabular values of all data points.

In order to use the CODAS data, key data points must be selected from the profiles and converted to numerical values. These key values are the unstrained or undeflected values and all subsequent peaks and valleys in the profile for each pass of a vehicle. The example profile shown in figure 30 has the peaks and valleys marked in the raw CODAS data file. The CODAS software has a utility program available to locate peak and valley occurrences in a raw data file. The software must be used in a trial and error process to set a sensitivity value and determine if the all peaks and valleys were selected properly by the software. It was almost impossible to have the software select all peak and valley data accurately. It was inevitable that once the optimum sensitivity level was selected for each file, the data file had to be reviewed manually and some peak and valley information added and some false peak and valley information deleted from the data files. This was a long manual process, but resulted in data files with all peaks and valleys identified or

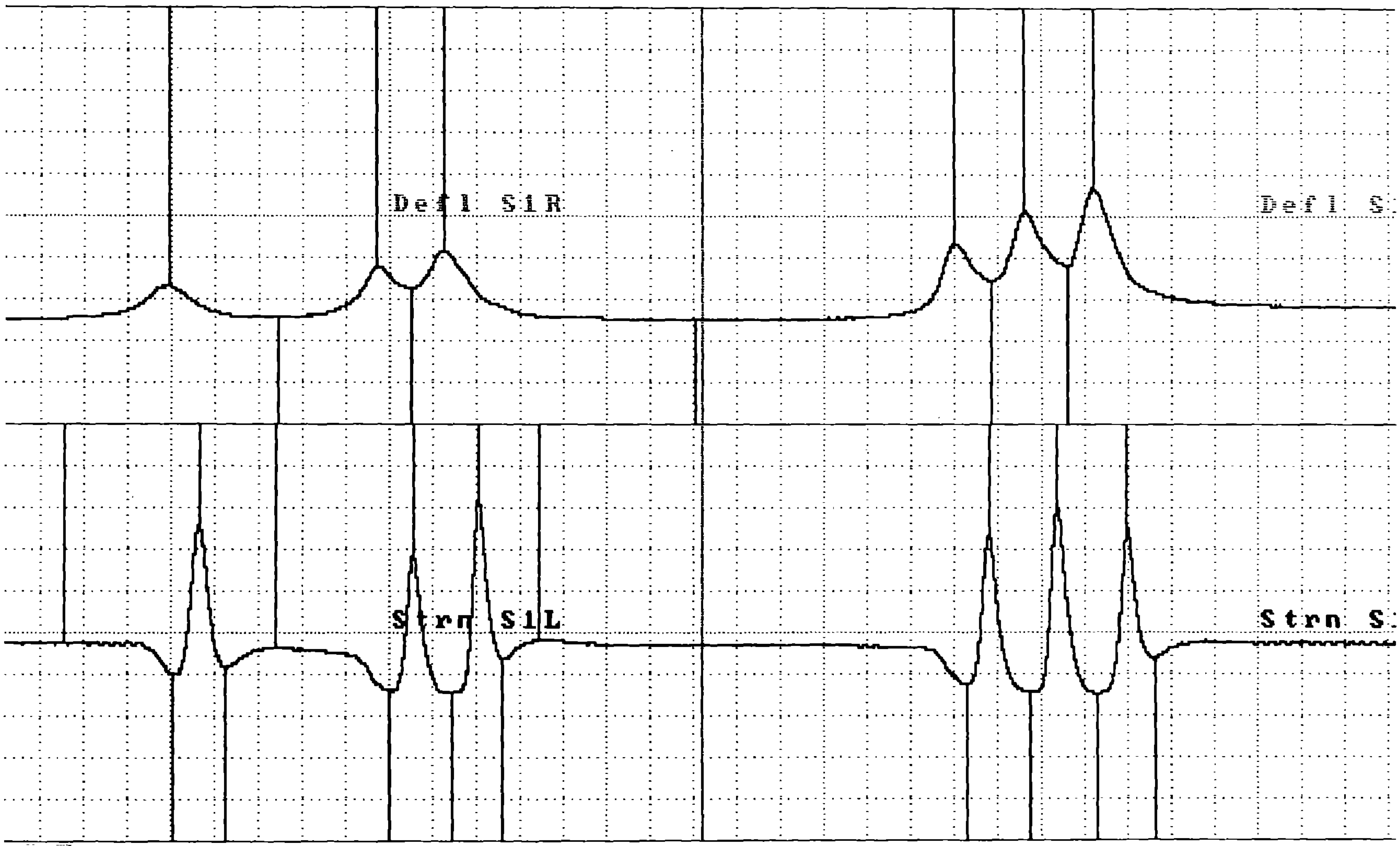


Figure 30. Typical strain and deflection profile in CODAS for the tridem tractor trailer.

indicated as "dummies" where poor data existed. In this way all data from all channels became consistent and further procedures could be automated to analyze the data in a consistent manner. The process of identifying and verifying all peaks and valleys in the huge data files was extremely time-consuming.

Once the raw data files were marked with peak and valley data point occurrences, a CODAS software utility program was used to transform the binary data to an ASCII data file. These raw data files are all in units of volts. The ASCII files were loaded into MicroSoft EXCEL and transformed by automated EXCEL macros into a complete summary worksheet for each individual channel of strain and deflection data. Tables 6 and 7 provide one-page examples of these initial level data worksheets for deflection and strain data, respectively. The worksheets contain peak and valley information, calibration factors, amplification factors, the calculated strain and deflection values, and identifier information indicating the factor level of each individual run. Referring to table 6, the columns include "Valley" which is the low point on the raw data profile; "Peak" is the high point; "Calibration Factor" is the calibration for the deflection measuring DCDT's; "AMP" is the amplification of the signal processing equipment; "Line" indicates how close to the gauges the truck passed; "Deflection in Mils" is a calculated value using the raw peak and valley information; "Run" is a unique identifier for each pass of the vehicle; "Truck" is the truck type, either 1, 2, or 3; "Load" is the load level, either 1 for high, 2 for medium, and 3 for low loads; "TP" is tire pressure; "Speed" is the speed of the vehicle, either 5- or 45-mi/h (8- or 72-km/h); and "Wheel" is the indicator of each wheel on the vehicle.

Calibration of Raw Deflection Readings

The raw deflection signals are acquired in units of volts. Therefore, they must be calibrated to deflection units of mils. Each DC-LVDT has a separate calibration factor (CF); however, they are all approximately 34 volts per inch of deflection. The signal conditioning unit provides a gain factor of 1.784. Therefore, a multiplication factor (MF) can be derived to convert the voltage output of the gauges to units of mils as follows:

$$\begin{aligned} \text{MF (mils/volt)} &= 1/(34 \text{ (volt/in)} * .001 \text{ (in/mil)} * 1.784) \\ &= 16.486 \text{ mil/volt} \end{aligned} \qquad (1)$$

The actual values for each gauge was measured in a calibration procedure and used in the worksheet that corresponds to its channel.

Calibration of Raw Strain Readings

Because of its outstanding sensitivity, the Wheatstone bridge circuit is the most frequently used circuit for static strain measurements.[11] By using a computer in conjunction with the measurement instrumentation, we can simplify using the bridge circuit, increase measurement accuracy, and compile large quantities of data from multichannel systems. The computer can also remove the requirement for balancing the bridge, compensate for

Table 6. Example page from initial level worksheets for deflection data.

					DEFLECTION - 7" PVT - 1L						
Data file: E:MAIN1											
Source channel: 1					Deflection	Identification					
Valley	Peak	Cal.Fac.	Amp	Line	(mils)	Run	Truck	Load	TP	Speed	Wheel
-0.6689	0.08545	17.182	1	5	12.96	m1-1	I	1	110	5	1
-0.6689	0.6982	17.182	1	5	23.49	m1-1	I	1	110	5	2
-0.7373	0.07324	17.182	1	3	13.93	m1-2	I	1	110	5	1
-0.7373	0.7666	17.182	1	3	25.84	m1-2	I	1	110	5	2
-0.7153	-0.105	17.182	1	30	10.49	m1-3	I	1	110	45	1
-0.7153	0.5322	17.182	1	30	21.43	m1-3	I	1	110	45	2
-0.7227	-0.1538	17.182	1	1	9.77	m1-4	I	1	110	45	1
-0.7227	0.6348	17.182	1	1	23.32	m1-4	I	1	110	45	2
-0.7153	-0.1025	17.182	1	1	10.53	m1-5	I	1	110	45	1
-0.7153	0.6494	17.182	1	1	23.45	m1-5	I	1	110	45	2
-0.7397	-0.2173	17.182	1	1	8.98	m1-6	I	1	110	45	1
-0.7397	0.5933	17.182	1	1	22.90	m1-6	I	1	110	45	2
-0.7227	-0.1196	17.182	1	1	10.36	m1-7	I	1	110	45	1
-0.7227	0.7397	17.182	1	1	25.13	m1-7	I	1	110	45	2
-0.708	0.05615	17.182	1	1	13.13	m1-8	I	1	75	5	1
-0.708	0.6348	17.182	1	1	23.07	m1-8	I	1	75	5	2
		17.182	1	30	0.00	m1-9	I	1	75	5	1
	0.6543	17.182	1	30	11.24	m1-9	I	1	75	5	2
-0.7764	0.05615	17.182	1	5	14.30	m1-10	I	1	75	5	1
-0.7764	0.6226	17.182	1	5	24.04	m1-10	I	1	75	5	2
-0.7202	-0.144	17.182	1	1	9.90	m1-11	I	1	75	45	1
-0.7202	0.6958	17.182	1	1	24.33	m1-11	I	1	75	45	2
-0.6958	-0.08545	17.182	1	1	10.49	m1-12	I	1	75	45	1
-0.6958	0.8105	17.182	1	1	25.88	m1-12	I	1	75	45	2
-0.5981	0.09766	17.182	1	5	11.95	m2-1	I	2	110	5	1
-0.5981	1.787	17.182	1	5	40.98	m2-1	I	2	110	5	2
-0.498	0.04395	17.182	1	6	9.31	m2-2	I	2	110	5	1
-0.498	1.511	17.182	1	6	34.52	m2-2	I	2	110	5	2
-0.4102	0.2563	17.182	1	5	11.45	m2-3	I	2	110	5	1
-0.4102	1.768	17.182	1	5	37.43	m2-3	I	2	110	5	2
-0.5029	0.08789	17.182	1	3	10.15	m2-4	I	2	110	45	1
-0.5029	1.821	17.182	1	3	39.93	m2-4	I	2	110	45	2
-0.4224	0.2295	17.182	1	3	11.20	m2-5	I	2	110	45	1
-0.4224	1.79	17.182	1	3	38.01	m2-5	I	2	110	45	2
-0.3101	0.4102	17.182	1	5	12.38	m2-6	I	2	75	5	1
-0.3101	2.068	17.182	1	5	40.86	m2-6	I	2	75	5	2
-0.1807	0.4199	17.182	1	5	10.32	m2-7	I	2	75	5	1
-0.1807	2.336	17.182	1	5	43.24	m2-7	I	2	75	5	2
-0.2197	0.3564	17.182	1	30	9.90	m2-8	I	2	75	45	1
-0.2197	1.733	17.182	1	30	33.55	m2-8	I	2	75	45	2
-0.3467	1.001	17.182	2	3	11.58	m2-9	I	2	75	45	1
-0.3467	4.055	17.182	2	3	37.82	m2-9	I	2	75	45	2
-0.2954	0.9644	17.182	2	1	10.82	m2-10	I	2	75	45	1
-0.2954	4.097	17.182	2	1	37.74	m2-10	I	2	75	45	2
-0.2563	0.8862	17.182	2	3	9.82	m2-11	I	2	75	45	1
-0.2563	3.499	17.182	2	3	32.26	m2-11	I	2	75	45	2
-0.3027	1.001	17.182	2	1	11.20	m2-12	I	2	75	45	1
-0.3027	3.853	17.182	2	1	35.70	m2-12	I	2	75	45	2
-0.03906	1.333	17.182	2	5	11.79	m2-13	I	2	75	5	1
-0.03906	4.243	17.182	2	5	36.79	m2-13	I	2	75	5	2
-0.09521	1.628	17.182	2	1	14.80	m3-1	I	3	110	5	1
-0.09521	1.396	17.182	2	1	12.81	m3-1	I	3	110	5	2
-0.1904	1.626	17.182	2	1	15.60	m3-2	I	3	110	5	1

Table 7. Example page from initial level worksheets for strain data.

			STRAIN - 7" PVT - 1L						
Data file: E:MAIN1									
Source channel: 3			Strain	Identification					
Peak	Amp	Line	(in/in)	Run	Truck	Load	TP	Speed	Wheel
-0.3345	1	5		m1-1	1	1	110	5	base
0.0195	1	5	4.99E-04	m1-1	1	1	110	5	1
0.0513	1	5	5.43E-04	m1-1	1	1	110	5	2
-0.3345	1	3		m1-2	1	1	110	5	base
0.0684	1	3	5.67E-04	m1-2	1	1	110	5	1
0.1245	1	3	6.46E-04	m1-2	1	1	110	5	2
-0.3271	1	1		m1-3	1	1	110	45	base
-0.1416	1	1	2.61E-04	m1-3	1	1	110	45	1
0.0513	1	1	5.33E-04	m1-3	1	1	110	45	2
-0.3296	1	1		m1-4	1	1	110	45	base
-0.1245	1	1	2.89E-04	m1-4	1	1	110	45	1
0.0269	1	1	5.02E-04	m1-4	1	1	110	45	2
-1.3040	4	1		m1-5	1	1	110	45	base
-0.4761	4	1	2.92E-04	m1-5	1	1	110	45	1
0.2368	4	1	5.43E-04	m1-5	1	1	110	45	2
-1.2620	4	1		m1-6	1	1	110	45	base
-0.4810	4	1	2.75E-04	m1-6	1	1	110	45	1
0.4688	4	1	6.09E-04	m1-6	1	1	110	45	2
-1.3350	4	1		m1-7	1	1	110	45	base
-0.4785	4	1	3.02E-04	m1-7	1	1	110	45	1
0.3638	4	1	5.98E-04	m1-7	1	1	110	45	2
-1.2040	4	1		m1-8	1	1	75	5	base
0.3247	4	1	5.38E-04	m1-8	1	1	75	5	1
0.8569	4	1	7.26E-04	m1-8	1	1	75	5	2
-1.1960	4	1		m1-9	1	1	75	5	base
-1.1910	4	30	1.76E-06	m1-9	1	1	75	5	1
0.5420	4	1	6.12E-04	m1-9	1	1	75	5	2
-1.1870	4	5		m1-10	1	1	75	5	base
0.3589	4	5	5.44E-04	m1-10	1	1	75	5	1
0.6030	4	5	6.30E-04	m1-10	1	1	75	5	2
-1.1790	4	1		m1-11	1	1	75	45	base
-0.3198	4	1	3.03E-04	m1-11	1	1	75	45	1
0.5103	4	1	5.95E-04	m1-11	1	1	75	45	2
-1.1740	4	1		m1-12	1	1	75	45	base
-0.3442	4	1	2.92E-04	m1-12	1	1	75	45	1
0.2368	4	1	4.97E-04	m1-12	1	1	75	45	2
0.0562	1	5		m2-1	1	2	110	5	base
0.4419	1	5	5.43E-04	m2-1	1	2	110	5	1
0.8691	1	5	1.14E-03	m2-1	1	2	110	5	2
0.0659	1	6		m2-2	1	2	110	5	base
0.3345	1	6	3.78E-04	m2-2	1	2	110	5	1
0.6592	1	6	8.35E-04	m2-2	1	2	110	5	2
0.0659	1	5		m2-3	1	2	110	5	base
0.3833	1	5	4.47E-04	m2-3	1	2	110	5	1
0.8179	1	5	1.06E-03	m2-3	1	2	110	5	2
0.1392	2	3		m2-4	1	2	110	45	base
0.5518	2	3	2.91E-04	m2-4	1	2	110	45	1
1.7850	2	3	1.16E-03	m2-4	1	2	110	45	2
0.1465	2	3		m2-5	1	2	110	45	base
0.4834	2	3	2.37E-04	m2-5	1	2	110	45	1
1.6110	2	3	1.03E-03	m2-5	1	2	110	45	2
0.1318	2	5		m2-6	1	2	75	5	base
0.8765	2	5	5.24E-04	m2-6	1	2	75	5	1

nonlinearities in output and handle the switching and data storage in multichannel applications.

Assume V_{IN} is the input voltage to the bridge, R_g is the resistance of the strain gauge, R_1, R_2, and R_3 are the resistances of the bridge completion resistors, and V_{OUT} is the bridge output voltage. A 1/4 bridge configuration exists when one arm of the bridge is an active gauge and the other arms are fixed value resistors. Ideally the strain gauge, R_g, is the only resistor in the circuit that varies, and then only due to a change in strain on the bottom of the asphalt concrete layer to which it is attached. V_{OUT} is a function of V_{IN}, R_1, R_2, R_3 and R_g. This relationship is:

$$V_{OUT}/V_{IN} = [(R_3/(R_3+R_g)) - (R_2/(R_1+R_2))] \tag{2}$$

This equation holds for both the unstrained and the strained condition. Defining the unstrained value of gauge resistance as R_g and the change due to strain as ΔR_g, the strained value of gauge resistance is $R_g + \Delta R_g$. The actual effective value of resistance in each bridge arm is the sum of all the resistances in that arm and may include such things as lead wires, printed circuit board traces, switch contact resistance, interconnects, etc. As long as these resistances remain unchanged between the strained and unstrained readings, the measurement will be valid. Assume V_r is the difference of the ratios of V_{OUT} to V_{IN} from the unstrained to the strained state:

$$V_r = [(V_{OUT}/V_{IN})\text{strained} - (V_{OUT}/V_{IN})\text{unstrained}] \tag{3}$$

By substituting the resistor values that correspond to the two (V_{OUT}/V_{IN}) terms into this equation, we can derive an equation for $\Delta R_g/R_g$. This equation is:

$$\Delta R_g/R_g = (-4V_r)/(1+2V_r) \tag{4}$$

Note that it was assumed in this derivation that ΔR_g was the only change in resistance from the unstrained to the strained condition. The equation for gauge factor is:

$$GF = (\Delta R_g/R_g)/\epsilon \tag{5}$$

and combining these two equations we get an equation for strain in terms of V_r and GF.

$$\epsilon = (-4V_r)/GF(1+2V_r) \tag{6}$$

For the strain gauges used on this project, the gauge factor is 2.055. The input voltage is a constant 5 volts for both the strained and unstrained states.

Therefore, the equation for converting the voltage readings obtained to strain is:

$$\epsilon = (-.8(V_{OUT(S)}-V_{OUT(U)}))/(2.055+0.822(V_{OUT(S)}-V_{OUT(U)})) \tag{7}$$

In a bridge circuit the relationship between V_{OUT} and strain is nonlinear but for strains up to a few thousand microstrain the error is usually small enough to be ignored. At large values of strain, corrections must be applied to the indicated reading to compensate for this nonlinearity.

COMBINED RESPONSE DATA

The basic pavement response data calculation worksheets extracted from the raw CODAS pavement response data were combined by automated EXCEL macros to compare the left and right pavement instruments for each instrument location. Each pair of instruments in a single location was recorded and the maximum strain or deflection for each pair was selected for each wheel that passed over the instruments. This method was used in order to allow for slight lateral variation of the vehicles and still select the maximum response value. Tables 8 and 9 show example sheets that were used to compare the left and right instruments of an instrument set to select the maximum deflection or strain value for use at that instrument location. The EXCEL macro also deletes records that have errors in the data collection or a poor lateral line of the truck indicating that the wheel was not directly over one of the gauges. Complete lists of the strain and deflection data for each instrument location over all vehicle runs are available from FHWA.

SUMMARY DATA WORKSHEETS

The worksheets that contain the values from each instrument location from which the maximum response was obtained were then used to create a summary worksheet containing the average response for each cell in the test factorial. This involved averaging all observations which has a lateral line of 6 or less and fit in a particular cell. This data was incorporated into summary deflection and strain worksheets in which equivalency factors using the selected methods are calculated. Example of summary worksheets for deflection and strain data is shown in tables 10 and 11. The factors on the sheet are "pavement" thickness, instrument location, truck number, load number, tire pressure, speed, and the individual wheel location on each truck. For each of these cells in the matrix, the average value of the response, the standard deviation of all observations in the cell, the number of observations in the cell, and the maximum and minimum observations in the cell are also indicated on the worksheet. This worksheet is used to calculate the equivalency factors for every wheel on every truck. A complete listing of the summary worksheets is available from FHWA. A number of methods were used to calculate the equivalency factors as discussed in the following sections. Besides the various methods for calculating equivalency factors, the value used as the standard load response in the calculation also affects the resulting equivalency factors. The determination of these standard loading responses is discussed in the following section.

STANDARD LOAD RESPONSES

To calculate primary response load equivalency factors, a pavement response to some standard loading condition is required to compare against the response of the loading condition under consideration. The load under consideration can be at any tire pressure or speed, axle configuration, or axle weight desired. In this study, two levels of speed, two levels of tire pressure, three axle types, three load levels per axle, two different

Table 8. Example page from combined response data worksheets for deflection.

							7"PVT-1L		7" PVT-1R	
							Channel:1		Channel:2	
Identification						1L	Deflection	1R	Deflection	MAXIMUM
Run	Truck	Load	TP	Speed	Wheel	Line_L	(mils)	Line_R	(mils)	DEFLECTION
m3-6	I	3	110	45	1	1	10.54	1	4.81	10.54
m3-6	I	3	110	45	2	1	9.40	1	7.98	9.40
m3-7	I	3	110	45	1	1	10.47	1	4.66	10.47
m3-7	I	3	110	45	2	1	9.47	1	8.60	9.47
m3-8	I	3	75	5	1	1	15.52	1	5.57	15.52
m3-8	I	3	75	5	2	1	13.75	1	14.03	14.03
m3-9	I	3	75	5	1	1	16.72	1	6.08	16.72
m3-9	I	3	75	5	2	1	15.49	1	14.68	15.49
m3-10	I	3	75	5	1	1	16.80	1	6.22	16.80
m3-10	I	3	75	5	2	1	14.90	1	14.26	14.90
m3-12	I	3	75	45	1	1	11.58	1	5.36	11.58
m3-12	I	3	75	45	2	1	10.05	1	9.07	10.05
m3-13	I	3	75	45	1	5	10.92	5	4.55	10.92
m3-13	I	3	75	45	2	5	8.96	5	7.39	8.96
m4-1	II	1	110	5	1	5	9.73	5	8.95	9.73
m4-1	II	1	110	5	2	5	21.76	5	28.87	28.87
m4-1	II	1	110	5	2.5	5	17.66	5	23.19	23.19
m4-1	II	1	110	5	3	5	24.96	5	34.42	34.42
m4-1	II	1	110	5	4	5	15.48	5	26.84	26.84
m4-1	II	1	110	5	4.5	5	14.30	5	23.88	23.88
m4-1	II	1	110	5	5	5	20.72	5	39.11	39.11
m4-2	II	1	110	5	1	3	10.15	3	4.52	10.15
m4-2	II	1	110	5	2	3	26.72	3	22.77	26.72
m4-2	II	1	110	5	2.5	3	20.97	3	18.93	20.97
m4-2	II	1	110	5	3	3	30.45	3	26.17	30.45
m4-2	II	1	110	5	4	3	26.09	3	27.10	27.10
m4-2	II	1	110	5	4.5	3	21.35	3	23.07	23.07
m4-2	II	1	110	5	5	3	37.03	3	40.80	40.80
m4-3	II	1	110	5	1	5	9.73	5	9.08	9.73
m4-3	II	1	110	5	2	5	24.62	5	32.05	32.05
m4-3	II	1	110	5	2.5	5	18.21	5	24.74	24.74
m4-3	II	1	110	5	3	5	27.06	5	37.01	37.01
m4-3	II	1	110	5	4	5	15.81	5	28.53	28.53
m4-3	II	1	110	5	4.5	5	14.93	5	26.63	26.63
m4-3	II	1	110	5	5	5	21.36	5	40.65	40.65
m4-4	II	1	110	5	1	5	13.13	5	8.13	13.13
m4-4	II	1	110	5	2	5	30.29	5	33.61	33.61
m4-4	II	1	110	5	2.5	5	23.07	5	26.50	26.50
m4-4	II	1	110	5	3	5	33.60	5	38.77	38.77
m4-4	II	1	110	5	4	5	24.24	5	34.17	34.17
m4-4	II	1	110	5	4.5	5	20.89	5	29.61	29.61
m4-4	II	1	110	5	5	5	30.17	5	45.52	45.52
m4-5	II	1	110	5	2	1	27.18	1	26.60	27.18
m4-5	II	1	110	5	2.5	1	19.12	1	18.84	19.12
m4-5	II	1	110	5	3	1	29.48	1	30.46	30.46
m4-5	II	1	110	5	4	1	22.32	1	25.59	25.59
m4-5	II	1	110	5	4.5	1	18.45	1	21.59	21.59
m4-5	II	1	110	5	5	1	30.87	1	39.23	39.23
m4-6	II	1	110	5	1	1	12.25	1	6.37	12.25
m4-6	II	1	110	5	2	1	28.40	1	28.83	28.83
m4-6	II	1	110	5	2.5	1	20.64	1	21.82	21.82
m4-6	II	1	110	5	3	1	31.46	1	32.66	32.66
m4-6	II	1	110	5	4	1	25.05	1	29.04	29.04
m4-6	II	1	110	5	4.5	1	19.93	1	24.09	24.09

Table 9. Example page from combined response data worksheets for strain.

							7"PVT-2L		7" PVT-2R	
							Channel:7		Channel:8	
Identification						2L	Strain	2R	Strain	MAXIMUM
Run	Truck	Load	TP	Speed	Wheel	Line_L	(in/in)	Line_R	(in/in)	STRAIN
m1-1	1	1	110	5	base	5		5		0.00E+00
m1-1	1	1	110	5	1	5	3.99E-04	5	0.00014787	3.99E-04
m1-1	1	1	110	5	2	5	4.54E-04	5	0.00081469	8.15E-04
m1-2	1	1	110	5	base	3		3		0.00E+00
m1-2	1	1	110	5	1	3	4.06E-04	3	0.00013759	4.06E-04
m1-2	1	1	110	5	2	3	4.68E-04	3	0.00081131	8.11E-04
m1-3	1	1	110	45	base	1		1		0.00E+00
m1-3	1	1	110	45	1	1	2.68E-04	1	8.6095E-05	2.68E-04
m1-3	1	1	110	45	2	1	5.26E-04	1	0.00066357	6.64E-04
m1-4	1	1	110	45	base	1		1		0.00E+00
m1-4	1	1	110	45	1	1	2.31E-04	1	6.5383E-05	2.31E-04
m1-4	1	1	110	45	2	1	4.61E-04	1	0.00049523	4.95E-04
m1-5	1	1	110	45	base	1		1		0.00E+00
m1-5	1	1	110	45	1	1	2.41E-04	1	7.9121E-05	2.41E-04
m1-5	1	1	110	45	2	1	4.82E-04	1	0.00065312	6.53E-04
m1-6	1	1	110	45	base	1		1		0.00E+00
m1-6	1	1	110	45	1	1	2.63E-04	1	5.7633E-05	2.63E-04
m1-6	1	1	110	45	2	1	4.72E-04	1	0.00041353	4.72E-04
m1-7	1	1	110	45	base	1		1		0.00E+00
m1-7	1	1	110	45	1	1	2.86E-04	1	7.3943E-05	2.86E-04
m1-7	1	1	110	45	2	1	5.36E-04	1	0.0005364	5.36E-04
m1-8	1	1	75	5	base	1		1		0.00E+00
m1-8	1	1	75	5	1	1	3.96E-04	1	0.00013157	3.96E-04
m1-8	1	1	75	5	2	1	5.23E-04	1	0.00086093	8.61E-04
m1-9	1	1	75	5	base	1		1		0.00E+00
m1-9	1	1	75	5	1	30	0.00E+00	1	0	0.00E+00
m1-9	1	1	75	5	2	1	4.81E-04	1	0.00082742	8.27E-04
m1-10	1	1	75	5	base	5		5		0.00E+00
m1-10	1	1	75	5	1	5	3.90E-04	5	0.00017457	3.90E-04
m1-10	1	1	75	5	2	5	4.99E-04	5	0.00091446	9.14E-04
m1-11	1	1	75	45	base	1		1		0.00E+00
m1-11	1	1	75	45	1	1	2.55E-04	1	5.8479E-05	2.55E-04
m1-11	1	1	75	45	2	1	5.00E-04	1	0.00053038	5.30E-04
m1-12	1	1	75	45	base	1		1		0.00E+00
m1-12	1	1	75	45	1	1	2.18E-04	1	5.33E-05	2.18E-04
m1-12	1	1	75	45	2	1	4.58E-04	1	0.00045993	4.60E-04
m2-1	1	2	110	5	base	5		5		0.00E+00
m2-1	1	2	110	5	1	5	4.06E-04	5	0.00011696	4.06E-04
m2-1	1	2	110	5	2	5	8.15E-04	5	0.00154975	1.55E-03
m2-2	1	2	110	5	base	6		6		0.00E+00
m2-2	1	2	110	5	1	6	3.03E-04	6	0.00014448	3.03E-04
m2-2	1	2	110	5	2	6	6.57E-04	6	0.00152557	1.53E-03
m2-3	1	2	110	5	base	5		5		0.00E+00
m2-3	1	2	110	5	1	5	3.37E-04	5	0.00014792	3.37E-04
m2-3	1	2	110	5	2	5	7.77E-04	5	0.0016317	1.63E-03
m2-4	1	2	110	45	base	3		3		0.00E+00
m2-4	1	2	110	45	1	3	2.55E-04	3	4.9883E-05	2.55E-04
m2-4	1	2	110	45	2	3	8.47E-04	3	0.00085448	8.54E-04
m2-5	1	2	110	45	base	3		3		0.00E+00
m2-5	1	2	110	45	1	3	1.74E-04	3	3.785E-05	1.74E-04
m2-5	1	2	110	45	2	3	8.29E-04	3	0.00089341	8.93E-04
m2-6	1	2	75	5	base	5		5		0.00E+00
m2-6	1	2	75	5	1	5	3.58E-04	5	0.00010836	3.58E-04
m2-6	1	2	75	5	2	5	8.03E-04	5	0.00150319	1.50E-03

Table 10. Example page from summary worksheet for calculating deflection based equivalency factors.

			SUMMARY DEFLECTION DATA - AASHO STANDARD BY INSTRUMENT											
PVMT	INST	TRK	LOD	TP	SPD	WHL	DEFL	SD	N	MAX	MIN	DEFL RATIO	EQUIV.FAC CHRISTISON	EQUIV.FAC HUTCHINSON
7	1	1	1	75	5	1	11.49	1.06	3	12.38	10.32	0.46	0.05	0.05
7	1	1	1	75	5	2	40.30	3.26	3	43.24	36.79	1.61	6.04	6.04
7	1	1	1	75	45	1	10.85	0.76	4	11.58	9.82	0.43	0.04	0.04
7	1	1	1	75	45	2	35.88	2.60	4	37.82	32.26	1.43	3.88	3.88
7	1	1	1	110	5	1	10.91	1.40	3	11.95	9.31	0.43	0.04	0.04
7	1	1	1	110	5	2	37.64	3.24	3	40.98	34.52	1.50	4.66	4.66
7	1	1	1	110	45	1	10.68	0.74	2	11.20	10.15	0.43	0.04	0.04
7	1	1	1	110	45	2	38.97	1.35	2	39.93	38.01	1.55	5.32	5.32
7	1	1	2	75	5	1	13.72	0.83	2	14.30	13.13	0.55	0.10	0.10
7	1	1	2	75	5	2	23.55	0.68	2	24.04	23.07	0.94	0.78	0.78
7	1	1	2	75	45	1	10.19	0.41	2	10.49	9.90	0.41	0.03	0.03
7	1	1	2	75	45	2	25.11	1.10	2	25.88	24.33	1.00	1.00	1.00
7	1	1	2	110	5	1	13.44	0.68	2	13.93	12.96	0.54	0.09	0.09
7	1	1	2	110	5	2	24.66	1.66	2	25.84	23.49	0.98	0.93	0.93
7	1	1	2	110	45	1	9.91	0.70	4	10.53	8.98	0.39	0.03	0.03
7	1	1	2	110	45	2	23.70	0.98	4	25.13	22.90	0.94	0.80	0.80
7	1	1	3	75	5	1	16.35	0.72	3	16.80	15.52	0.65	0.20	0.20
7	1	1	3	75	5	2	14.81	0.73	3	15.49	14.03	0.59	0.13	0.13
7	1	1	3	75	45	1	11.25	0.47	2	11.58	10.92	0.45	0.05	0.05
7	1	1	3	75	45	2	9.50	0.77	2	10.05	8.96	0.38	0.02	0.02
7	1	1	3	110	5	1	15.44	0.48	4	15.96	14.80	0.62	0.16	0.16
7	1	1	3	110	5	2	13.89	0.98	4	15.06	12.71	0.55	0.11	0.11
7	1	1	3	110	45	1	10.62	0.21	3	10.87	10.47	0.42	0.04	0.04
7	1	1	3	110	45	2	9.50	0.12	3	9.62	9.40	0.38	0.02	0.02
7	1	2	1	75	5	1	12.85	0.67	3	13.30	12.08	0.51	0.08	0.08
7	1	2	1	75	5	2	30.69	1.53	3	32.43	29.57	1.22	2.18	3.04
7	1	2	1	75	5	2.5	23.50	2.51	3	26.26	21.35			
7	1	2	1	75	5	3	33.61	2.33	3	36.24	31.80			
7	1	2	1	75	5	4	32.57	2.10	3	34.03	30.17	1.30	3.10	9.75
7	1	2	1	75	5	4.5	25.83	1.55	3	27.10	24.10			
7	1	2	1	75	5	5	45.71	0.80	3	46.64	45.18			
7	1	2	1	75	45	1	7.51	#DIV/0!	1	7.51	7.51	0.30	0.01	0.01
7	1	2	1	75	45	2	27.89	#DIV/0!	1	27.89	27.89	1.11	1.51	2.41
7	1	2	1	75	45	2.5	22.77	#DIV/0!	1	22.77	22.77			
7	1	2	1	75	45	3	31.62	#DIV/0!	1	31.62	31.62			
7	1	2	1	75	45	4	28.39	#DIV/0!	1	28.39	28.39	1.13	1.63	3.99
7	1	2	1	75	45	4.5	25.63	#DIV/0!	1	25.63	25.63			
7	1	2	1	75	45	5	36.13	#DIV/0!	1	36.13	36.13			
7	1	2	1	110	5	1	11.31	1.61	6	13.13	9.73	0.45	0.05	0.05
7	1	2	1	110	5	2	29.44	2.51	7	33.61	26.72	1.17	1.87	3.05
7	1	2	1	110	5	2.5	22.62	2.45	7	26.50	19.12			
7	1	2	1	110	5	3	33.66	3.23	7	38.77	30.45			
7	1	2	1	110	5	4	28.72	2.80	7	34.17	25.59	1.14	1.87	6.63
7	1	2	1	110	5	4.5	24.76	2.62	7	29.61	21.59			
7	1	2	1	110	5	5	41.30	2.17	7	45.52	39.11			
7	1	2	1	110	45	1	8.60	1.48	3	9.86	6.96	0.34	0.02	0.02
7	1	2	1	110	45	2	26.74	1.83	3	28.14	24.67	1.06	1.28	1.75
7	1	2	1	110	45	2.5	21.75	1.01	3	22.48	20.60			
7	1	2	1	110	45	3	29.09	1.49	3	30.62	27.64			
7	1	2	1	110	45	4	26.65	0.41	3	27.06	26.25	1.06	1.31	3.94
7	1	2	1	110	45	4.5	24.39	0.54	3	24.91	23.83			
7	1	2	1	110	45	5	36.01	0.96	3	36.96	35.03			
7	1	2	2	75	5	1	10.04	0.79	3	10.53	9.12	0.40	0.03	0.03
7	1	2	2	75	5	2	18.11	1.75	3	19.77	16.28	0.72	0.30	0.54
7	1	2	2	75	5	2.5	14.71	1.98	3	16.45	12.56			
7	1	2	2	75	5	3	21.37	1.92	3	23.42	19.61			
7	1	2	2	75	5	4	17.73	1.88	3	19.19	15.60	0.71	0.29	0.97

Table 11. Example page from summary worksheet for calculating strain based equivalency factors.

swh/16jan91			SUMMARY STRAIN DATA - AASHO STANDARD LOAD BY INSTRUMENT											
												STRAIN	EQUIV.FAC	EQUIV.FAC
PVMT	INST	TRK	LOD	TP	SPD	WHL	STRAIN	SD	N	MAX	MIN	RATIO	CHRISTISON	SOUTHGATE
7	1	1	1	75	5	1	6.21E-04	2.00E-04	4	9.21E-04	5.06E-04	1.14	1.63	2.07
7	1	1	1	75	5	2	1.46E-03	4.54E-04	4	1.98E-03	1.05E-03	2.67	41.68	250.92
7	1	1	1	75	45	1	3.15E-04	8.13E-05	3	3.94E-04	2.31E-04	0.58	0.12	0.05
7	1	1	1	75	45	2	1.29E-03	3.66E-04	3	1.71E-03	1.06E-03	2.36	26.02	124.85
7	1	1	1	110	5	1	4.56E-04	8.29E-05	3	5.43E-04	3.78E-04	0.84	0.51	0.36
7	1	1	1	110	5	2	1.01E-03	1.59E-04	3	1.14E-03	8.35E-04	1.86	10.48	32.45
7	1	1	1	110	45	1	2.64E-04	3.77E-05	2	2.91E-04	2.37E-04	0.48	0.06	0.02
7	1	1	1	110	45	2	1.09E-03	9.01E-05	2	1.16E-03	1.03E-03	2.01	14.07	50.24
7	1	1	2	75	5	1	5.41E-04	4.28E-06	2	5.44E-04	5.38E-04	0.99	0.97	0.95
7	1	1	2	75	5	2	6.56E-04	6.10E-05	3	7.26E-04	6.12E-04	1.20	2.01	2.81
7	1	1	2	75	45	1	2.97E-04	7.32E-06	2	3.03E-04	2.92E-04	0.54	0.10	0.03
7	1	1	2	75	45	2	5.46E-04	6.93E-05	2	5.95E-04	4.97E-04	1.00	1.00	1.00
7	1	1	2	110	5	1	5.33E-04	4.86E-05	2	5.67E-04	4.99E-04	0.98	0.91	0.88
7	1	1	2	110	5	2	5.95E-04	7.29E-05	2	6.46E-04	5.43E-04	1.09	1.39	1.62
7	1	1	2	110	45	1	2.84E-04	1.57E-05	5	3.02E-04	2.61E-04	0.52	0.08	0.03
7	1	1	2	110	45	2	5.57E-04	4.54E-05	5	6.09E-04	5.02E-04	1.02	1.08	1.12
7	1	1	3	75	5	1	7.21E-04	7.04E-06	3	7.27E-04	7.13E-04	1.32	2.88	4.80
7	1	1	3	75	5	2	4.75E-04	1.94E-05	3	4.97E-04	4.63E-04	0.87	0.59	0.46
7	1	1	3	75	45	1	4.21E-04	4.16E-06	2	4.24E-04	4.18E-04	0.77	0.37	0.23
7	1	1	3	75	45	2	2.81E-04	1.37E-05	2	2.90E-04	2.71E-04	0.51	0.08	0.02
7	1	1	3	110	5	1	6.84E-04	3.33E-05	4	7.22E-04	6.41E-04	1.25	2.36	3.57
7	1	1	3	110	5	2	4.68E-04	3.10E-05	4	4.97E-04	4.25E-04	0.86	0.56	0.42
7	1	1	3	110	45	1	3.29E-04	3.61E-05	3	3.71E-04	3.05E-04	0.60	0.15	0.06
7	1	1	3	110	45	2	2.33E-04	3.40E-05	3	2.54E-04	1.94E-04	0.43	0.04	0.01
7	1	2	1	75	5	1	4.49E-04	7.82E-06	3	4.58E-04	4.44E-04	0.82	0.48	0.34
7	1	2	1	75	5	2	6.68E-04	1.27E-05	3	6.79E-04	6.54E-04	1.22	4.29	6.20
7	1	2	1	75	5	3	6.66E-04	3.79E-05	3	7.10E-04	6.41E-04			
7	1	2	1	75	5	4	5.73E-04	5.58E-05	3	6.11E-04	5.09E-04	1.05	6.74	13.95
7	1	2	1	75	5	5	8.57E-04	3.94E-05	3	8.84E-04	8.11E-04			
7	1	2	1	75	45	1	2.33E-04	#DIV/0!	1	2.33E-04	2.33E-04	0.43	0.04	0.01
7	1	2	1	75	45	2	5.67E-04	#DIV/0!	1	5.67E-04	5.67E-04	1.04	2.28	2.42
7	1	2	1	75	45	3	5.62E-04	#DIV/0!	1	5.62E-04	5.62E-04			
7	1	2	1	75	45	4	5.29E-04	#DIV/0!	1	5.29E-04	5.29E-04	0.97	3.16	4.21
7	1	2	1	75	45	5	6.77E-04	#DIV/0!	1	6.77E-04	6.77E-04			
7	1	2	1	110	5	1	3.88E-04	6.26E-05	6	4.57E-04	3.10E-04	0.71	0.27	0.15
7	1	2	1	110	5	2	6.72E-04	5.27E-05	7	7.36E-04	6.12E-04	1.23	4.53	6.71
7	1	2	1	110	5	3	6.81E-04	5.51E-05	7	7.55E-04	6.17E-04			
7	1	2	1	110	5	4	5.49E-04	3.17E-05	7	5.88E-04	4.92E-04	1.01	4.98	8.70
7	1	2	1	110	5	5	7.84E-04	4.23E-05	7	8.34E-04	7.01E-04			
7	1	2	1	110	45	1	2.61E-04	6.47E-05	3	3.16E-04	1.90E-04	0.48	0.06	0.02
7	1	2	1	110	45	2	5.34E-04	3.82E-05	3	5.68E-04	4.93E-04	0.98	1.73	1.61
7	1	2	1	110	45	3	5.16E-04	1.03E-05	3	5.26E-04	5.06E-04			
7	1	2	1	110	45	4	5.07E-04	2.91E-05	3	5.30E-04	4.74E-04	0.93	2.72	3.37
7	1	2	1	110	45	5	6.52E-04	1.25E-05	3	6.66E-04	6.44E-04			
7	1	2	2	75	5	1	3.61E-04	3.30E-05	3	3.96E-04	3.31E-04	0.66	0.21	0.10
7	1	2	2	75	5	2	4.15E-04	6.44E-06	3	4.21E-04	4.08E-04	0.76	0.90	0.62
7	1	2	2	75	5	3	4.65E-04	2.36E-05	3	4.84E-04	4.38E-04			
7	1	2	2	75	5	4	3.38E-04	2.92E-05	3	3.71E-04	3.18E-04	0.62	0.80	0.58
7	1	2	2	75	5	5	4.84E-04	4.53E-05	3	5.36E-04	4.49E-04			
7	1	2	2	75	45	1	2.49E-04	3.75E-05	4	2.84E-04	1.96E-04	0.46	0.05	0.01
7	1	2	2	75	45	2	3.20E-04	1.61E-05	4	3.34E-04	2.97E-04	0.59	0.32	0.13
7	1	2	2	75	45	3	3.52E-04	1.38E-05	4	3.65E-04	3.33E-04			
7	1	2	2	75	45	4	3.14E-04	1.05E-05	4	3.27E-04	3.04E-04	0.58	0.48	0.26
7	1	2	2	75	45	5	4.17E-04	1.68E-05	4	4.33E-04	4.01E-04			
7	1	2	2	110	5	1	4.32E-04	3.16E-05	5	4.69E-04	3.88E-04	0.79	0.41	0.27
7	1	2	2	110	5	2	4.90E-04	3.74E-05	5	5.47E-04	4.54E-04	0.90	1.52	1.34
7	1	2	2	110	5	3	5.24E-04	1.05E-05	5	5.31E-04	5.06E-04			

pavement structures, and with two sets of instruments in each pavement were recorded. To determine the standard load for any of these conditions it is apparent that different standards of load response would be required for the two different pavement types. It is also a convention since the AASHO Road Test that the standard axle load is an 18,000-lb (8,172-kg) load on a single axle with dual tires. However, the standard loading condition has not been defined relative to a tire pressure or speed. Therefore, an option exists to fix tire pressure and speed to be as close to the AASHO Road Test as possible in order to model the road test standard loading condition. Another option exists to vary the tire pressure and speed of the standard load to coincide with the tire pressure and speed of the load under consideration for calculating load equivalency factors. However, if variable tire pressure and speed are used, then the effects of tire pressure and speed on pavement response may be masked. This is because most of the methods of load equivalencies are calculated as ratios of pavement response and the effect of tire pressure or speed on the load in question may also be the same effect as on the standard load and thus those effects will cancel in the ratio of the load equivalency factor. A somewhat more logical choice for a standard load condition may be to use the same speed and tire pressure that was used at the AASHO Road Test. Therefore, the effects in changes of speed and tire pressure will affect the response of the load in question but will not affect the standard load. Therefore, it is reasonable to assume that the actual effects of speed and tire pressure will be quantified better when the standard loading conditions remain fixed.

In order to test these possibilities, load equivalencies were calculated using standards determined by several different methods, both with a fixed AASHO standard and with a variable standard due to tire pressure and speed. It was decided that for the best analysis a standard at fixed loading conditions including tire pressure and speed that simulated the AASHO Road Test would be the most useful for the purposes of this project. It was determined that equivalency factors calculated in this manner would be more likely to indicate whether speed or tire pressure affected the actual rate of damage of the pavement. Since pavement damage is the basis for load equivalency factors, it is logical to fix all loading conditions in determining the standard load by which all other damage is compared.

The standard load of 18,000-lb (8,172-kg) was the medium load in the factorial on the single unit truck. This was the first load run in the testing because the truck had previously been loaded to 18,000-lb (8,172-kg) on the single axle before testing had begun and this was the most cost effective way to proceed rather than changing the load for the first run. The values obtained in all of the various cells from both pavements and both sets of instruments within pavements at all levels of tire pressure and speed could be considered as estimators of the standard load response. Due to the decision taken to use the standard AASHO tire pressure and speeds, the standard load was taken at a tire pressure of 75-psi (515-kPa) and a speed of 45-mi/h (72-km/h) which were the values closest to the AASHO Road Test conditions. Therefore, the best estimators of the standard load condition are the response values in each of the cells of pavement types and instruments within pavements and the replicate values run in those cells. The method initially selected to calculate the standard was the average response of all values within a pavement section; therefore, the values from all replicates and both sets of instruments were averaged to obtain the standard load response for a pavement. In an effort to reduce variation between instruments, an analysis was performed with the standard taken within instruments as well. Therefore, it is possible that any consistent variations in instruments will cancel in the ratio of the equivalency factors if there are definite positive or negative trends in the instrument variation.

This method of using a single value for the standard response over a large factorial of responses from other loading conditions has some advantages and some disadvantages. One advantage is that the random variation in the standard load measurement will be removed as a source of error in the equivalency factor calculation. Only the source of variation from the load in question will affect the calculations. This is only useful if the estimate used for the standard load response is unbiased and can be accepted as a good estimator for the standard load response at all non-load related factor levels such as temperature or moisture. Another advantage, which was the main reason on this project, was the fact that a dedicated truck is not needed to provide the standard load. The project only had one single axle truck to apply the standard load available due to budget limitations. Two other loading conditions were required for the single unit truck at which time no standard loaded truck would be available to apply the standard load immediately after the load in question. Also, the single unit truck was not available to the project during the testing of the tridem or tandem axle trucks. Therefore, the method used saved considerable costs on the project by not requiring an additional truck to be available at all times and doubling the number of measurements and thus the amount of data collected and analyzed. Although a significant cost savings was obtained, the results of this project indicate that standard load measurements under other variable conditions such as temperature, moisture, time of day, and other environmental variables may be necessary to get more accurate LEF's in order to develop reliable models. A separate analysis was run in order to determine the effects of using a variable standard to vary with all environmental parameters. This separate analysis is described in the following section.

VARIABLE STANDARD ANALYSIS

An analysis was run to estimate the effects of varying the standard load response value used to calculate load equivalency factors. The ideal way to perform such an analysis is to run the experiment with a standard load vehicle available at all times to run immediately before all loads in the experiment to get an unbiased estimator of the standard load response at that instant in time. In order to simulate this effect, we took advantage of the fact that the load on the steering axle could not be varied by any significant amount. This load ranged almost invariably between 4,000- and 4,500-lb (1,816- and 2,043-kg). Therefore, the use of the steering axle response as the standard load for each pass of the vehicle acts as a useful surrogate standard load condition to determine the effects of varying moisture, temperature, tire pressure, speed, and all other environmental and loading factors associated with each run of the vehicles.

This analysis was run on the deflection data only. The analysis showed that temperature was not significant in either one of the deflection based methods of load equivalency factors. This supports the theory that the effect of temperature and probably other environmental factors such as moisture cancel out in the ratio of the load equivalency factor calculations. This supports the conclusion that estimate of a standard load response should be taken at the same time the response for all other loading conditions are measured.

CALCULATION OF EQUIVALENCY FACTORS

Although there are many possible ways to calculate load equivalency factors as described in the background section of this report, a finite number were selected for analysis on this project. Part of this selection process is described in section 3 "Candidate Primary Response Methods" of this report. The final analyses were performed using two strain based methods of primary response load equivalency factors and two deflection based methods.

Deflection Based Equivalency Factor Methods

The two deflection based methods chosen were those proposed by Christison and Hutchinson.[2,10] The details of how these two methods work and how they differ are described in appendix A of this report. Equivalency factors for single axle loads for both methods are predicted using the expression:

$$LEF = (D/D_b)^{3.8} \tag{8}$$

where:

D/D_b = the ratio of pavement surface deflections caused by a single axle load to those recorded under the standard 18,000-lb (8,160-kg) single (D_b) axle-dual tire load of the Benkelman Beam vehicle, and

3.8 = the slope of the deflection - anticipated traffic loading relationship.

Equivalency factors for tandem and tridem axle configurations for the Christison method are predicted using the expression:

$$F = (D_i/D_b)^{3.8} + \sum_{i=1}^{n-1} (D_i/D_b)^{3.8} \tag{9}$$

where:

D_i/D_b = the ratio of maximum surface deflections under the leading axle of the axle group to those caused by the standard 18,000-lb (8160-kg) load,

D_i/D_b = the ratio of the difference in magnitude between the maximum deflection recorded under each succeeding axle and the minimum residual deflection preceding the axle to deflections caused by the standard load, and n = the number of axles in the axle group.

The basic difference in the two methods is in how the peaks are selected for consideration in the equation. The differences are illustrated in figure 31. In the Christison method, the primary effect in the equivalency factor calculation is the first deflection peak of a tandem, tridem, or any multiaxle group. In the Hutchinson method, the same expression as defined above is used except the primary factor is the largest peak in the multiaxle group. Therefore, the Hutchinson method of calculating equivalency factor will tend to produce higher load equivalency factors. It is worth noting that the methods are exactly the same for single-axle trucks.

Strain Based Equivalency Factor Methods

Two methods were selected to calculate load equivalency factors using the strain primary responses. These methods are the ones proposed by Christison and Southgate.[2,7,8] The Christison method, uses approximately the same procedures as in the deflection based method described above and in appendix A. Equivalency factor predictions from the tensile strain measurements were predicted using the expression:

$$F = \sum_{i=1}^{n} (S_i/S_b)^{3.8} \tag{10}$$

where:

S_i/S_b = the ratio of longitudinal interfacial tensile strains recorded under each axle to those recorded under the standard load (S_b),

n = the number of axles in the axle group, and

3.8 = the slope of the fatigue life-tensile strain relationship.

As described in appendix A, the Southgate method is based on the work strain concept. In deriving this equivalency factor method, only mechanistic models were used to estimate load equivalencies. Using the strain energy, W, of a body, an expression for the "work strain", S_w, was found to be:

$$S_w = (2W/E)^{0.5} \tag{11}$$

where E is Young's modulus of elasticity.

To apply conventional concepts of load equivalency factors, work strain was related to tensile strain at the bottom of the asphalt concrete, S_a, through regression, (equation 12). The expression used for the load equivalency calculations related the number of standard axle load repetitions, N, to work strain, (equation 13) as follows:

$$\log(S_a) = 1.1483\log(S_w) - 0.1638 \tag{12}$$

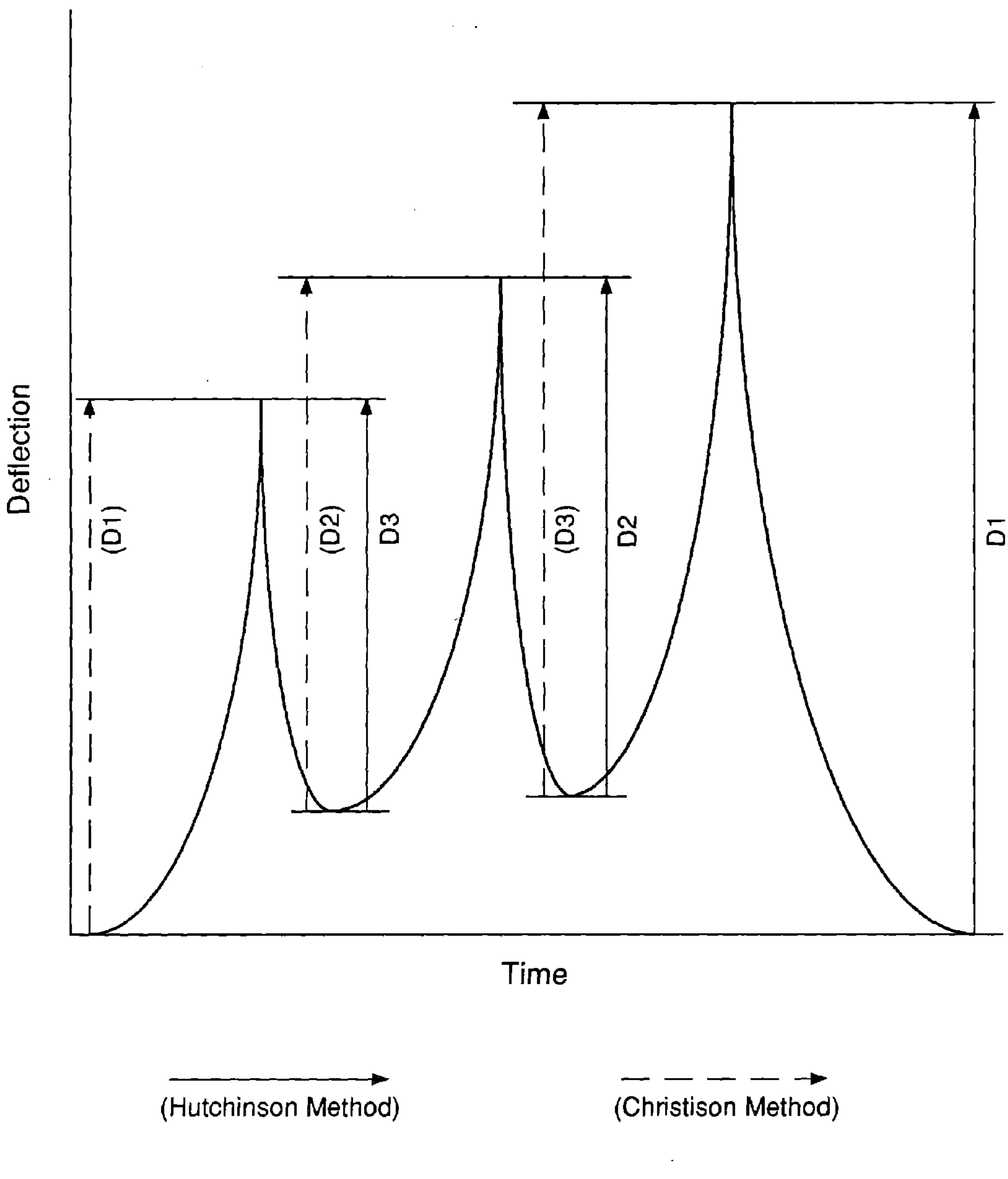


Figure 31. Comparison of Hutchinson and Christison methods of accounting for the peaks in pavement surface deflection under a tridem axle.

$$\log(N) = -6.4636\log(S_w) - 17.3081 \quad (13)$$

The load equivalency factor $LEF_i = N_{18}/N_L$ (14)

where:

N_{18} = repetitions calculated by equation 13 in which the work strain is that due to an 18-kip (8,160-kg) four-tired single axle load, and

N_L = repetitions calculated by equation 13 in which the work strain is that due to the total load on the axle or group of axles.

Summary worksheets which show the equivalency factor values for all of the vehicle runs on all of their wheels are available from FHWA. This includes equivalence factors calculated from both strain and deflection data using the method of obtaining a standard response as described in the sections above. Tables 10 and 11 provide examples of these summary worksheets on which the equivalency factor values were calculated for the deflection data and strain data respectively.

ANALYSIS OF VARIANCE

The worksheets which contain the summary strain and deflection data were used as a basis to select the equivalency factor for the single-axle, the middle tandem axle on the tandem vehicle, and the tridem axle on the tridem vehicle. These were the equivalency factor values for the three axle types used in the analysis. Figure 32 shows a diagram of the three truck configurations used indicating which axle was used to calculate the equivalency factor on that vehicle. It should be noted that the equivalency factors calculated were for the individual axles and not for the entire vehicles. Since the pavement response returned essentially to zero after each pass of an axle group, vehicle equivalencies can be calculated by merely adding up the individual axle group equivalencies.

The data was transformed directly into an ASCII file which can be used in an analysis of variance by the SPSS microcomputer statistical analysis package. The final files used are shown in tables 12 and 13. The ASCII file contains the various levels of pavement (PVMT), instrument (INST), truck (TRK), load (LOD), tire pressure (TP), speed (SPD), and pavement temperature (TEMP). The dependent variables input into the analysis also include the two equivalency factors calculated from the two methods. The levels of the independent factors were transformed into single digit levels as follows:

PAVEMENT	TIRE PRESSURE	SPEED
7-in (178-mm) = 1	75-psi (515-kPa) = 1	5-mi/h (8-km/h) = 1
3½-in (89-mm) = 2	110-psi (760-kPa) = 2	45-mi/h (72-km/h) = 2

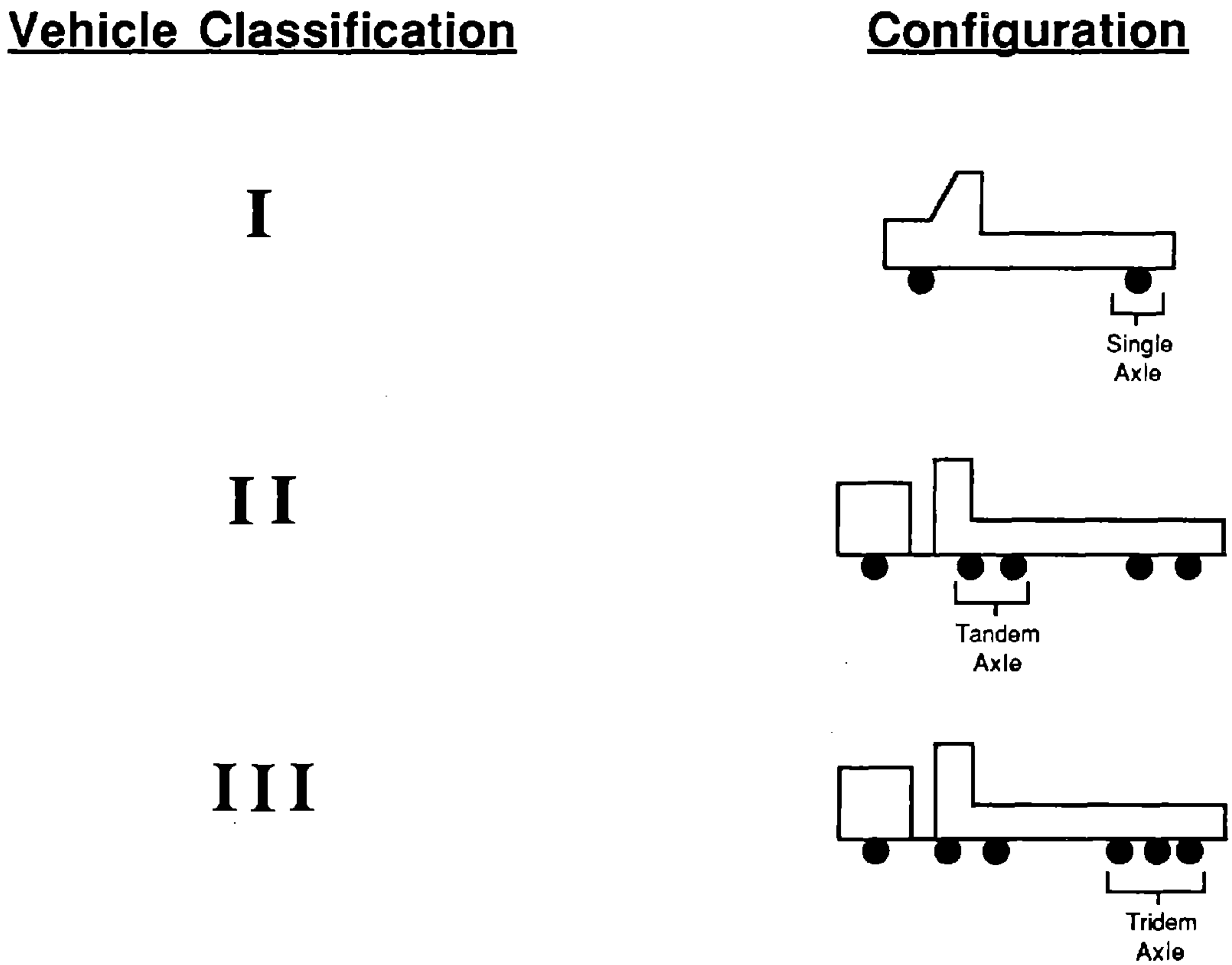


Figure 32. Definition of vehicle classifications for the primary experiment.

Table 12. ASCII file used as input to SPSS for deflection method analysis.

PVMT	INST	TRK	LOD	TP	SPD	TEMP	CHRISTISON	HUTCHINSON
1	1	1	1	1	1	088	0006.04	0006.04
1	1	1	1	1	2	088	0003.88	0003.88
1	1	1	1	2	1	087	0004.66	0004.66
1	1	1	1	2	2	087	0005.32	0005.32
1	1	1	2	1	1	082	0000.78	0000.78
1	1	1	2	1	2	082	0001.00	0001.00
1	1	1	2	2	1	082	0000.93	0000.93
1	1	1	2	2	2	082	0000.80	0000.80
1	1	1	3	1	1	082	0000.13	0000.13
1	1	1	3	1	2	082	0000.02	0000.02
1	1	1	3	2	1	081	0000.11	0000.11
1	1	1	3	2	2	081	0000.02	0000.02
1	1	2	1	1	1	077	0002.18	0003.04
1	1	2	1	1	2	077	0001.51	0002.41
1	1	2	1	2	1	072	0001.87	0003.05
1	1	2	1	2	2	072	0001.28	0001.75
1	1	2	2	1	1	074	0000.30	0000.54
1	1	2	2	1	2	074	0000.22	0000.46
1	1	2	2	2	1	072	0000.52	0000.94
1	1	2	2	2	2	072	0000.21	0000.43
1	1	2	3	1	1	074	0000.05	0000.07
1	1	2	3	1	2	074	0000.04	0000.08
1	1	2	3	2	1	074	0000.04	0000.06
1	1	2	3	2	2	074	0000.05	0000.08
1	1	3	1	1	1	085	0004.48	0007.45
1	1	3	1	1	2	085	0001.65	0003.44
1	1	3	1	2	1	085	0002.58	0005.01
1	1	3	1	2	2	085	0001.73	0003.79
1	1	3	2	1	1	095	0002.32	0005.27
1	1	3	2	1	2	095	0001.00	0002.21
1	1	3	2	2	1	092	0002.36	0004.34
1	1	3	2	2	2	092	0000.76	0001.74
1	1	3	3	1	1	095	0000.62	0000.71
1	1	3	3	1	2	095	0000.40	0000.65
1	1	3	3	2	1	100	0000.58	0000.76
1	1	3	3	2	2	100	0000.27	0000.61
1	2	1	1	1	1	088	0002.84	0002.84
1	2	1	1	1	2	088	0004.52	0004.52
1	2	1	1	2	1	087	0002.16	0002.16
1	2	1	1	2	2	087	0008.26	0008.26
1	2	1	2	1	1	082	0000.51	0000.51
1	2	1	2	1	2	082	0001.00	0001.00
1	2	1	2	2	1	082	0000.55	0000.55
1	2	1	2	2	2	082	0001.09	0001.09
1	2	1	3	1	1	082	0000.15	0000.15
1	2	1	3	1	2	082	0000.12	0000.12
1	2	1	3	2	1	081	0000.12	0000.12
1	2	1	3	2	2	081	0000.09	0000.09
1	2	2	1	1	1	077	0001.58	0002.11
1	2	2	1	1	2	077	0000.61	0001.51
1	2	2	1	2	1	072	0001.21	0001.89
1	2	2	1	2	2	072	0000.61	0001.37
1	2	2	2	1	1	074	0000.28	0000.48
1	2	2	2	1	2	074	0000.21	0000.44
1	2	2	2	2	1	072	0000.43	0000.70
1	2	2	2	2	2	072	0000.20	0000.40
1	2	2	3	1	1	074	0000.06	0000.07

Table 12. ASCII file used as input to SPSS for deflection method analysis.

PVMT	INST	TRK	LOD	TP	SPD	TEMP	CHRISTISON	HUTCHINSON
1	2	2	3	1	2	074	0000.06	0000.09
1	2	2	3	2	1	074	0000.05	0000.07
1	2	2	3	2	2	074	0000.04	0000.07
1	2	3	1	1	1	085	0002.47	0004.01
1	2	3	1	1	2	085	0001.41	0002.57
1	2	3	1	2	1	085	0001.25	0002.18
1	2	3	1	2	2	085	0001.32	0002.22
1	2	3	2	1	1	095	0001.82	0002.95
1	2	3	2	1	2	095	0001.23	0001.73
1	2	3	2	2	1	092	0001.47	0002.95
1	2	3	2	2	2	092	0001.27	0002.03
1	2	3	3	1	1	095	0000.49	0000.65
1	2	3	3	1	2	095	0000.36	0000.45
1	2	3	3	2	1	100	0000.44	0000.62
1	2	3	3	2	2	100	0000.44	0000.51
2	1	1	1	1	1	090	0006.31	0006.31
2	1	1	1	1	2	090	0010.29	0010.29
2	1	1	1	2	1	092	0003.26	0003.26
2	1	1	1	2	2	092	0007.74	0007.74
2	1	1	2	1	1	082	0001.40	0001.40
2	1	1	2	1	2	082	0001.00	0001.00
2	1	1	2	2	1	082	0001.53	0001.53
2	1	1	2	2	2	082	0000.83	0000.83
2	1	1	3	1	1	087	0000.16	0000.16
2	1	1	3	1	2	087	0000.03	0000.03
2	1	1	3	2	1	083	0000.12	0000.12
2	1	1	3	2	2	083	0000.02	0000.02
2	1	2	1	1	1	077	0001.72	0002.65
2	1	2	1	1	2	077	0001.74	0002.45
2	1	2	1	2	1	074	0001.19	0001.91
2	1	2	1	2	2	074	0001.52	0001.92
2	1	2	2	1	1	074	0000.40	0000.71
2	1	2	2	1	2	074	0000.34	0000.41
2	1	2	2	2	1	070	0000.40	0000.63
2	1	2	2	2	2	070	0000.33	0000.43
2	1	2	3	1	1	074	0000.08	0000.10
2	1	2	3	1	2	074	0000.07	0000.07
2	1	2	3	2	1	074	0000.09	0000.12
2	1	2	3	2	2	074	0000.07	0000.07
2	1	3	1	1	1	088	0005.57	0010.99
2	1	3	1	1	2	088	0002.66	0004.53
2	1	3	1	2	1	088	0001.62	0003.45
2	1	3	1	2	2	088	0001.91	0003.87
2	1	3	2	1	1	103	0002.56	0003.72
2	1	3	2	1	2	103	0001.63	0002.42
2	1	3	2	2	1	101	0002.90	0005.00
2	1	3	2	2	2	101	0001.80	0002.79
2	1	3	3	1	1	103	0000.45	0000.66
2	1	3	3	1	2	103	0000.33	0000.57
2	1	3	3	2	1	107	0000.45	0000.83
2	1	3	3	2	2	107	0000.30	0000.41
2	2	1	1	1	1	090	0012.47	0012.47
2	2	1	1	1	2	090	0014.37	0014.37
2	2	1	1	2	1	092	0012.33	0012.33
2	2	1	1	2	2	092	0009.87	0009.87
2	2	1	2	1	1	082	0001.39	0001.39
2	2	1	2	1	2	082	0001.00	0001.00

Table 12. ASCII file used as input to SPSS for deflection method analysis.

PVMT	INST	TRK	LOD	TP	SPD	TEMP	CHRISTISON	HUTCHINSON
2	2	1	2	2	1	082	0001.76	0001.76
2	2	1	2	2	2	082	0001.17	0001.17
2	2	1	3	1	1	087	0000.11	0000.11
2	2	1	3	1	2	087	0000.03	0000.03
2	2	1	3	2	1	083	0000.13	0000.13
2	2	1	3	2	2	083	0000.02	0000.02
2	2	2	1	1	1	077	0001.93	0002.30
2	2	2	1	1	2	077	0001.34	0002.00
2	2	2	1	2	1	074	0000.93	0001.13
2	2	2	1	2	2	074	0001.09	0001.68
2	2	2	2	1	1	074	0000.49	0000.73
2	2	2	2	1	2	074	0000.26	0000.37
2	2	2	2	2	1	070	0000.44	0000.59
2	2	2	2	2	2	070	0000.30	0000.47
2	2	2	3	1	1	074	0000.07	0000.08
2	2	2	3	1	2	074	0000.03	0000.05
2	2	2	3	2	1	074	0000.08	0000.10
2	2	2	3	2	2	074	0000.04	0000.05
2	2	3	1	1	1	088	0004.78	0006.44
2	2	3	1	1	2	088	0002.91	0003.78
2	2	3	1	2	1	088	0001.35	0001.94
2	2	3	1	2	2	088	0001.85	0002.57
2	2	3	2	1	1	103	0002.15	0003.17
2	2	3	2	1	2	103	0001.44	0001.75
2	2	3	2	2	1	101	0002.05	0003.12
2	2	3	2	2	2	101	0001.44	0001.92
2	2	3	3	1	1	103	0000.41	0000.53
2	2	3	3	1	2	103	0000.34	0000.40
2	2	3	3	2	1	107	0000.39	0000.53
2	2	3	3	2	2	107	0000.11	0000.13

Table 13. ASCII file used as input to SPSS for strain method analysis.

PVMT	INST	TRK	LOD	TP	SPD	TEMP	CHRISTISON	SOUTHGATE
1	1	1	1	1	1	088	0041.68	0250.92
1	1	1	1	1	2	088	0026.02	0124.85
1	1	1	1	2	1	087	0010.48	0032.45
1	1	1	1	2	2	087	0014.07	0050.24
1	1	1	2	1	1	082	0002.01	0002.81
1	1	1	2	1	2	082	0001.00	0001.00
1	1	1	2	2	1	082	0001.39	0001.62
1	1	1	2	2	2	082	0001.08	0001.12
1	1	1	3	1	1	082	0000.59	0000.46
1	1	1	3	1	2	082	0000.08	0000.02
1	1	1	3	2	1	081	0000.56	0000.42
1	1	1	3	2	2	081	0000.04	0000.01
1	1	2	1	1	1	077	0004.29	0006.20
1	1	2	1	1	2	077	0002.28	0002.42
1	1	2	1	2	1	072	0004.53	0006.71
1	1	2	1	2	2	072	0001.73	0001.61
1	1	2	2	1	1	074	0000.90	0000.62
1	1	2	2	1	2	074	0000.32	0000.13
1	1	2	2	2	1	072	0001.52	0001.34
1	1	2	2	2	2	072	0000.24	0000.09
1	1	2	3	1	1	074	0000.23	0000.08
1	1	2	3	1	2	074	0000.04	0000.01
1	1	2	3	2	1	074	0000.25	0000.09
1	1	2	3	2	2	074	0000.04	0000.01
1	1	3	1	1	1	085	0024.47	0068.43
1	1	3	1	1	2	085	0009.37	0016.62
1	1	3	1	2	1	085	0010.10	0018.48
1	1	3	1	2	2	085	0008.11	0013.60
1	1	3	2	1	1	095	0034.68	0115.83
1	1	3	2	1	2	095	0013.71	0028.82
1	1	3	2	2	1	092	0029.22	0091.59
1	1	3	2	2	2	092	0012.97	0026.87
1	1	3	3	1	1	095	0008.35	0013.75
1	1	3	3	1	2	095	0002.61	0002.49
1	1	3	3	2	1	100	0009.64	0018.09
1	1	3	3	2	2	100	0004.79	0006.03
1	2	1	1	1	1	088	0210.18	2756.61
1	2	1	1	1	2	088	0059.74	0427.64
1	2	1	1	2	1	087	0080.05	0659.74
1	2	1	1	2	2	087	0008.66	0024.48
1	2	1	2	1	1	082	0008.43	0023.50
1	2	1	2	1	2	082	0001.00	0001.00
1	2	1	2	2	1	082	0006.58	0016.30
1	2	1	2	2	2	082	0001.64	0002.08
1	2	1	3	1	1	082	0000.79	0000.70
1	2	1	3	1	2	082	0000.05	0000.01
1	2	1	3	2	1	081	0000.44	0000.30
1	2	1	3	2	2	081	0000.05	0000.01
1	2	2	1	1	1	077	0001.75	0001.70
1	2	2	1	1	2	077	0000.97	0000.69
1	2	2	1	2	1	072	0003.28	0004.22
1	2	2	1	2	2	072	0000.89	0000.61
1	2	2	2	1	1	074	0000.32	0000.13
1	2	2	2	1	2	074	0000.21	0000.07
1	2	2	2	2	1	072	0000.86	0000.57
1	2	2	2	2	2	072	0000.19	0000.06
1	2	2	3	1	1	074	0000.10	0000.02

Table 13. ASCII file used as input to SPSS for strain method analysis.

PVMT	INST	TRK	LOD	TP	SPD	TEMP	CHRISTISON	SOUTHGATE
1	2	2	3	1	2	074	0000.03	0000.00
1	2	2	3	2	1	074	0000.11	0000.03
1	2	2	3	2	2	074	0000.03	0000.00
1	2	3	1	1	1	085	0005.77	0008.03
1	2	3	1	1	2	085	0003.03	0003.18
1	2	3	1	2	1	085	0002.29	0002.06
1	2	3	1	2	2	085	0001.99	0001.73
1	2	3	2	1	1	095	0007.48	0011.78
1	2	3	2	1	2	095	0004.54	0005.68
1	2	3	2	2	1	092	0009.30	0016.33
1	2	3	2	2	2	092	0004.65	0005.78
1	2	3	3	1	1	095	0002.14	0001.91
1	2	3	3	1	2	095	0000.95	0000.55
1	2	3	3	2	1	100	0003.22	0003.35
1	2	3	3	2	2	100	0002.07	0001.77
2	1	1	1	1	1	090	0013.88	0049.23
2	1	1	1	1	2	090	0012.81	0043.73
2	1	1	1	2	1	092	0014.57	0052.90
2	1	1	1	2	2	092	0012.82	0043.74
2	1	1	2	1	1	082	0000.84	0000.77
2	1	1	2	1	2	082	0000.04	0000.01
2	1	1	2	2	1	082	0005.06	0011.05
2	1	1	2	2	2	082	0000.68	0000.57
2	1	1	3	1	1	087	0001.28	0001.45
2	1	1	3	1	2	087	0000.07	0000.02
2	1	1	3	2	1	083	0001.06	0001.09
2	1	1	3	2	2	083	0000.03	0000.00
2	1	2	1	1	1	077	0002.22	0002.34
2	1	2	1	1	2	077	0000.09	0000.02
2	1	2	1	2	1	074	0003.57	0004.72
2	1	2	1	2	2	074	0000.70	0000.43
2	1	2	2	1	1	074	0001.29	0001.05
2	1	2	2	1	2	074	0000.18	0000.06
2	1	2	2	2	1	070	0001.55	0001.38
2	1	2	2	2	2	070	0000.42	0000.20
2	1	2	3	1	1	074	0000.35	0000.15
2	1	2	3	1	2	074	0000.08	0000.02
2	1	2	3	2	1	074	0000.57	0000.32
2	1	2	3	2	2	074	0000.07	0000.01
2	1	3	1	1	1	088	0001.12	0000.73
2	1	3	1	1	2	088	0000.71	0000.37
2	1	3	1	2	1	088	0004.80	0006.13
2	1	3	1	2	2	088	0000.30	0000.11
2	1	3	2	1	1	103	0000.88	0000.51
2	1	3	2	1	2	103	0000.46	0000.20
2	1	3	2	2	1	101	0001.04	0000.67
2	1	3	2	2	2	101	0000.71	0000.37
2	1	3	3	1	1	103	0004.73	0006.27
2	1	3	3	1	2	103	0002.40	0002.38
2	1	3	3	2	1	107	0000.30	0000.10
2	1	3	3	2	2	107	0003.21	0003.57
2	2	1	1	1	1	090	#DIV/0!	#DIV/0!
2	2	1	1	1	2	090	#DIV/0!	#DIV/0!
2	2	1	1	2	1	092	#DIV/0!	#DIV/0!
2	2	1	1	2	2	092	#DIV/0!	#DIV/0!
2	2	1	2	1	1	082	#DIV/0!	#DIV/0!
2	2	1	2	1	2	082	#DIV/0!	#DIV/0!

Table 13. ASCII file used as input to SPSS for strain method analysis.

PVMT	INST	TRK	LOD	TP	SPD	TEMP	CHRISTISON	SOUTHGATE
2	2	1	2	2	1	082	0004.39	0008.96
2	2	1	2	2	2	082	#DIV/0!	#DIV/0!
2	2	1	3	1	1	087	0001.26	0001.41
2	2	1	3	1	2	087	0000.02	0000.00
2	2	1	3	2	1	083	0000.70	0000.58
2	2	1	3	2	2	083	0000.05	0000.01
2	2	2	1	1	1	077	0005.07	0007.97
2	2	2	1	1	2	077	0003.42	0004.44
2	2	2	1	2	1	074	0002.72	0003.17
2	2	2	1	2	2	074	0002.65	0003.05
2	2	2	2	1	1	074	0001.07	0000.79
2	2	2	2	1	2	074	0000.60	0000.34
2	2	2	2	2	1	070	#DIV/0!	#DIV/0!
2	2	2	2	2	2	070	0000.56	0000.31
2	2	2	3	1	1	074	0000.36	0000.16
2	2	2	3	1	2	074	0000.20	0000.07
2	2	2	3	2	1	074	0000.37	0000.17
2	2	2	3	2	2	074	0000.18	0000.06
2	2	3	1	1	1	088	0011.96	0023.31
2	2	3	1	1	2	088	0016.68	0038.11
2	2	3	1	2	1	088	0018.76	0045.33
2	2	3	1	2	2	088	0014.92	0032.43
2	2	3	2	1	1	103	0016.69	0039.53
2	2	3	2	1	2	103	0011.11	0021.18
2	2	3	2	2	1	101	0019.71	0048.85
2	2	3	2	2	2	101	0022.41	0059.05
2	2	3	3	1	1	103	0007.98	0012.78
2	2	3	3	1	2	103	0003.08	0003.18
2	2	3	3	2	1	107	0015.72	0034.90
2	2	3	3	2	2	107	0009.80	0017.36

The ASCII files were used as input to the SPSS computer software to produce an initial analysis of variance. Since the SPSS software cannot handle a nested factorial as we are using in this experiment with instruments nested within pavements, an additional analysis was performed on the output from the SPSS results. This additional analysis used the mean square error of the factor instruments and all interactions with instruments as the basis for testing the other factors and interactions in the experiment. This procedure considers the variation of instruments to determine if the data actually shows significance of the other factors or whether the large instrument variation masks the variation of the other factors. Tables 14 through 17 show the initial analysis of variance results from SPSS. The corrected analyses of variance accounting for the random variable instruments for each equivalency factor method are presented in the next section "Discussion of Results." The results of these runs can be interpreted by examining the corrected sheets in the "Sig of F" (significance of F-test) column. In this column the significance of variations are indicated with numbers. The lower the number indicates a higher probability that that factor or interaction can be considered significant. Thus, the lower the number, the more significant the factor could be considered to be. Note that these initial results are misleading because the variation of instruments is not accounted for. The corrected ANOVA's presented in the next section indicate the proper results.

Additional analyses were performed to calculate the cell means for each of the factors and plots were made to show the trends of each of the significant factors. The most significant of these plots, as well as a detailed interpretation of the results of the statistical analyses are presented in the next section.

SUMMARY OF DATA ANALYSIS

This section has presented the summary of the methods used to analyze the data from this project. It covers the collection of the raw data in voltage and the conversion of the data to the base strain and deflection measurements which are the primary pavement responses. These primary responses are converted to load equivalency factors by several methods - two for strain and two for deflection. The variation of the equivalency factors based on the levels of the parameters in the study was analyzed using analysis of various techniques. The interpretation of the results of these analyses and the intermediate steps which are interesting are described in the next section.

Table 14. Primary ANOVA output from SPSS for the Christison deflection method.

Southgate Strain Equivalency Factor Method

Full Factorial - AASHTO Standard - By Instrument

ORIGINAL ANOVA

Source of Variation	SSquares	DF	MSquare	F	Sig.
Covariates	38,143	1	38,143	1.289	0.261
TEMP	38,143	1	38,143	1.289	0.261
Main Effects	1,227,799	8	153,475	5.185	0.000
PVMT	158,702	1	158,702	5.361	0.024
INST	125,936	1	125,936	4.254	0.044
AXLE	518,177	2	259,089	8.752	0.000
LOAD	496,827	2	248,414	8.392	0.001
TP	37,531	1	37,531	1.268	0.265
SPD	82,368	1	82,368	2.783	0.101
2-way Interactions	2,331,595	26	89,677	3.029	0.000
PVMT INST	1,675	1	1,675	0.057	0.813
PVMT AXLE	56,685	2	28,343	0.957	0.390
PVMT LOAD	63,694	2	31,847	1.076	0.348
PVMT TP	24,257	1	24,257	0.819	0.369
PVMT SPD	43,336	1	43,336	1.464	0.231
INST AXLE	354,455	2	177,228	5.987	0.004
INST LOAD	331,815	2	165,908	5.605	0.006
INST TP	61,261	1	61,261	2.069	0.156
INST SPD	73,290	1	73,290	2.476	0.121
AXLE LOAD	702,859	4	175,715	5.936	0.000
AXLE TP	174,048	2	87,024	2.940	0.061
AXLE SPD	170,611	2	85,305	2.882	0.064
LOAD TP	160,021	2	80,010	2.703	0.076
LOAD SPD	158,168	2	79,084	2.672	0.078
TP SPD	29,095	1	29,095	0.983	0.326
3-way Interactions	2,874,512	44	65,330	2.207	0.003
PVMT INST AXLE	5,871	2	2,936	0.099	0.906
PVMT INST LOAD	872	2	436	0.015	0.985
PVMT INST TP	4,397	1	4,397	0.149	0.701
PVMT INST SPD	4,177	1	4,177	0.141	0.709
PVMT AXLE LOAD	23,149	4	5,787	0.195	0.940
PVMT AXLE TP	20,921	2	10,460	0.353	0.704
PVMT AXLE SPD	12,905	2	6,453	0.218	0.805
PVMT LOAD TP	6,179	2	3,090	0.104	0.901
PVMT LOAD SPD	29,725	2	14,863	0.502	0.608
PVMT TP SPD	12,473	1	12,473	0.421	0.519
INST AXLE LOAD	636,370	4	159,093	5.374	0.001
INST AXLE TP	133,293	2	66,646	2.251	0.115
INST AXLE SPD	243,417	2	121,709	4.111	0.022
INST LOAD TP	131,751	2	65,875	2.225	0.117
INST LOAD SPD	224,953	2	112,476	3.800	0.028
INST TP SPD	25,898	1	25,898	0.875	0.354
AXLE LOAD TP	324,438	4	81,110	2.740	0.037
AXLE LOAD SPD	454,391	4	113,598	3.837	0.008
AXLE TP SPD	63,040	2	31,520	1.065	0.352
LOAD TP SPD	54,988	2	27,494	0.929	0.401
Explained	6,472,049	79	81,925	2.768	0.000
Residual	1,657,729	56	29,602		
Total	8,129,777	135	60,221		

Table 15. Primary ANOVA output from SPSS for the Hutchinson deflection method.

Christison Strain Equivalency Factor Method
Full Factorial - AASHTO Standard - By Instrument

ORIGINAL ANOVA

Source of Variation			SSquares	DF	MSquare	F	Sig.
Covariates			1,903	1	1,903	14.755	0.000
TEMP			1,903	1	1,903	14.755	0.000
Main Effects			12,404	8	1,551	12.021	0.000
PVMT			1,595	1	1,595	12.364	0.001
INST			965	1	965	7.480	0.008
AXLE			3,630	2	1,815	14.070	0.000
LOAD			6,769	2	3,384	26.237	0.000
TP			163	1	163	1.267	0.265
SPD			999	1	999	7.748	0.007
2-way Interactions			18,016	26	693	5.372	0.000
PVMT	INST		295	1	295	2.286	0.136
PVMT	AXLE		334	2	167	1.295	0.282
PVMT	LOAD		426	2	213	1.650	0.201
PVMT	TP		269	1	269	2.085	0.154
PVMT	SPD		385	1	385	2.986	0.090
INST	AXLE		2,523	2	1,262	9.780	0.000
INST	LOAD		2,295	2	1,147	8.895	0.000
INST	TP		192	1	192	1.488	0.228
INST	SPD		292	1	292	2.267	0.138
AXLE	LOAD		7,615	4	1,904	14.758	0.000
AXLE	TP		1,169	2	584	4.530	0.015
AXLE	SPD		842	2	421	3.263	0.046
LOAD	TP		1,204	2	602	4.667	0.013
LOAD	SPD		883	2	441	3.422	0.040
TP	SPD		109	1	109	0.843	0.363
3-way Interactions			17,525	44	398	3.088	0.000
PVMT	INST	AXLE	793	2	396	3.074	0.054
PVMT	INST	LOAD	78	2	39	0.304	0.739
PVMT	INST	TP	13	1	13	0.102	0.751
PVMT	INST	SPD	7	1	7	0.054	0.817
PVMT	AXLE	LOAD	260	4	65	0.504	0.733
PVMT	AXLE	TP	166	2	83	0.643	0.529
PVMT	AXLE	SPD	81	2	41	0.316	0.731
PVMT	LOAD	TP	44	2	22	0.171	0.843
PVMT	LOAD	SPD	166	2	83	0.644	0.529
PVMT	TP	SPD	53	1	53	0.413	0.523
INST	AXLE	LOAD	3,826	4	956	7.415	0.000
INST	AXLE	TP	723	2	361	2.802	0.069
INST	AXLE	SPD	1,504	2	752	5.829	0.005
INST	LOAD	TP	601	2	300	2.329	0.107
INST	LOAD	SPD	1,128	2	564	4.373	0.017
INST	TP	SPD	37	1	37	0.290	0.592
AXLE	LOAD	TP	1,887	4	472	3.658	0.010
AXLE	LOAD	SPD	2,505	4	626	4.854	0.002
AXLE	TP	SPD	166	2	83	0.645	0.528
LOAD	TP	SPD	132	2	66	0.513	0.601
Explained			49,849	79	631	4.892	0.000
Residual			7,223	56	129		
Total			57,073	135	423		

Table 16. Primary ANOVA output from SPSS for the Christison strain method.

Hutchinson Deflection Equivalency Factor Method
Full Factorial - AASHTO Standard - By Instrument

ORIGINAL ANOVA

Source of Variation	SSquares	DF	MSquare	F	Sig.
Covariates	72.378	1	72.378	78.433	0.000
TEMP	72.378	1	72.378	78.433	0.000
Main Effects	544.333	8	68.042	73.734	0.000
PVMT	4.183	1	4.183	4.533	0.037
INST	1.003	1	1.003	1.087	0.301
AXLE	32.693	2	16.347	17.714	0.000
LOAD	488.098	2	244.049	264.466	0.000
TP	3.267	1	3.267	3.541	0.064
SPD	3.566	1	3.566	3.864	0.054
2-way Interactions	246.95	26	9.498	10.293	0.000
PVMT INST	4.76	1	4.760	5.158	0.027
PVMT AXLE	22.284	2	11.142	12.074	0.000
PVMT LOAD	26.336	2	13.168	14.270	0.000
PVMT TP	3.453	1	3.453	3.742	0.057
PVMT SPD	0.005	1	0.005	0.005	0.944
INST AXLE	21.885	2	10.943	11.858	0.000
INST LOAD	0.529	2	0.264	0.287	0.752
INST TP	0.334	1	0.334	0.362	0.549
INST SPD	1.592	1	1.592	1.725	0.194
AXLE LOAD	119.272	4	29.818	32.312	0.000
AXLE TP	2.46	2	1.230	1.333	0.271
AXLE SPD	19.221	2	9.611	10.415	0.000
LOAD TP	12.094	2	6.047	6.553	0.003
LOAD SPD	2.686	2	1.343	1.455	0.241
TP SPD	1.909	1	1.909	2.069	0.155
3-way Interactions	118.178	44	2.686	2.911	0.000
PVMT INST AXLE	7.549	2	3.774	4.090	0.021
PVMT INST LOAD	9.051	2	4.525	4.904	0.010
PVMT INST TP	0.054	1	0.054	0.058	0.810
PVMT INST SPD	3.416	1	3.416	3.702	0.059
PVMT AXLE LOAD	31.715	4	7.929	8.592	0.000
PVMT AXLE TP	1.168	2	0.584	0.633	0.534
PVMT AXLE SPD	0.229	2	0.115	0.124	0.883
PVMT LOAD TP	6.05	2	3.025	3.278	0.044
PVMT LOAD SPD	0.289	2	0.145	0.157	0.855
PVMT TP SPD	0.161	1	0.161	0.175	0.677
INST AXLE LOAD	23.28	4	5.820	6.307	0.000
INST AXLE TP	0.245	2	0.123	0.133	0.876
INST AXLE SPD	1.568	2	0.784	0.850	0.432
INST LOAD TP	0.54	2	0.270	0.293	0.747
INST LOAD SPD	0.916	2	0.458	0.496	0.611
INST TP SPD	0.164	1	0.164	0.178	0.675
AXLE LOAD TP	4.77	4	1.192	1.292	0.282
AXLE LOAD SPD	13.753	4	3.438	3.726	0.009
AXLE TP SPD	2.263	2	1.131	1.226	0.300
LOAD TP SPD	5.093	2	2.546	2.759	0.071
Explained	981.838	79	12.428	13.468	0.000
Residual	59.059	64	0.923		
Total	1040.897	143	7.279		

Table 17. Primary ANOVA output from SPSS for the Southgate strain method.

Christison Deflection Equivalency Factor Method
Full Factorial - AASHTO Standard - By Instrument

ORIGINAL ANOVA

Source of Variation	SSquares	DF	MSquare	F	Sig.
Covariates	48.202	1	48.202	66.982	0.000
TEMP	48.202	1	48.202	66.982	0.000
Main Effects	471.649	8	58.956	81.926	0.000
PVMT	2.621	1	2.621	3.642	0.061
INST	0.451	1	0.451	0.627	0.431
AXLE	115.534	2	57.767	80.273	0.000
LOAD	364.291	2	182.146	253.111	0.000
TP	1.258	1	1.258	1.748	0.191
SPD	0.523	1	0.523	0.727	0.397
2-way Interactions	222.127	26	8.543	11.872	0.000
PVMT INST	5.46	1	5.460	7.587	0.008
PVMT AXLE	20.025	2	10.013	13.913	0.000
PVMT LOAD	25.654	2	12.827	17.824	0.000
PVMT TP	2.662	1	2.662	3.700	0.059
PVMT SPD	0.033	1	0.033	0.046	0.831
INST AXLE	9.179	2	4.590	6.378	0.003
INST LOAD	2.076	2	1.038	1.442	0.244
INST TP	0.128	1	0.128	0.178	0.674
INST SPD	0.37	1	0.370	0.514	0.476
AXLE LOAD	123.299	4	30.825	42.834	0.000
AXLE TP	1.61	2	0.805	1.118	0.333
AXLE SPD	9.145	2	4.572	6.354	0.003
LOAD TP	6.787	2	3.393	4.716	0.012
LOAD SPD	1.476	2	0.738	1.026	0.364
TP SPD	1.054	1	1.054	1.465	0.231
3-way Interactions	96.748	44	2.199	3.056	0.000
PVMT INST AXLE	6.932	2	3.466	4.816	0.011
PVMT INST LOAD	11.622	2	5.811	8.075	0.001
PVMT INST TP	0.077	1	0.077	0.106	0.745
PVMT INST SPD	3.803	1	3.803	5.284	0.025
PVMT AXLE LOAD	30.712	4	7.678	10.669	0.000
PVMT AXLE TP	1.426	2	0.713	0.991	0.377
PVMT AXLE SPD	0.222	2	0.111	0.155	0.857
PVMT LOAD TP	2.613	2	1.306	1.815	0.171
PVMT LOAD SPD	0.646	2	0.323	0.449	0.641
PVMT TP SPD	0.474	1	0.474	0.658	0.420
INST AXLE LOAD	14.354	4	3.588	4.986	0.001
INST AXLE TP	0.343	2	0.172	0.239	0.788
INST AXLE SPD	0.762	2	0.381	0.530	0.591
INST LOAD TP	0.266	2	0.133	0.185	0.832
INST LOAD SPD	0.128	2	0.064	0.089	0.915
INST TP SPD	0.049	1	0.049	0.068	0.795
AXLE LOAD TP	1.778	4	0.444	0.618	0.652
AXLE LOAD SPD	10.033	4	2.508	3.486	0.012
AXLE TP SPD	0.629	2	0.315	0.437	0.648
LOAD TP SPD	2.807	2	1.403	1.950	0.151
Explained	838.727	79	10.617	14.753	0.000
Residual	46.056	64	0.720		
Total	884.783	143	6.187		

SECTION 7. DISCUSSION OF RESULTS

The data analyses described in the previous section produced a wide variety of results worthy of interpretation or further analysis. There is a large amount of data in various forms over a wide range of parameters resulting from the experiments. Thus, there are a large number of possible analyses that could be performed. However, the scope of the project, designed to achieve certain objectives, limited the amount of analysis and interpretation of results that could be performed. Specifically, this investigation was to examine the viability of primary pavement response load equivalency factors. It was to identify the best currently available method for calculating primary response equivalencies and to identify which factors, such as vehicle operating parameters and environment, significantly affect load equivalency factors. Therefore, the analysis consisted of converting primary responses of deflection and strain to load equivalency factors using various methods and examining the statistical variability and trends of the various methods and to select one as the recommended method based on the results of this study.

The previous section describes the analysis undertaken. This section examines the results of these analyses and helps to provide insight and interpretation of the results as they relate to the engineering aspects of primary response load equivalency factors. Key to this discussion is defining the inherent variability of the measurements used to calculate load equivalency factors. These variabilities are calculated and then used to review the actual variations occurring in the data from the factors being studied in the experiments. The important factors include axle type, axle load, tire pressure, and speed as well as structural and environmental factors such as pavement thickness and pavement temperature. Two deflection based load equivalency factor methods were examined. They are compared and one selected for additional scrutiny. Two strain methods were also examined and compared. All primary response methods are compared to the AASHO method of load equivalency factors. This section discusses these detailed analyses and results obtained.

VARIATION OF INSTRUMENTS

A key to getting useful information from any data is to accurately determine the measurement errors in the experiment and to use the quantity of those errors in accessing the effects of the other factors of the experiment. A method was selected on this project to access the variability of instruments by placing replicate sets of instruments in each pavement section. This factor in the experiment became known as "instruments nested within pavements." This is a statistical experiment design in which two sets of random instruments are placed in each pavement section and thus cannot be analyzed as a fixed variable in the analysis. It must be analyzed as a nested factorial experiment with instruments nested in pavements. The SPSS computer program can not analyze this type of factorial directly, so a correction to the output was necessary as described in the previous section. This correction used the variation from instruments and interactions with instruments as the proper error terms to test for each effect and multi-factor interaction.

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Observations Concerning Instrument Variation

The actual variation between the replicate instruments within a pavement section was substantial. The repeatability between the same instruments, however, was quite good. That is to say that one instrument would repeatedly read about the same as in the previous run with the same truck on the same instrument. The reproducibility of instruments however, was not quite as good as the repeatability because the same load passing over the two sets of instruments in the pavement may give somewhat different results on each set of instruments. The variation between the deflection gauges was not large enough to significantly mask the affects of the factors and interactions in affecting load equivalency factors. The variation in the strain gauges is somewhat larger and tends to mask some of the estimation of these effects. However, similar trends in the variation of the factors were observed with the strain data as with the deflection data although the overall level of significance was not as high.

The actual reason for the variation between the replicate instruments is not clear or completely definable. The variation could in a large part be caused by one of two major reasons. Either the instruments themselves vary between instrument locations or the pavement strength and qualities from the two locations vary. The actual variation is likely caused by a combination of both reasons.

Significance of Variation on LEF Estimation

When calculating load equivalency factors using a primary response variable, a ratio of response measures is used. The ratio of the response at the load in question to the response at the standard load should be the same between the two sets of instruments if there is a consistent difference between the two instruments. Thus the analysis was performed using an estimate of the standard load response from one instrument to calculate the load equivalency factors of all responses from all loads for that instrument. It is believed that in this process the variation of instruments in estimating load equivalency factors tends to cancel and is minimized. In the statistical analyses, an attempt was made to pool each of the instrument associated error terms with the residual error term to make the F-tests on all of the factors. Each time these error terms could be pooled indicates that the instrument variation is not significantly affecting the load equivalency variations. Thus, this provides support for the assumption that instrument variations can essentially cancel out in a load equivalency factor determination. In the case where the error terms would not pool, then the instrument variation was used directly to make the F-test because it is significant and must be considered in analyzing the variations of the factors associated with that error term.

The trend of the instrument variation canceling in the equivalency factor calculation is more evident in the deflection measurements. The strain data does not show this trend as well. This is probably due to several reasons. First, as mentioned above, the reproducibility and repeatability of the strain gauges is not as good as the deflection gauges. Secondly, a number of strain measurements were missing from the data set due to malfunctioning data collection equipment, malfunctioning gauges, and the dedication of two strain channels at all times to the lateral distance measuring instrument. Therefore, the strain data was not nearly as complete as the deflection data and thus has probably contributed to the larger variations and lack of the canceling trend in the equivalency factor calculations. The problems with the strain gauges and measuring equipment were at their worst on the first day of measurements which is when all of the standard load data was collected for the project. Also, the channel 11 strain gauge was not functioning properly throughout the entire experiment. It is evident that this gauge was probably damaged in the paving process. Thus, the variation and problems with the strain data can probably be improved with greater quality control over the strain measurements and full replication of the strain gauges which was not accomplished in the current experiment due to the lateral position indicators using strain channels and non-functioning gauges. Although instrument variation was significant, especially in the case of the strain gauges, it was quantified and accounted for in the analyses.

It is clear that better control and more replication of the measurements, especially with regard to lateral position of the vehicle and full replication of all instruments within the sections would produce better results. This would require that only one pavement section be tested at a time in the future so that all channels can be utilized and the lateral position measurement can be replicated on each section. A larger number of replicate runs of the vehicle should also be used and better randomization of the measurements should be used. Also, each load should be accompanied by a run of the standard axle load at the same time. This would give a much better array of standard axle responses with which to calculate load equivalency factors. None of the suggestions presented here could have been accomplished on this project due to budget or other constraints. They should, however, be considered if additional such testing is planned. They should also be considered in any pavement response measurements made for research purposes.

Meaning of Significance Levels

In order to test for the significance of each of the factors, the 10 percent, 5 percent and 1 percent significance levels were used throughout the studies. The 10 percent level is used to expose the possibility that this particular effect or interaction could be important. This is not strong evidence, however. The 1 percent level indicates strong evidence of significance and definitely should be considered in engineering actions or decisions. The 5 percent level lies between these two extremes and usually is taken to cause engineering action or affect decisions.

If an effect or interaction is not significant at the 10 percent level (NS), no basis for engineering action or decisions is present. The reason for this is that the experiment was carried out extremely well so that the statistical tests of significance have excellent power. That is, the results reflect what is true better than any experiment to date because of the replication of instruments as a basis for the statistical tests.

DISCUSSION OF DEFLECTION METHOD VARIATIONS

The two load equivalency factor methods analyzed have been discussed previously. The first method selected was proposed by Christison and was used on the Canadian "Vehicle Weights and Dimensions Study."[34] This method was identified by the FHWA for study on this project. The second method selected is a modification of the method proposed by Christison. This is the method proposed at the University of Waterloo by Hutchinson.[10] The Hutchinson method uses the ASTM Standard Practice for Cycle Counting in Fatigue Analysis as a basis for modifying the analysis used by Christison.[12] This basically involves taking the largest hump of the response profile from multiaxles as the primary term in the equivalency factor equation. Christison used the first response hump of the multiaxle group as the primary term. This has the effect of making the Hutchinson equivalency factors either equal to or higher than Christison factors in all cases. For all single axles, the two methods give exactly the same value. For multiple axles where the first wheel of the group gives a higher response than the trailing wheels, or they give the exact same response, then the methods produce the same values for load equivalency factors. However, if the response of the trailing wheels is higher than the lead wheel in a multiaxle group, the Hutchinson method will always produce a higher factor. And based on the results of these experiment tests, the trailing wheels almost always gave a higher response than the lead wheel of an axle group.

Comparison of Deflection Methods to AASHTO Load Equivalency Factors

The question then arises which of the methods are most reasonable in predicting load equivalency factors. The most widely used set of truck-load equivalency factors are produced by AASHTO and derived from the large-scale experiments at the AASHO Road Test. Therefore, comparison of a load equivalency factor from each method with the AASHO Road Test factors is useful to determine which method most likely produces reasonable load equivalency factors. Table 18 shows a comparison of the AASHTO factors at each of the load levels tested on this project with the factors from each of the pavements, instruments, and methods used on this project for the deflection based data. The AASHTO factors are shown plotted against average values of the primary response equivalencies obtained on this project in figures 33 through 35 for each truck type.

It is apparent that for tandem axles the Hutchinson factors are the most reasonable if the AASHTO factors are used as a basis of comparison. For the tridem axle, the Christison method is somewhat closer to the AASHTO values although not at all loads. For purposes of these comparisons, the AASHTO factors are considered the best available empirically derived damage related load equivalency factors in existence. These factors were derived from the largest experiment ever undertaken on pavement loading and damage. They have been adopted by most of the State highway departments for design purposes and are used far more than any other load equivalency factors available. It should be noted that the AASHO Road Test was performed at a relatively constant speed and tire pressure. The values compared and plotted in this section are at the speed and tire pressure most closely resembling the AASHO Road Test. Thus, with these AASHO conditions as a basis for estimating the standard

Table 18. Comparison of deflection based primary response LEF methods with AASHTO factors.

		CHRIST. PAVE	HUTCH. PAVE		% DIFF	% DIFF
TRUCK	LOAD	AVERAGE	AVERAGE	AASHTO	CHRISTISON	HUTCHISON
1	27	8.27	8.27	5.11	61.90	61.90
1	18	0.88	0.88	1.00	-12.25	-12.25
1	9	0.05	0.05	0.08	-40.83	-40.83
2	44	1.30	2.09	2.99	-56.52	-30.02
2	32	0.26	0.42	0.89	-71.03	-52.76
2	20	0.05	0.07	0.16	-69.14	-55.25
3	60	2.16	3.58	2.48	-13.00	44.35
3	42	1.33	2.03	0.66	100.45	206.73
3	24	0.36	0.52	0.08	325.60	516.07

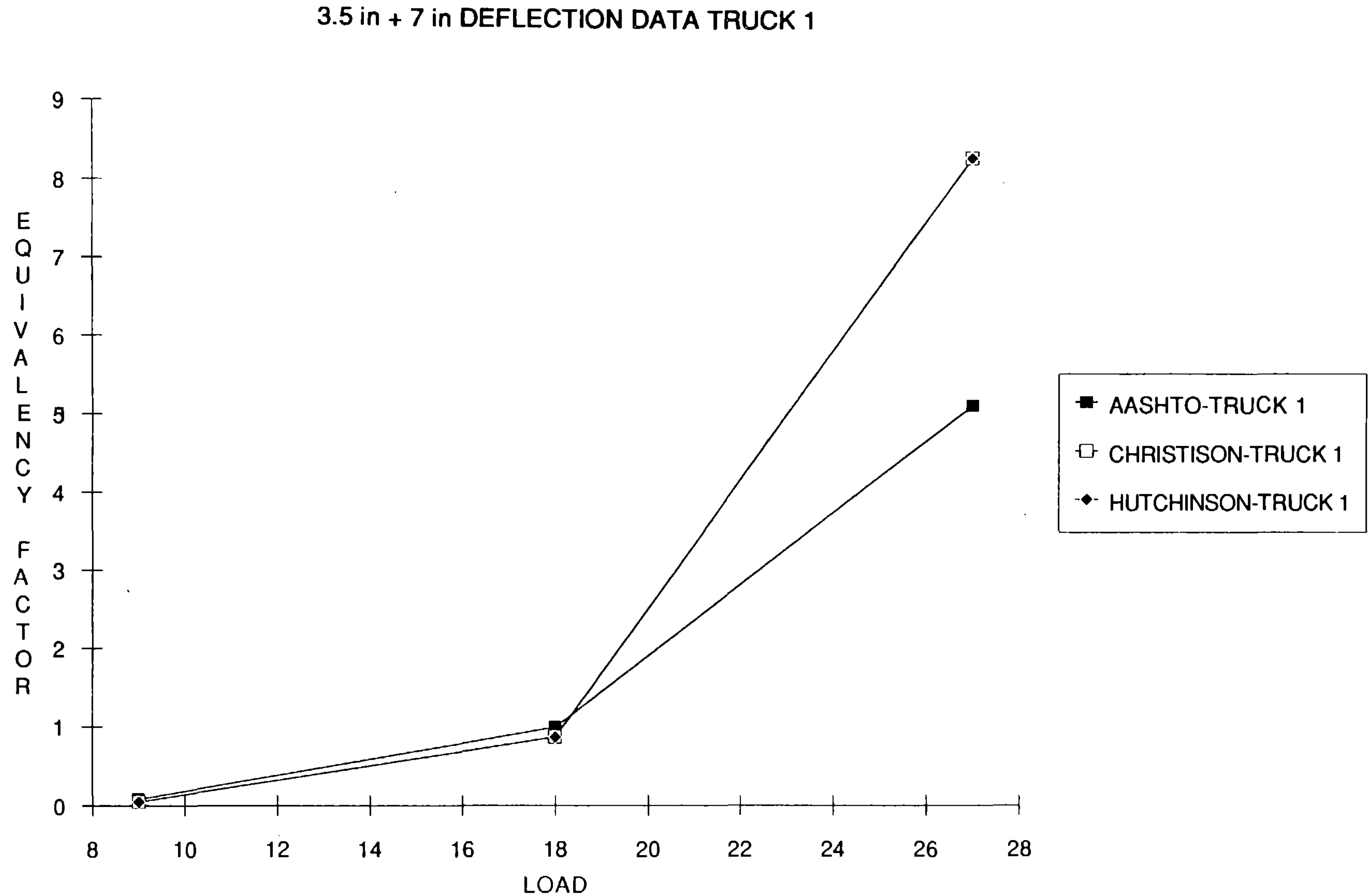


Figure 33. Plot of deflection based equivalency methods versus the AASHTO factors for single axles.

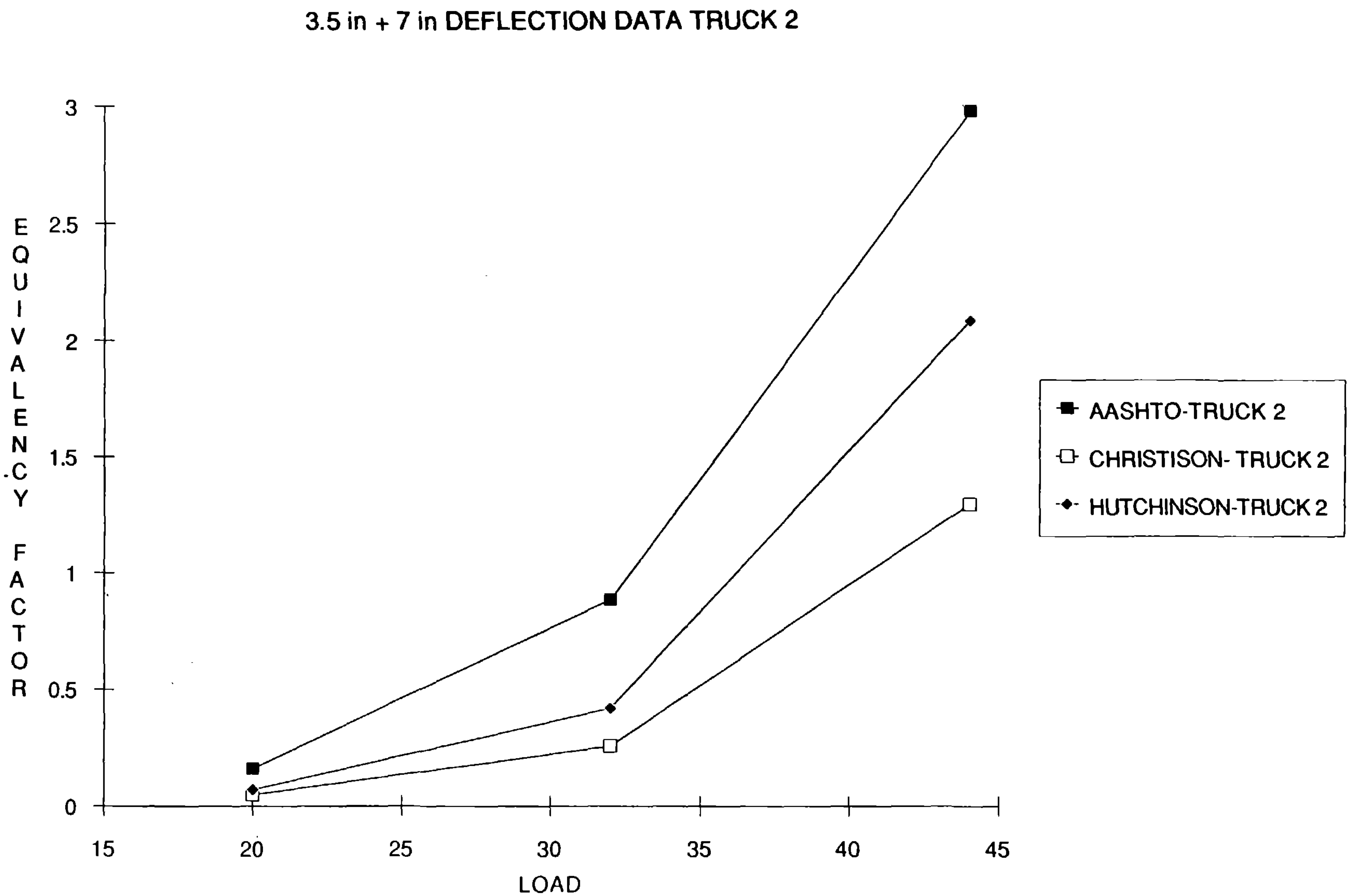


Figure 34. Plot of deflection based equivalency methods versus the AASHTO factors for tandem axles.

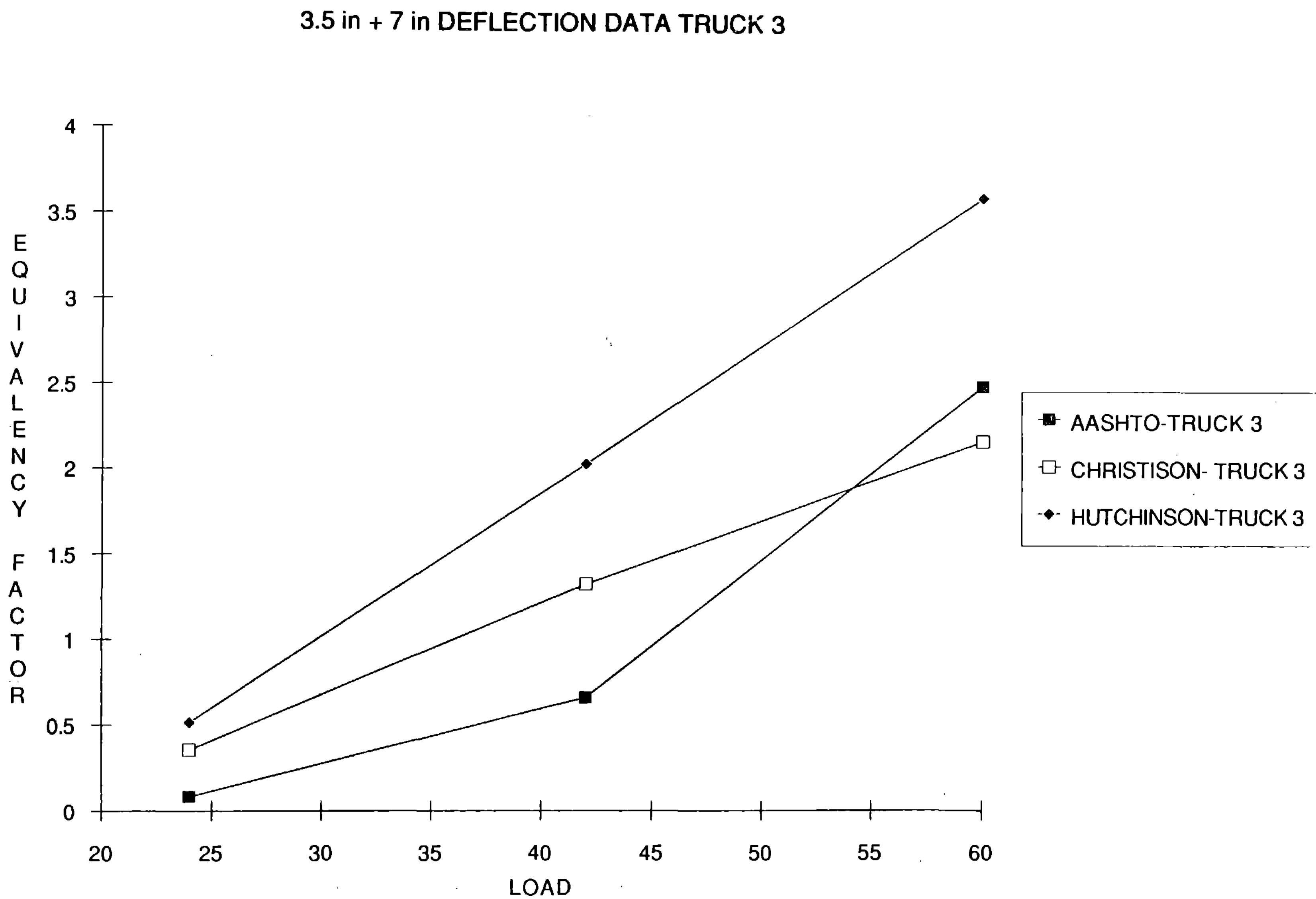


Figure 35. Plot of deflection based equivalency methods versus the AASHTO factors for tridem axles.

load response, the effects of tire pressure and speed on load equivalency factors can better be evaluated.

Based on the above comparisons, the Hutchinson method for calculating deflection based primary response load equivalency factors was selected for further evaluation and recommendation on this project. Several reasons for this selection exist. For tandem axles which are by far the most common type of multiaxle operating on U.S. highways, the Hutchinson method more closely predicted the AASHTO values. Both methods seemed to underpredict the values to some degree. For the tridem axles, however, both methods predict higher load equivalencies than the AASHTO factors. Since there were no tridem axles at the AASHO Road Test, the AASHTO tridem factors were derived in an indirect method from the AASHO Road Test results. Therefore, the tridem AASHTO factors are probably more questionable than the single axle or tandem axle factors. The Hutchinson method can be considered a more conservative estimator of load equivalency factors than the Christison method due to the fact that it will always predict the same or higher equivalency values. For all of these reasons, the Hutchinson method was selected as the preferred equivalency factor procedure on this project for deflection based measurements.

Effects of Experimental Factors on Deflection Methods

A main objective of these experiments was to determine which pavement or vehicle factors most greatly influence the load equivalency factors predicted from primary pavement response. It is desirable to know the influence due to changes in tire pressure, vehicle speed, axle load, axle type, pavement strength or thickness, and pavement temperature. The relative influence of these various factors and the interactions of these factors with each other were quantified by the analysis within the framework of the statistical variation of the measurement process as discussed above.

It is interesting to note that both equivalency factor methods, although different in their magnitudes of load equivalency factor predictions, show a similar pattern of which factors were significant in influencing the LEF's predicted by the method. This indicates that no matter which method is used to calculate a primary response based load equivalency factor, the main influencing factors affect the results in a relatively similar manner. This also indicates that the factors identified in this study as significantly affecting primary response load equivalency factors, are quite likely the most important factors in the overall concept of equivalent loading and damage. The magnitudes of the influence of each of the important factors in each of the methods is discussed in the following sections.

Christison Deflection Method

The initial analysis-of-variance (ANOVA) run on the data using the SPSS statistical package was presented in the previous section. Two corrections to these base runs are required to account for the instrument variation. The results of the first level correction to the ANOVA produced by SPSS for the Christison deflection method is shown in table 19. This is the direct result of using the correct instrument (and inter-

Table 19. First level corrected ANOVA - Christison deflection method.

Christison Deflection Equivalency Factor Method
Full Factorial - AASHTO Standard - By Instrument

Corrected ANOVA using the interactions with instruments (INST) to make the F tests

SOURCE	df	MS	F	Sig of F
TEMP	1	48.20	16.31	.1
PVMT	1	2.62	0.89	NS
Error (1)	2	2.96		
LOAD	2	182.15	53.19	.01
(PVMT)x(LOAD)	2	12.83	3.75	.25
Error (2)	4	3.42		
TP	1	1.26	12.27	.1
(PVMT)x(TP)	1	2.66	25.97	.05
Error (3)	2	0.10		
(LOAD)x(TP)	2	3.39	7.96	.05
(PVMT)x(LOAD)x(TP)	2	1.31	3.06	.25
Error (4)	4	0.43		
AXLE	2	57.77	14.34	.05
(PVMT)x(AXLE)	2	10.01	2.49	.25
Error (5)	4	4.03		
(LOAD)x(AXLE)	4	30.83	14.31	.01
(PVMT)x(LOAD)x(AXLE)	4	7.68	3.56	.1
Error(6)	8	2.15		
(TP)x(AXLE)	2	0.81	1.38	NS
(PVMT)x(TP)x(AXLE)	2	0.71	1.22	NS
(LOAD)x(TP)x(AXLE)	4	0.44	0.76	NS
Error (7)	8	0.58		
SPD	1	0.52	0.25	NS
(PVMT)x(SPD)	1	0.03	0.02	NS
Error (8)	2	2.09		
(LOAD)x(SPD)	2	0.74	1.88	NS
(PVMT)x(LOAD)x(SPD)	2	0.32	0.82	NS
Error(9)	4	0.39		
(TP)x(SPD)	1	1.05	1.91	.25
(PVMT)x(TP)x(SPD)	1	0.47	0.86	NS
(LOAD)x(TP)x(SPD)	2	1.40	2.54	.25
Error (10)	4	0.55		
(AXLE)x(SPD)	2	4.57	7.01	.05
(PVMT)x(AXLE)x(SPD)	2	0.11	0.17	NS
(LOAD)X(AXLE)x(SPD)	4	2.51	3.85	.05
(TP)x(AXLE)x(SPD)	2	0.32	0.48	NS
Error (11)	10	0.65		
Residual	64	0.72		

actions with instrument) mean squares to make the tests of significance on the main effects and interactions of the other factors in the experiments.

To obtain the most powerful tests for this analysis, the errors with mean squares that are not significantly different from the residual mean square are pooled. The 25 percent significance level is used for this purpose. Errors (3), (4), (7), (9), (10), and (11) have mean squares that are not significantly different from the residual mean square using the 25 percent confidence level. The calculations to obtain the pooled error are shown in table 20. The final corrected ANOVA is presented in table 21.

From these pooled results, plus the effects and interactions whose errors could not be pooled, the summary of the significant effects and interactions for the Christison deflection method follow:

At α = .10
- TEMP
- (PVMT) x (LOAD) x (AXLE)

At α = .05
- AXLE
- (PVMT) x (TP)
- (LOAD) x (AXLE) x (SPD)

At α = .01
- LOAD
- (LOAD) x (AXLE)
- (LOAD) x (TP)
- (AXLE) x (SPD)

This indicates that at a significance level of .01 (the most significant), load is the only main effect that is significant. This confirms the basic premise of a load equivalency factor and the results obtained at the AASHO Road Test that load was by far the most significant factor in producing pavement damage. Figure 36 shows the variation of the means of the Christison equivalency factors with load. The two factor interactions which were significant at the .01 level are plotted in figures 37, 38, and 39. These plots show the effects of the variations of both factors simultaneously.

At a significance level of .05, Axle is the only main effect that shows significance. Figure 40 shows the variation of equivalency factor with axle type. One reason for this strange pattern of variation with axle type is partly due to the load levels selected within each axle type. If a higher set of loads would have been used on the tandem axle, this pattern could have been significantly changed. Therefore, the one-way analysis of Load and Axle as described later helps sort out the effects of this load level selection. The only two-way interaction which is significant is Tire Pressure x Pavement as shown in figure 41. The effect of tire pressure is more pronounced on the thin pavement. The effect is opposite of what was expected with a higher LEF for the low tire pressure.

Table 20. Pooled error calculation for Christison deflection method.

SOURCE	df	MS	SS
Error (3)	2	.10	.20
Error (4)	4	.43	1.72
Error (7)	8	.58	4.64
Error (9)	4	.39	1.56
Error (10)	4	.55	2.20
Error (11)	10	.65	6.50
Residual	64	.72	46.08
Pooled Error	96	.66	62.90

Table 21. Final corrected ANOVA - Christison deflection method.

SOURCE	df	MS	F	Significance
TEMP	1	48.20	16.31	.10
PVMT	1	2.62	.89	NS
Error (1)	2	2.96		
LOAD	2	182.15	53.19	.01
(PVMT) x (LOAD)	2	12.83	3.75	NS
Error (2)	4	3.42		
AXLE	2	57.77	14.34	.05
(PVMT) x (AXLE)	2	10.01	2.49	NS
Error (5)	4	4.03		
(LOAD) x (AXLE)	4	30.83	14.31	.01
(PVMT) x (LOAD) x (AXLE)	4	7.68	3.56	.10
Error (6)	8	2.15		
SPD	1	.52	.25	NS
(PVMT) x (SPD)	1	.03	.02	NS
Error (8)	2	2.09		
TP	1	1.26	1.9	NS
(PVMT) x (TP)	1	2.66	4.0	.05
(LOAD) x (TP)	2	3.99	6.0	.01
(PVMT) x (LOAD) X (TP)	2	1.31	2.0	NS
(TP) x (AXLE)	2	.81	1.2	NS
(PVMT) x (AXLE)	2	.71	1.1	NS
(LOAD) x (TP) x (AXLE)	4	.44	< 1	NS
(LOAD) x (SPD)	2	.74	1.1	NS
(PVMT) x (LOAD) x (SPD)	2	.32	< 1	NS
(TP) x (SPD)	1	1.05	1.6	NS
(PVMT) x (TP) x (SPD)	1	.47	< 1	NS
(LOAD) x (TP) x (SPD)	2	1.40	2.1	NS
(AXLE) x (SPD)	2	4.57	6.9	.01
(PVMT) x (AXLE) x (SPD)	2	.11	< 1	NS
(LOAD) x (AXLE) x (SPD)	4	2.51	3.8	.05
(TP) x (AXLE) (SPD)	2	.32	< 1	NS
Pooled Error	96	.66		

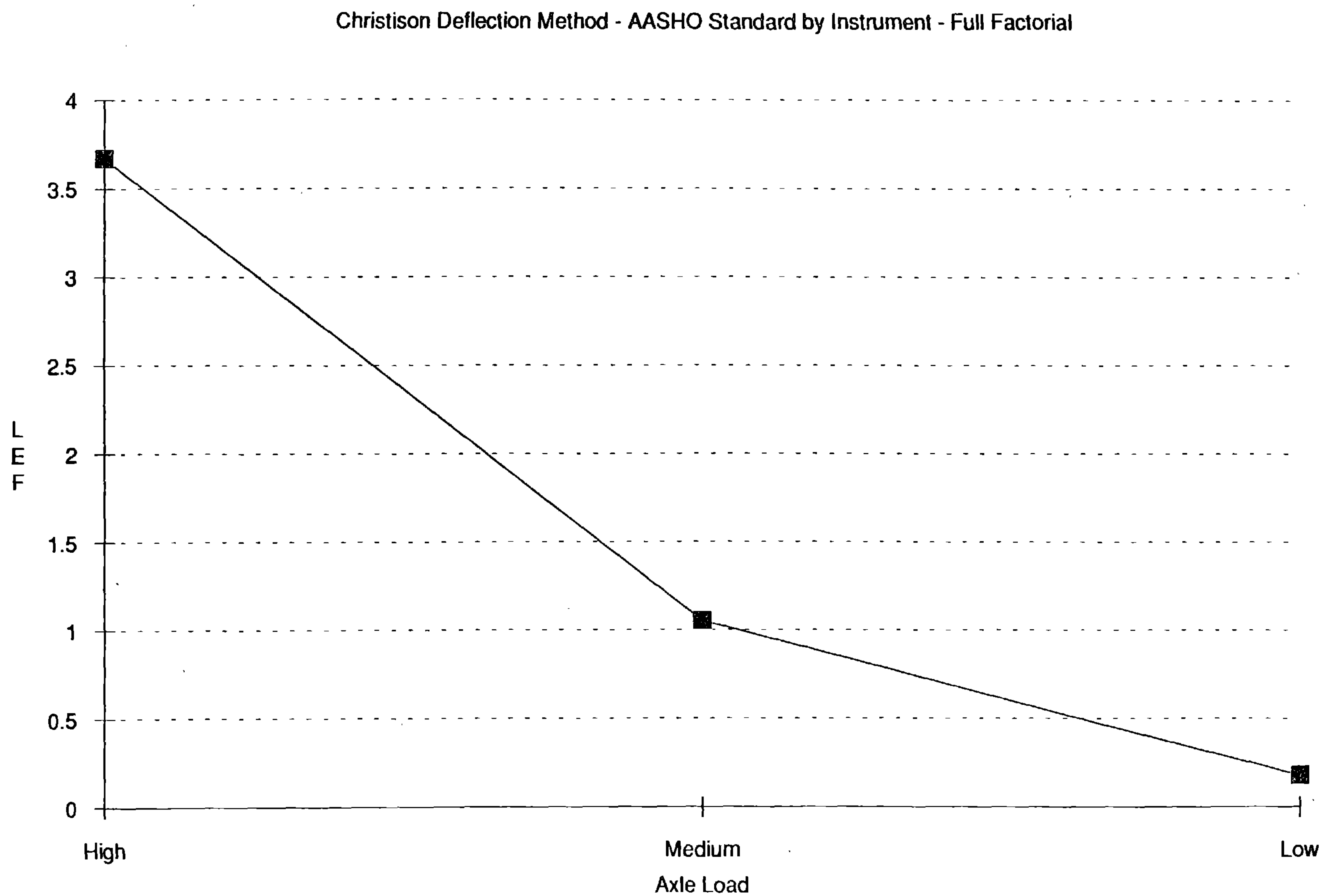


Figure 36. Cell mean plot of the axle load main effect for the Christison deflection method.

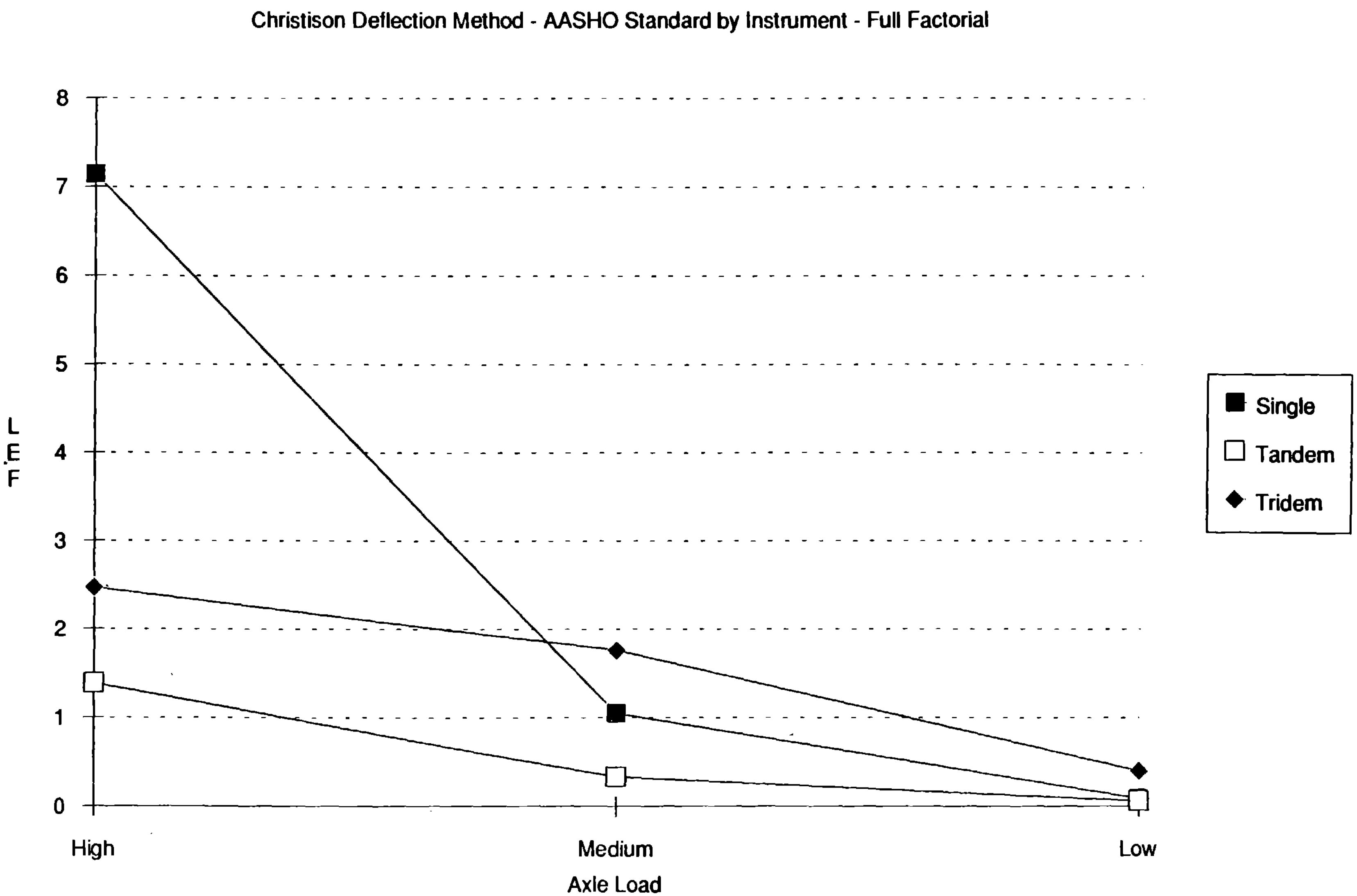


Figure 37. Cell mean plot of the axle load x axle type interaction for the Christison deflection method.

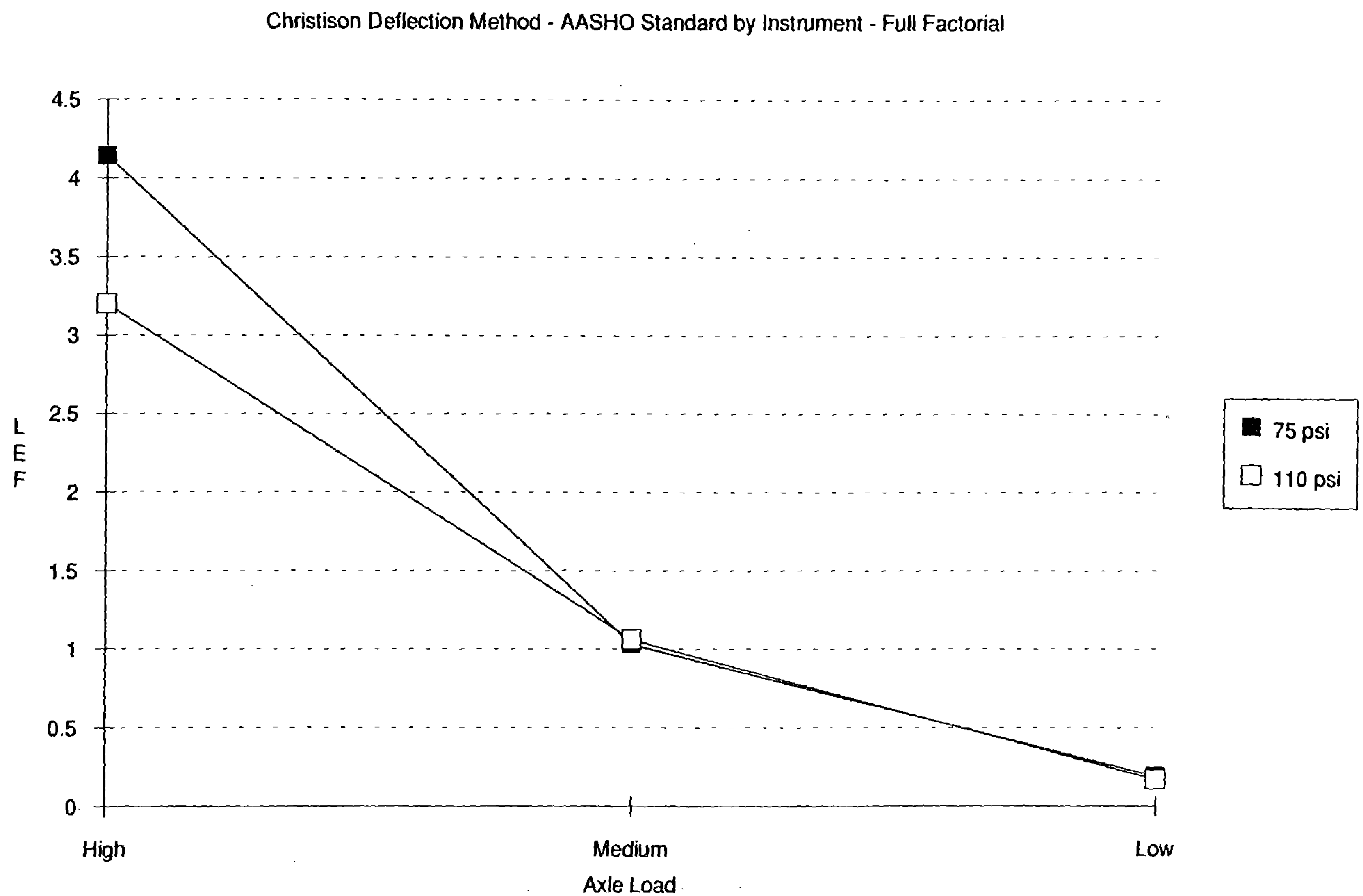


Figure 38. Cell mean plot of the axle load x tire pressure interaction for the Christison deflection method.

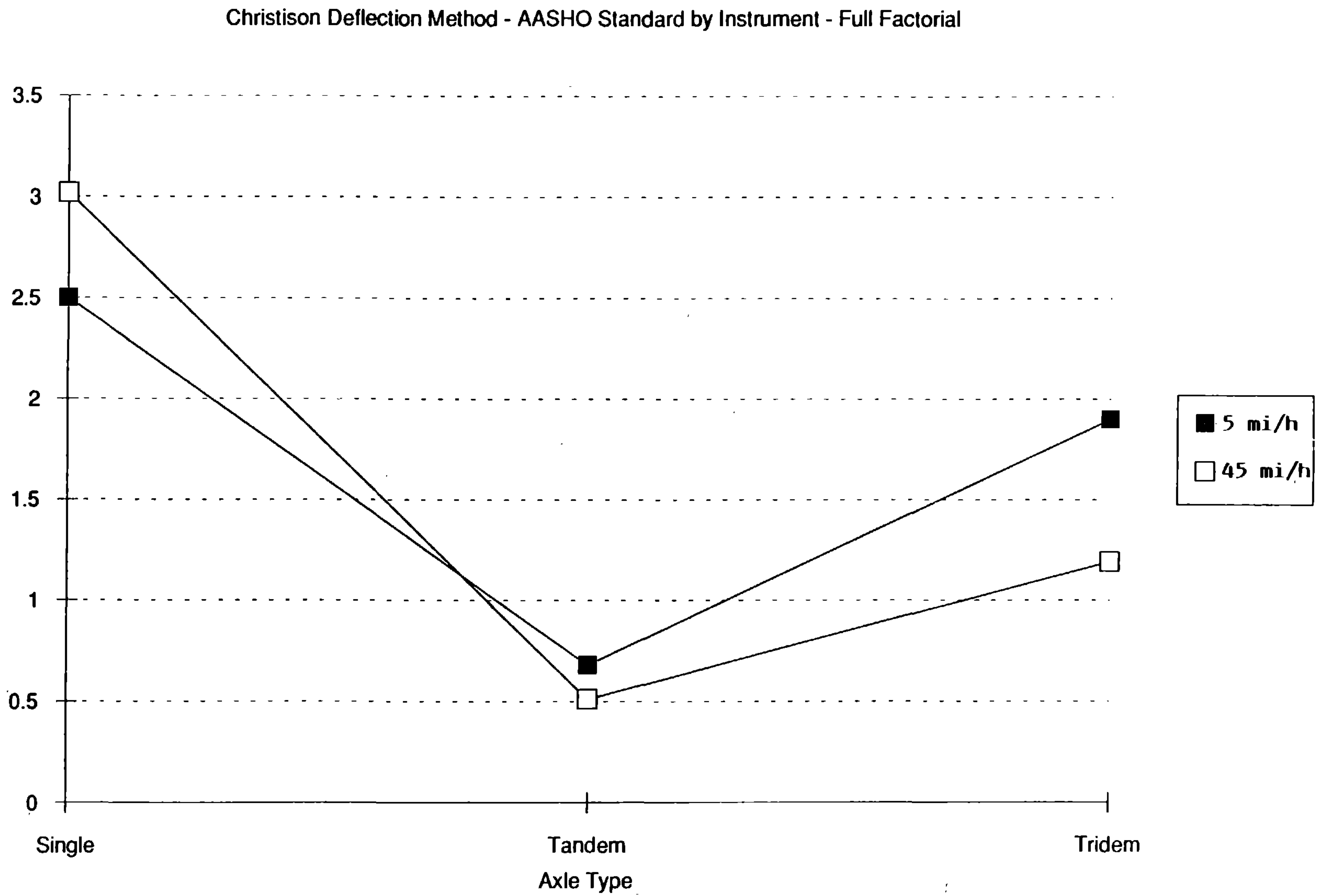


Figure 39. Cell mean plot of the axle load x speed interaction for the Christison deflection method.

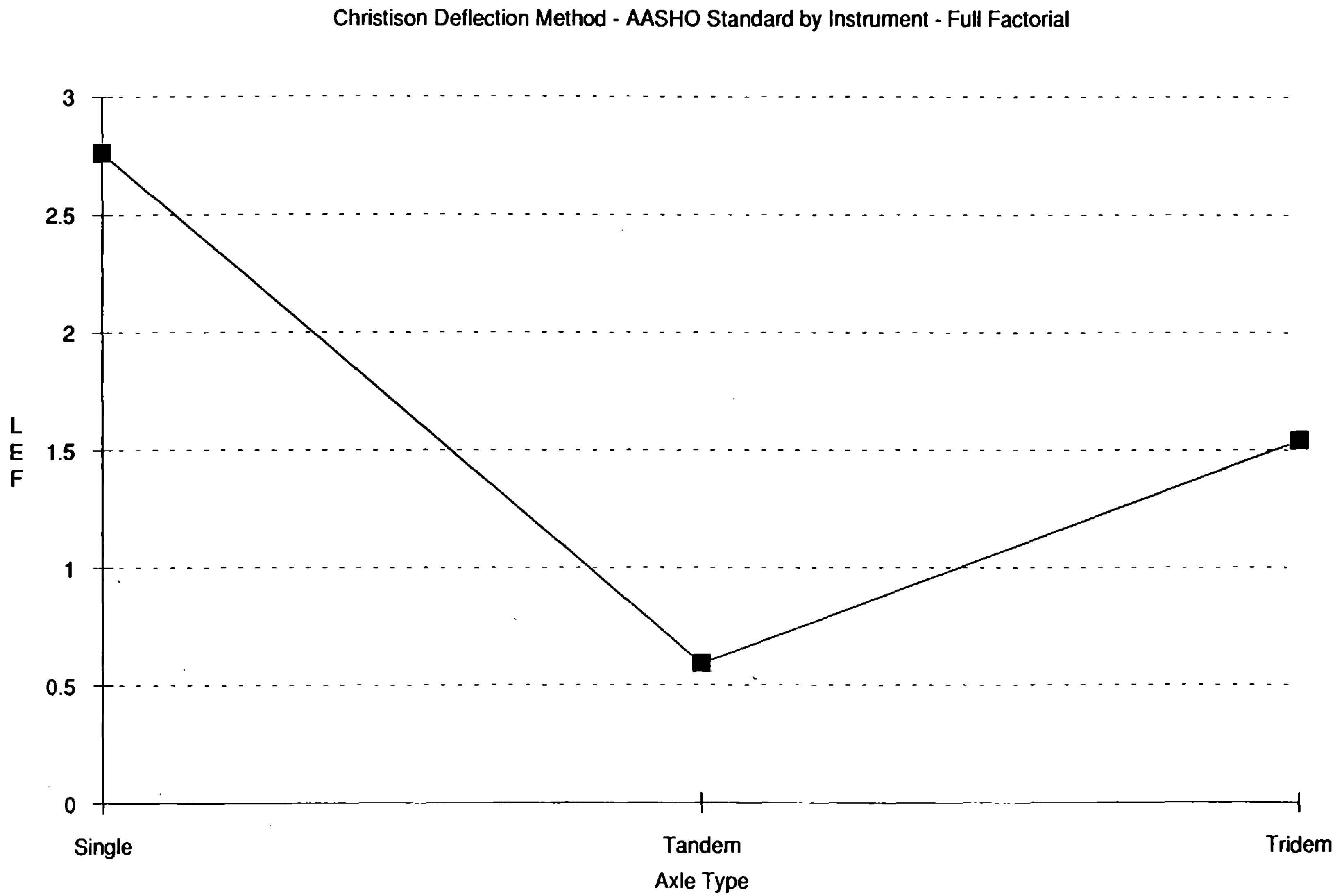


Figure 40. Cell mean plot of the axle type main effect for the Christison deflection method.

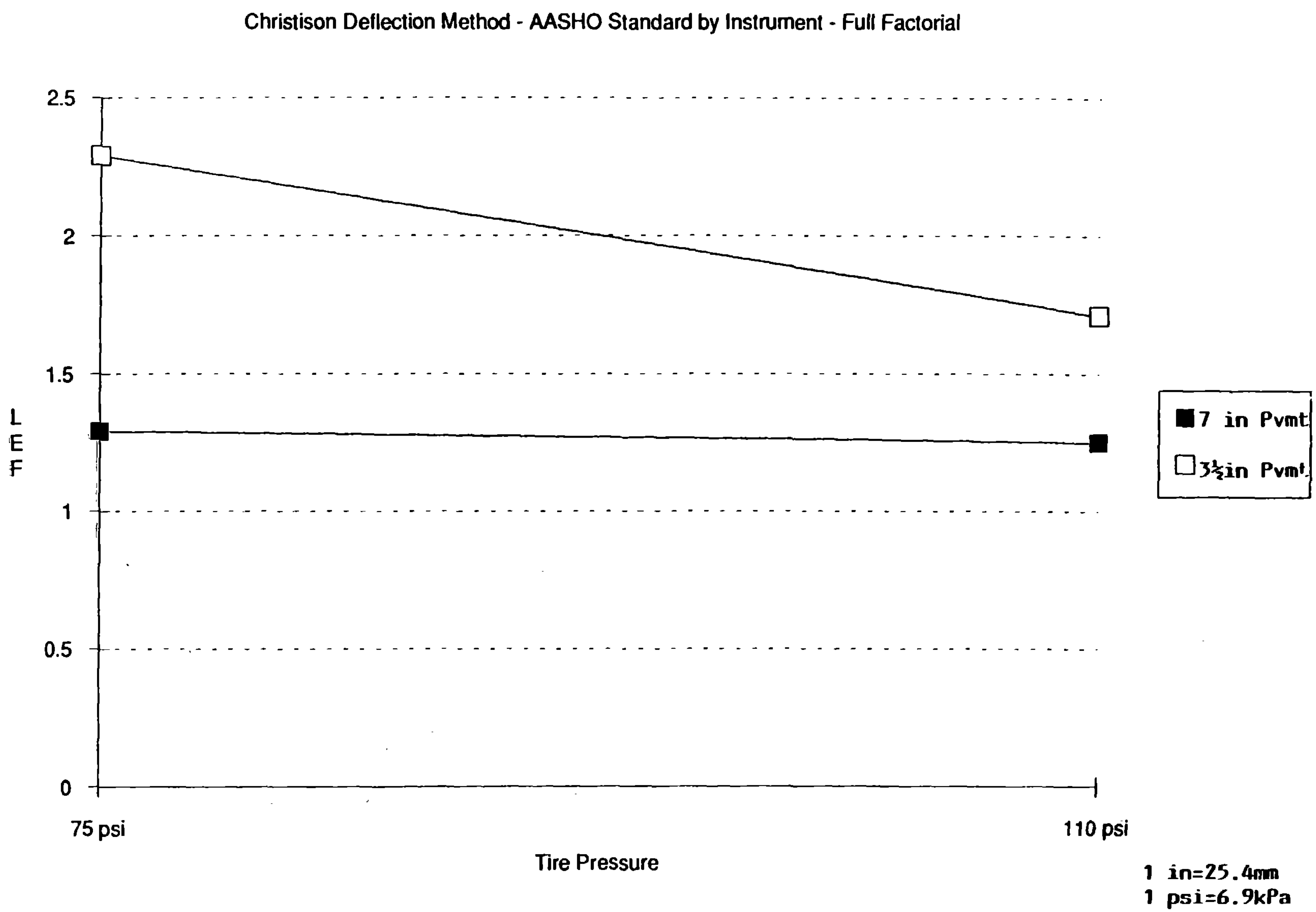


Figure 41. Cell mean plot of the tire pressure x pavement interaction for the Christison deflection method.

One three-way interaction was also significant at the .05 level. This interaction was Load x Axle x Speed. A plot of the three-way interaction is shown in figure 42. The one-way analysis of Axle x Load helps understand the effects of this three-way interaction with Speed.

Hutchinson Deflection Method

The results of the first level correction to the ANOVA produced by SPSS for the Hutchinson deflection method is shown in table 22. This is the direct result of using the correct instrument (and interactions with instrument) mean squares to make the tests of significance on the main effects and interactions of the other factors in the experiment.

The pooled error mean square is calculated as shown in table 23. The pooled error is then used to test each of the effects and interactions that are associated with the errors that were pooled. The resulting final ANOVA for the Hutchinson deflection method is shown in table 24.

From the final corrected ANOVA, the following significant effects and interactions are observed:

At α = .10
- TP
- (PVMT) x (LOAD)
- (PVMT) x (LOAD) x (TP)
- (LOAD) x (TP) x (SPD)

At α = .05
- TEMP
- (PVMT) x TP

At α = .01
- LOAD
- (LOAD) x (AXLE)
- (LOAD) x (TP)
- (AXLE) x (SPD)
- (LOAD) x (AXLE) x (SPD)

It is evident by the number of effects and interactions that are significant that the Hutchinson deflection method is more sensitive to each of these factors than is the Christison method. With this method Tire Pressure is showing to be significant at the .10 level. This indicates that there is a possibility that Tire Pressure should be considered as a factor in an overall LEF model using the Hutchinson deflection method. At the .05 significance level, Temperatures show to be significant. Temperature was analyzed as a covariance in the analysis. The fact that Temperature is showing to be significant indicates that a more detailed experiment and analysis to develop an equivalency factor model should consider pavement temperature when measuring the response of the standard axle load. That is to say, an estimate of the standard axle load response should be measured at the same time that the response of the load for which the equivalency factor is being developed is measured.

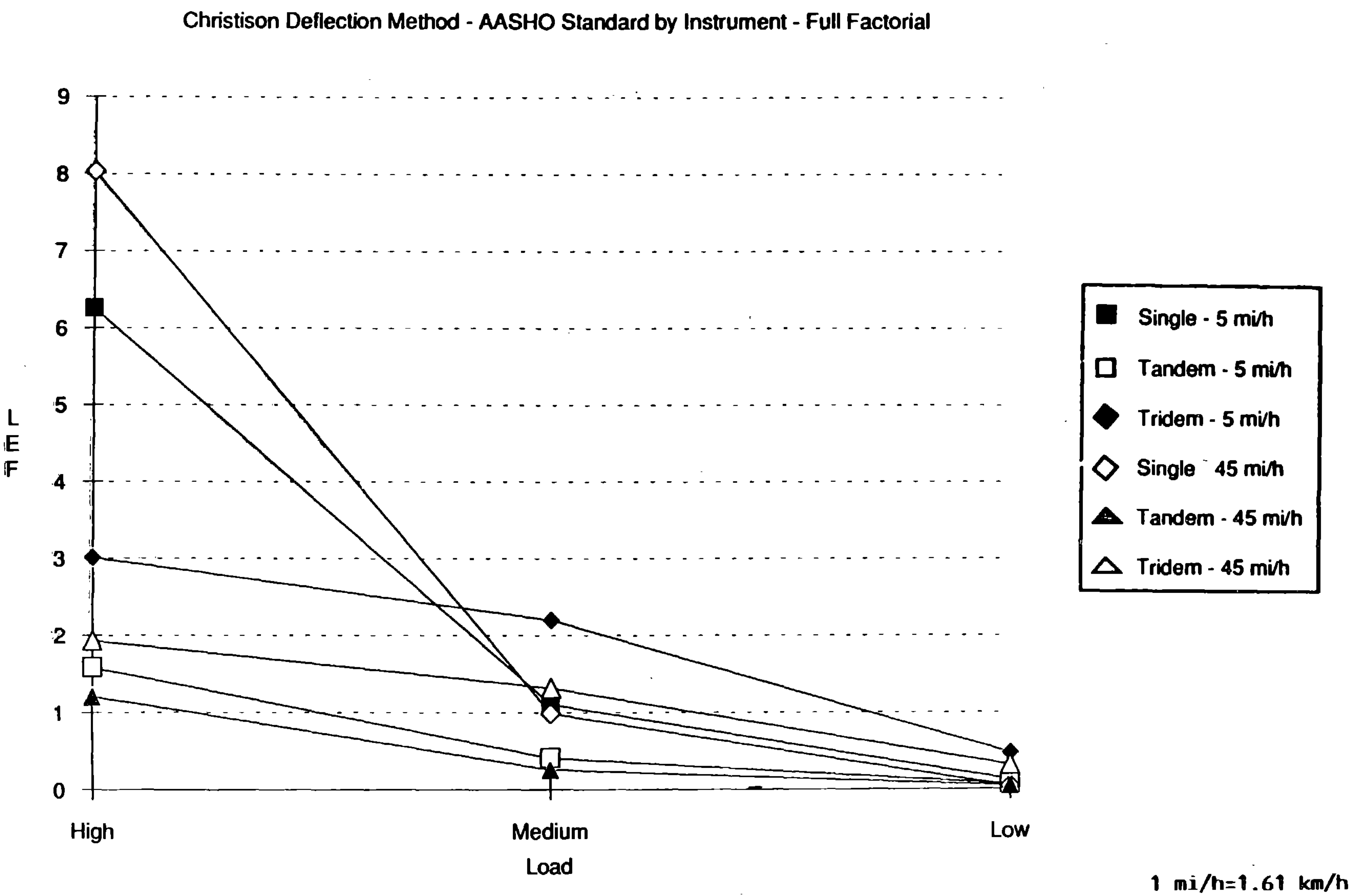


Figure 42. Cell mean plot of the axle load x axle type x speed interaction for the Christison deflection method.

Table 22. First level corrected ANOVA - Hutchinson deflection method.

Hutchinson Deflection Equivalency Factor Method
Full Factorial - AASHTO Standard - By Instrument

Corrected ANOVA using the interactions with instruments (INST) to make the F tests

SOURCE	df	MS	F	Sig of F
TEMP	1	72.38	25.12	.05
PVMT	1	4.18	1.45	NS
Error (1)	2	2.88		
LOAD	2	244.05	101.90	.01
(PVMT)x(LOAD)	2	13.17	5.50	.1
Error (2)	4	2.40		
TP	1	3.27	16.84	.1
(PVMT)x(TP)	1	3.45	17.80	.1
Error (3)	2	0.19		
(LOAD)x(TP)	2	6.05	10.14	.05
(PVMT)x(LOAD)x(TP)	2	3.03	5.07	.1
Error (4)	4	0.60		
AXLE	2	16.35	2.22	.25
(PVMT)x(AXLE)	2	11.14	1.51	NS
Error (5)	4	7.36		
(LOAD)x(AXLE)	4	29.82	8.84	.01
(PVMT)x(LOAD)x(AXLE)	4	7.93	2.35	.25
Error(6)	8	3.37		
(TP)x(AXLE)	2	1.23	1.70	.25
(PVMT)x(TP)x(AXLE)	2	0.58	0.81	NS
(LOAD)x(TP)x(AXLE)	4	1.19	1.65	NS
Error (7)	8	0.72		
SPD	1	3.57	1.42	NS
(PVMT)x(SPD)	1	0.01	0.00	NS
Error (8)	2	2.50		
(LOAD)x(SPD)	2	1.34	1.94	NS
(PVMT)x(LOAD)x(SPD)	2	0.15	0.21	NS
Error(9)	4	0.69		
(TP)x(SPD)	1	1.91	2.60	.25
(PVMT)x(TP)x(SPD)	1	0.16	0.22	NS
(LOAD)x(TP)x(SPD)	2	2.55	3.47	.25
Error (10)	4	0.73		
(AXLE)x(SPD)	2	9.61	10.74	.01
(PVMT)x(AXLE)x(SPD)	2	0.12	0.13	NS
(LOAD)X(AXLE)x(SPD)	4	3.44	3.84	.05
(TP)x(AXLE)x(SPD)	2	1.13	1.26	NS
Error (11)	10	0.90		
Residual	64	0.92		

Table 23. Pooled error calculation for Hutchinson deflection method.

SOURCE	df	MS	SS
Error (3)	2	.19	.38
Error (4)	4	.60	2.40
Error (7)	8	.72	5.76
Error (9)	4	.69	2.76
Error (10)	4	.73	2.92
Error (11)	10	.90	9.00
Residual	64	.92	58.88
Pooled Error	96	.85	82.10

Table 24. Final corrected ANOVA - Hutchinson deflection method.

SOURCE	df	MS	F	Significance
TEMP	1	72.38	25.12	.05
PVMT	1	4.18	1.45	NS
Error (1)	2	2.88		
LOAD	2	244.05	101.90	.01
(PVMT) x (LOAD)	2	13.17	5.50	.10
Error (2)	4	2.40		
AXLE	2	16.35	2.22	NS
(PVMT) x (AXLE)	2	11.14	1.51	NS
Error (5)	4	7.36		
(LOAD) x (AXLE)	4	29.82	8.84	.01
(PVMT) x (LOAD) x (AXLE)	4	7.93	2.35	NS
Error (6)	8	3.37		
SPD	1	3.57	1.42	NS
(PVMT) x (SPD)	1	.01	.00	NS
Error (8)	2	2.50		
TP	1	3.27	3.85	.10
(PVMT) x (TP)	1	3.45	4.06	.05
(LOAD) x (TP)	2	6.05	7.12	.01
(PVMT) x (LOAD) X (TP)	2	3.03	3.56	.10
(TP) x (AXLE)	2	1.23	1.45	NS
(PVMT) x (TP) x (AXLE)	2	.58	.68	NS
(LOAD) x (TP) x (SPD)	4	1.19	1.40	NS
(LOAD) x (SPD)	2	1.34	1.58	NS
(PVMT) x (LOAD) x (SPD)	2	.15	.18	NS
(TP) x (SPD)	1	1.19	1.40	NS
(PVMT) x (TP) x (SPD)	1	.16	.19	NS
(LOAD) x (TP) x (SPD)	2	2.55	3.00	.10
(AXLE) x (SPD)	2	9.61	11.31	.01
(PVMT) x (AXLE) x (SPD)	2	.12	.14	NS
(LOAD) x (AXLE) x (SPD)	4	3.44	4.05	.01
(TP) x (AXLE) x (SPD)	2	1.13	1.33	NS
Pooled Error	96	.85		

At a significance level of .01, a number of effects and interactions show to be significant. As with the Christison method, load is the most highly significant factor. A plot of the variation of Hutchinson equivalency factors with load is shown in figure 43. Several two factor interactions also show to be significant at .01 level. The Load x Axle interaction is shown in figure 44. Figure 45 shows the Load x Tire Pressure interaction and figure 46 shows the Axle x Speed interaction. This interaction seems to indicate that the tridem axle is more damaging at a slower speed than the single or tandem axle. The one three-way interaction which shows to be significant at the .01 level is Load x Axle x Speed as shown in figure 47. The reason for these Load x Axle two- and three-way interactions is similar for the Hutchinson method as with the Christison method previously described. The one-way analysis of variance is useful to pick out the effects of the load axle type combinations which most greatly influence these results. At the .05 significance level, the Pavement x Tire Pressure two-factor interaction is significant and is shown in figure 48. The same trend is evident in this interaction for the Hutchinson method as was in the Christison method.

It is important to examine the three-way interactions since they explain all the combination effects of the significant variables. Tables 25, 26, and 27 show the three-way interaction tables for the Hutchinson deflection method for calculating load equivalency factors. Table 25 at the .01 significance level is the most important. Because the Hutchinson deflection method ultimately proved to be the best of all four models examined, additional significance testing was accomplished. These tests were all run at a significance level of .05 to see if the equivalency factors were truly different.

Table 25 shows that axle load is significant as previously stated. For example, a single axle medium load 18-kip (8,172-kg) is about 1.0 regardless of speed. The nested significance listing below the table shows that the .05 significance level 1.11 and 0.99 are not significantly different. These two equivalence factors are not significantly different than all the equivalence factors for lower loads on all three axle types 0.66, 0.43, 0.13, 0.08, 0.07, and 0.04. On the other end of this analysis table, the tridem axle high load 60-kip (27,240-kg) equivalence factor of 8.03 at 45-mi/h (72.5-km/h) is significantly different and higher than all other equivalence factors. At the slower speed of 5-mi/h (8.1-km/h), the equivalence factors for the single axle high load 27-kip (12,258-kg) and tridem axle high load 60-kip (27,240-kg) are 6.26 and 5.18, respectively, and are not significantly different.

Table 26 shows the effects of tire pressure, pavement type, and axle load levels (with the results of all axle types combined). Here the nested significance listing is very clear at a significance level of 0.05. The highest equivalency factor is for the highest loads (27-kip, 44-kip, and 60-kip combined) (12,258-kg, 19,976-kg, and 27,240-kg) on the thinnest pavement. However, tire pressure does not show significance at the high load level if the pavement is stronger. Load level is always definitely different and significant but not across tire pressures of 75- and 110-psi (515- to 760-kPa) or pavement thicknesses 3½-in (89-mm) and 7-in (178-mm). However, for a given pavement thickness and tire pressure, the equivalency factor increases significantly with load having a strong effect.

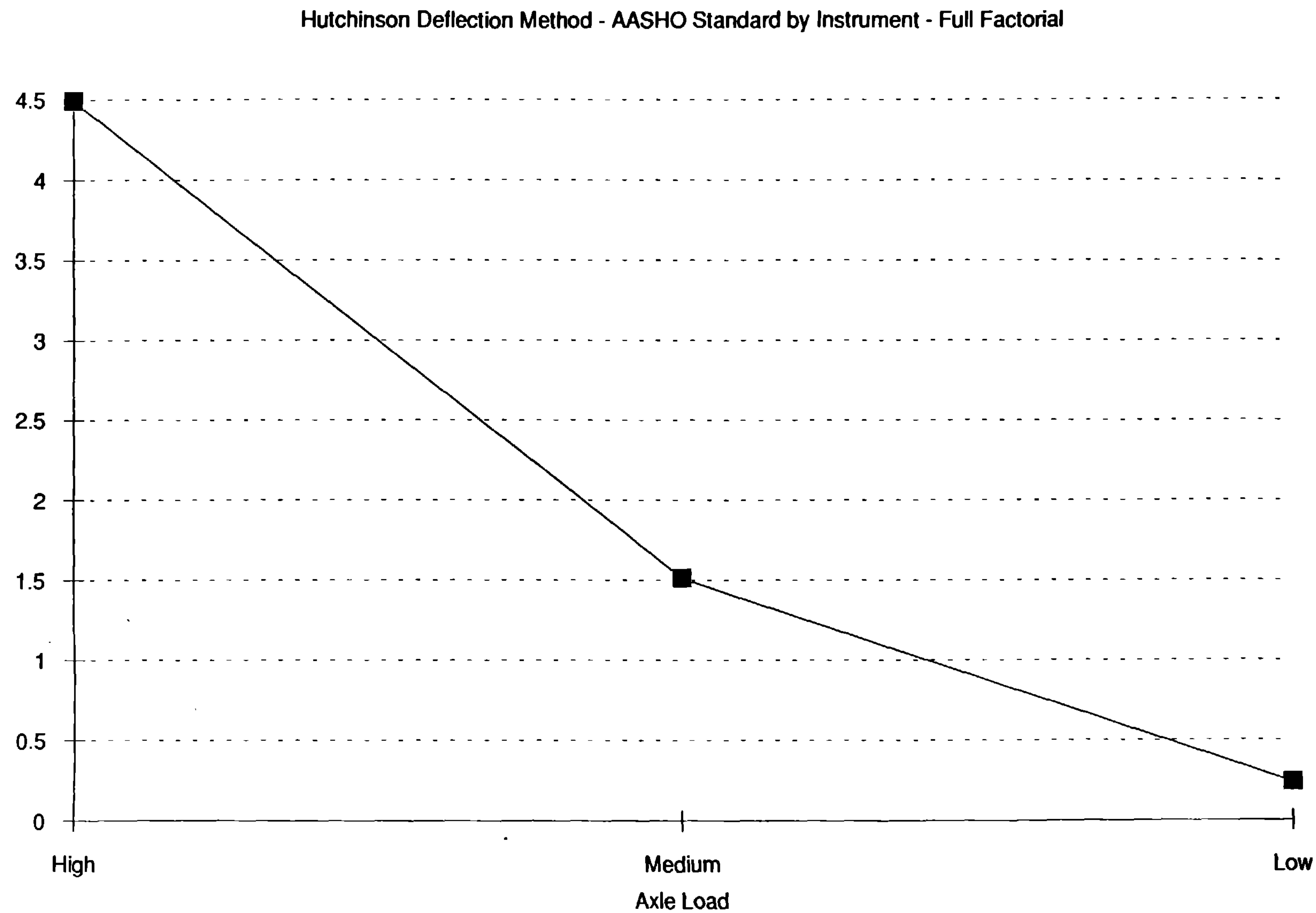


Figure 43. Cell mean plot of the axle load main effect for the Hutchinson deflection method.

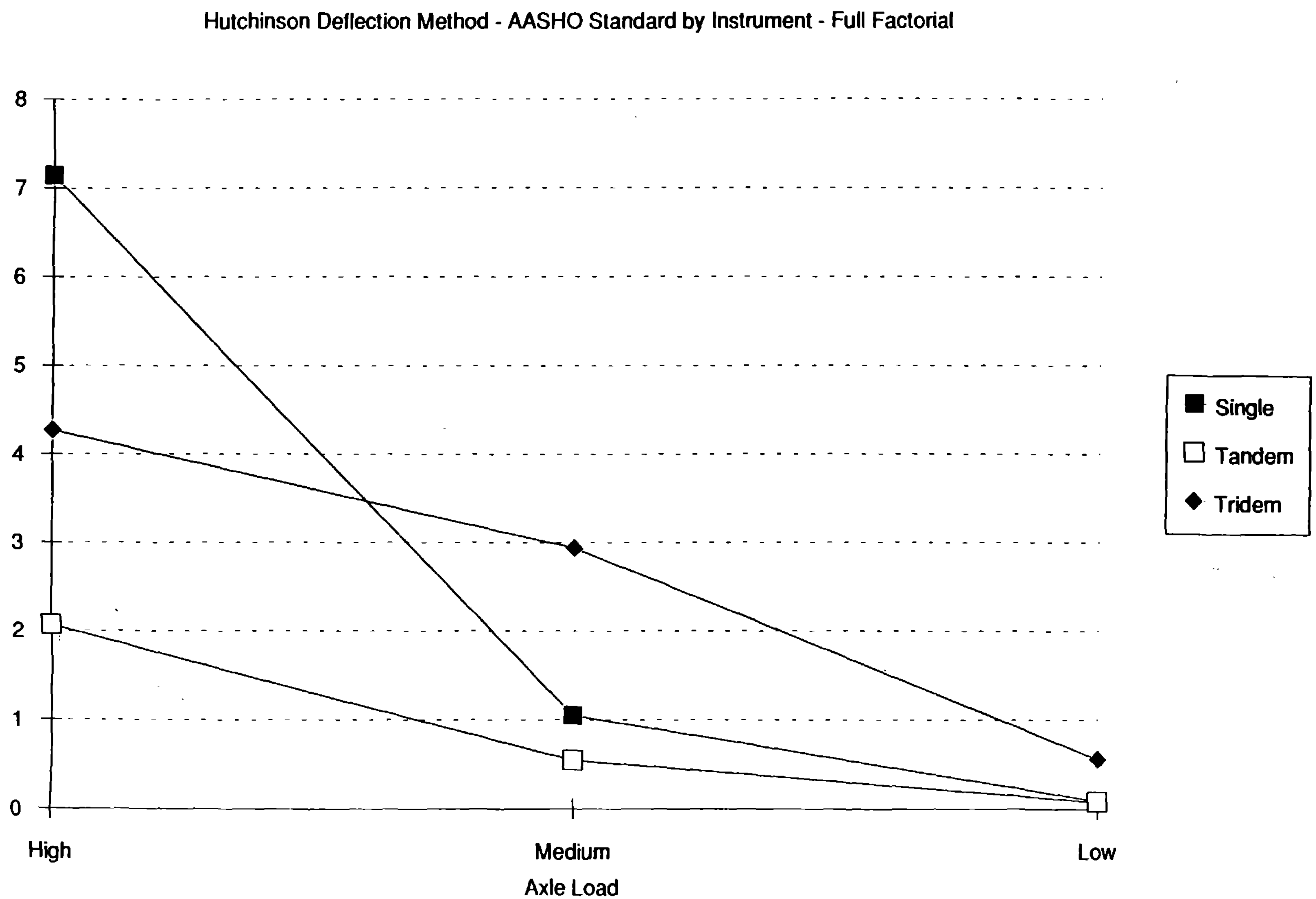
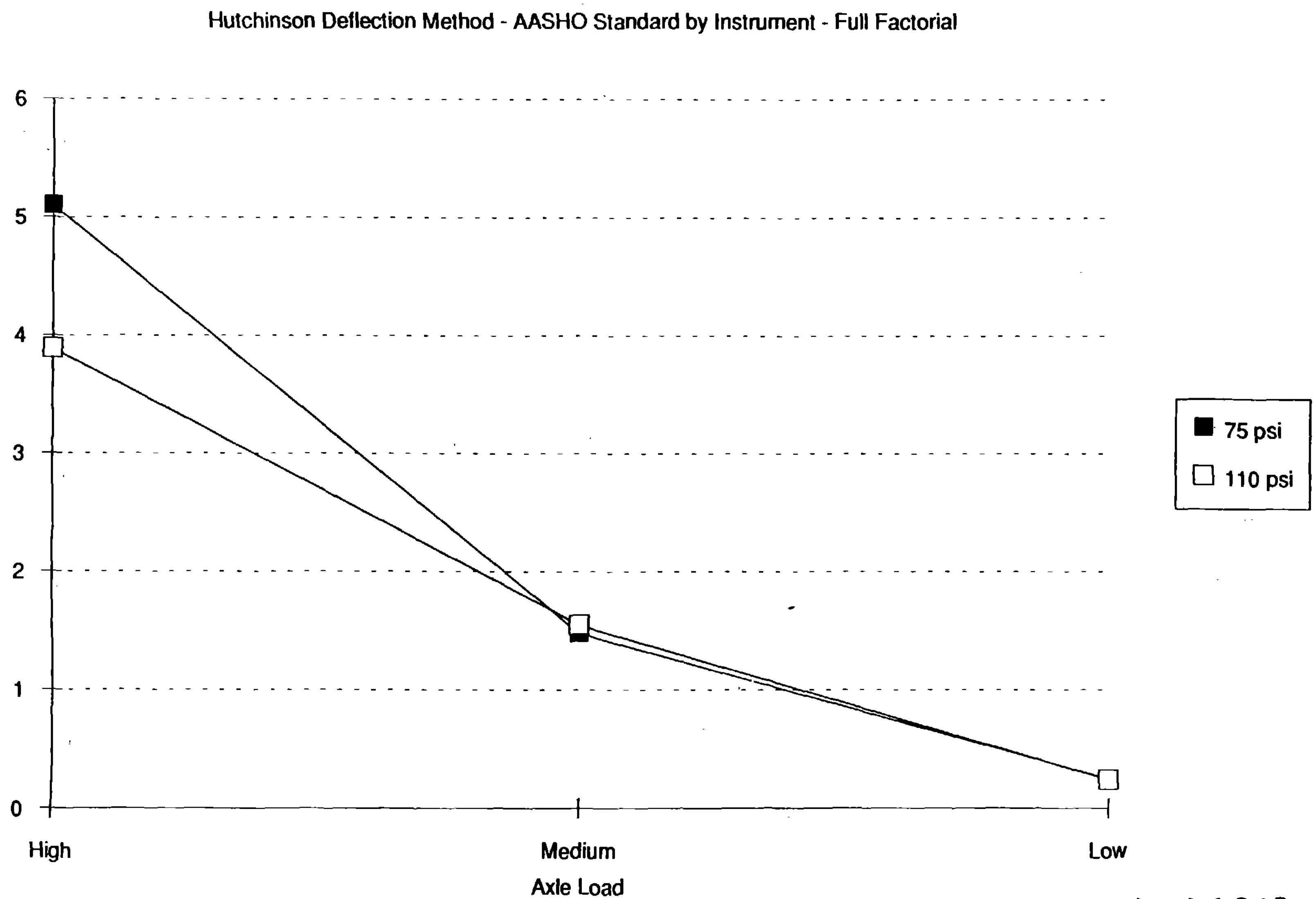


Figure 44. Cell mean plot of the axle load x axle type interaction for the Hutchinson deflection method.



1 psi=6.9 kPa

Figure 45. Cell mean plot of the axle load x tire pressure interaction for the Hutchinson deflection method.

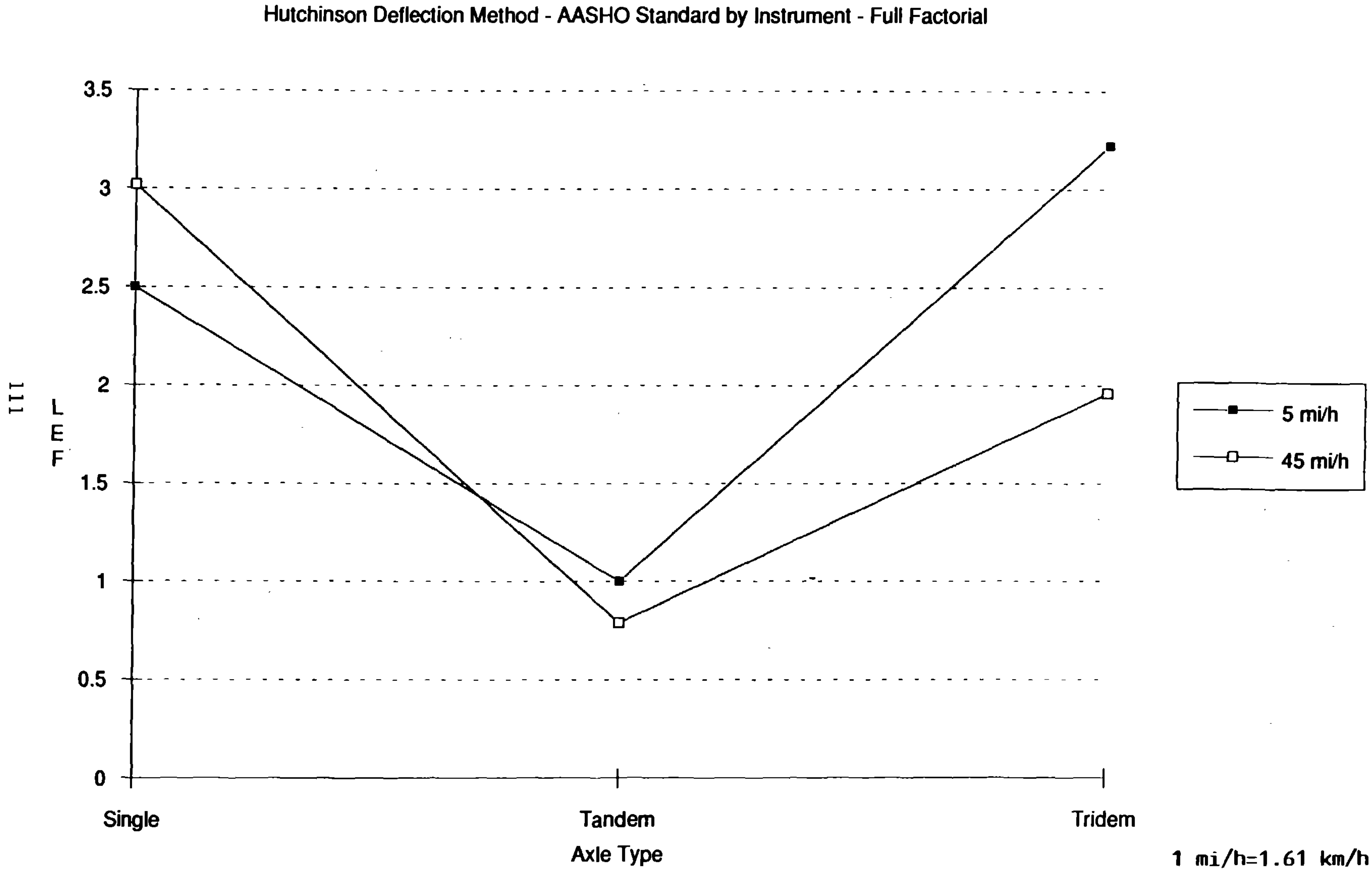


Figure 46. Cell mean plot of the axle type x speed interaction for the Hutchinson deflection method.

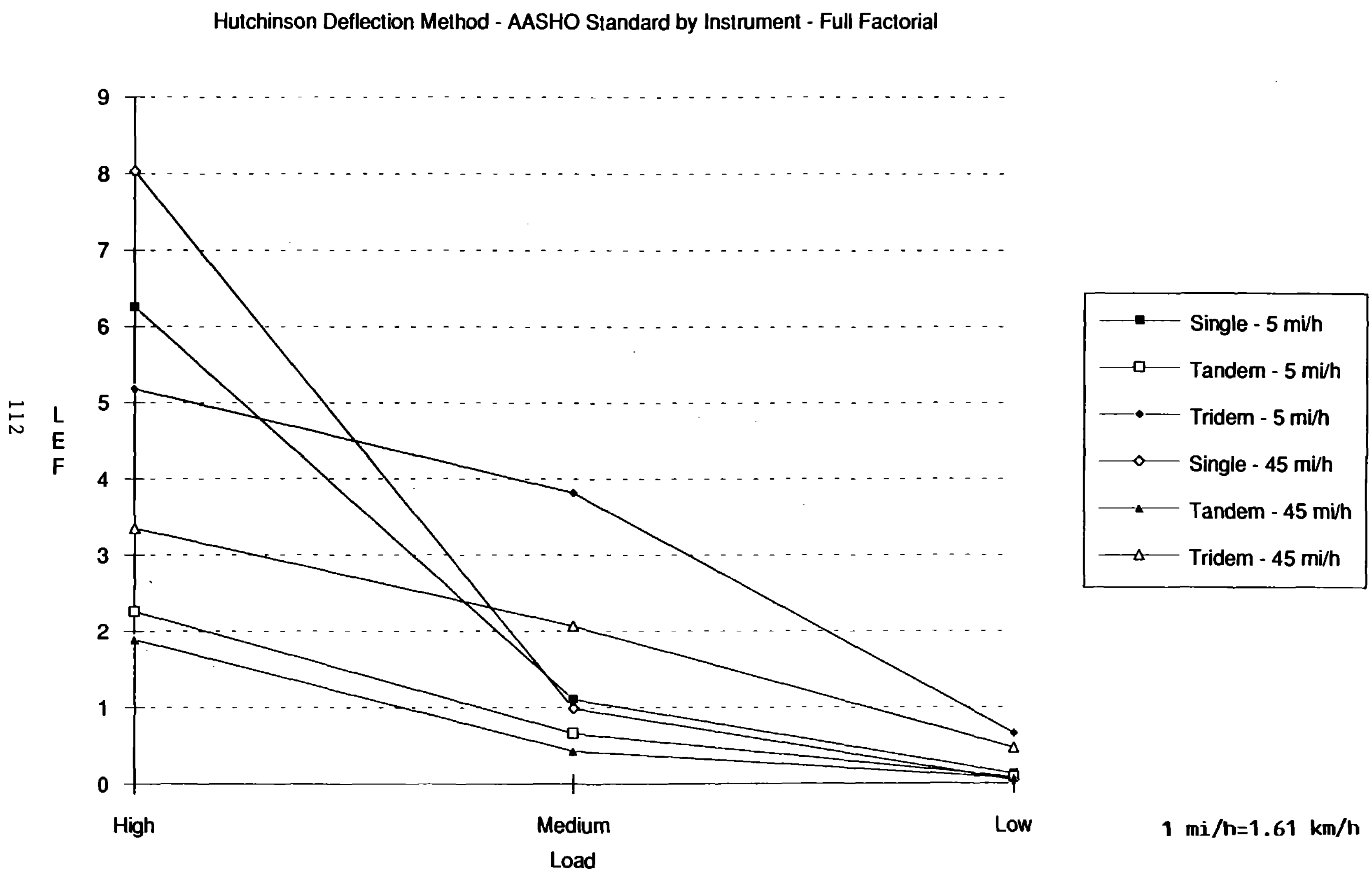
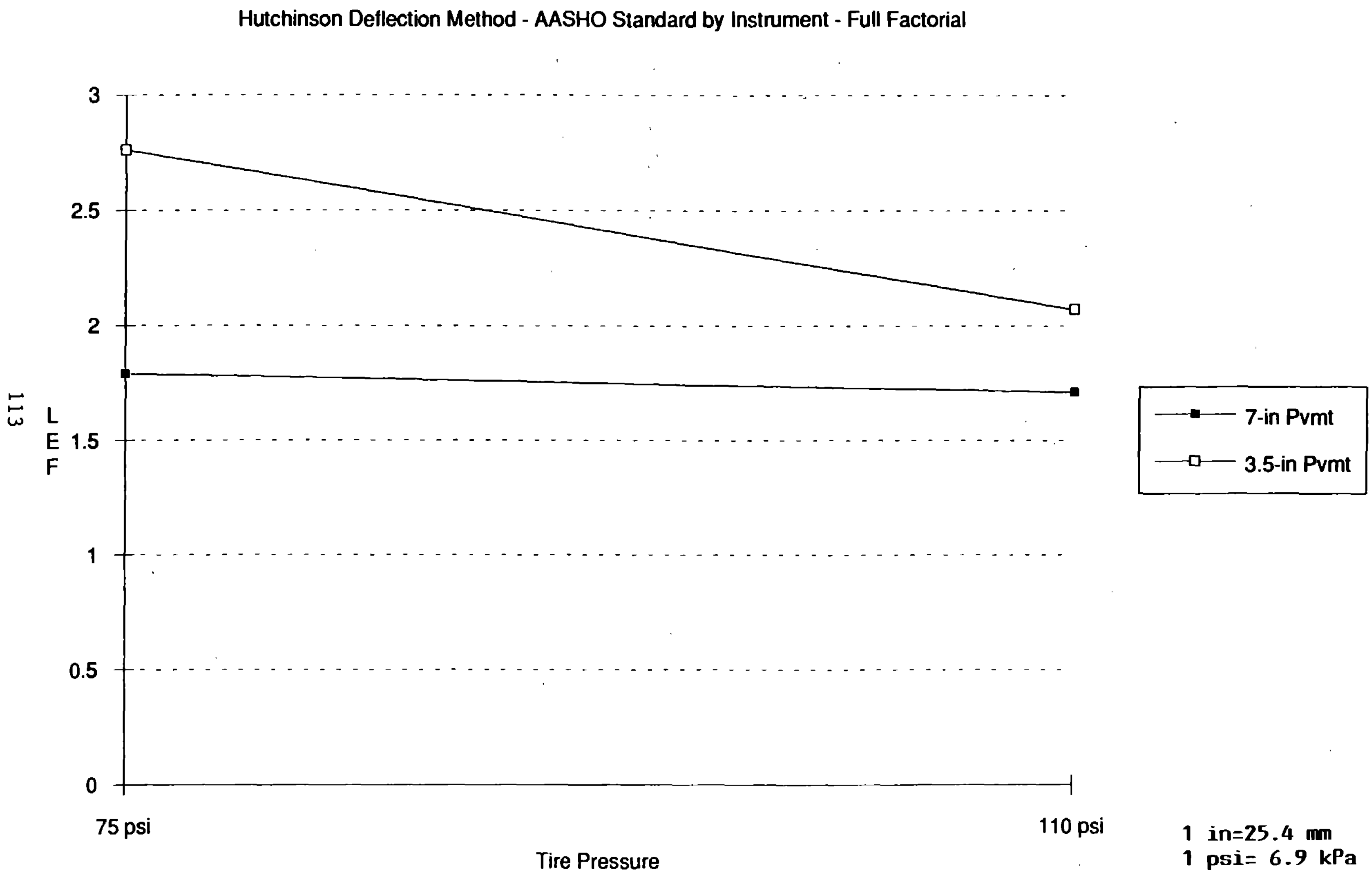


Figure 47. Cell mean plot of the axle load x axle type x speed three-way interaction for the Hutchinson deflection method.



1 in=25.4 mm
1 psi= 6.9 kPa

Figure 48. Cell mean plot of the tire pressure x pavement strength two-way interaction for the Hutchinson deflection method.

Table 25. Three-way interaction table for Hutchinson deflection method. equivalency factors at a significance level of .01.

(AXLE LOAD) x (AXLE TYPE) x (SPEED)

		AXLE LOAD	
	1 = High	2 = Medium	3 = Low
SPEED 1 = 5-mi/h (8.1-km/h)			
AXLE TYPES 1 = Single	6.26	1.11	.13
2 = Tandem	2.26	.66	.08
3 = Tridem	5.18	3.82	.66
SPEED 2 = 45-mi/h (72.5-km/h)			
AXLE TYPES 1 = Single	8.03	.99	.04
2 = Tandem	1.89	.43	.07
3 = Tridem	3.35	2.07	.47

Listing for Significance of Differences at α = .05 for all Significance Tests

Each equivalence factor within a bracket is not significantly different than the other factors in that bracket:

8.03
6.26
5.18
3.82
3.35
2.26
2.07
1.89
1.11
.99
.66
.43
.13
.08
.07
.04

Table 26. Three-way interaction table for Hutchinson deflection method equivalency factors at a significance level of .10.

(PAVEMENT TYPE) x (AXLE TYPE) x (TIRE PRESSURE)

		AXLE LOAD	
	1 = High	2 = Medium	3 = Low
TIRE PRESSURE 1 = 75-psi (515-kPa)			
PAVEMENT TYPE 1 = 7-in (177-mm)	3.65	1.45	.27
2 = 3½-in (88-mm)	6.55	1.51	.23
TIRE PRESSURE 2 = 110-psi (760-kPa)			
PAVEMENT TYPE 1 = 7-in (177-mm)	3.47	1.41	.26
2 = 3½-in (88-mm)	1.89	1.69	.21

Listing for Significance of Differences at α = .05 for all Significance Tests

Each equivalence factor within a bracket is not significantly different than the other factors in that bracket:

6.55]

4.31]
3.65]
3.47]

1.69]
1.51]
1.45]
1.41]

.27]
.26]
.23]
.21]

Table 27. Three-way interaction table for Hutchinson deflection method equivalency factors at a significance level of .10.

(AXLE LOAD) x (TIRE PRESSURE) x (SPEED)

		TIRE PRESSURE	
		1 = 75-psi (515-kPa)	2 = 110-psi (760-kPa)
SPEED 1 = 5-mi/h (8-km/h)			
AXLE LOAD	1 = High	5.55	3.58
	2 = Medium	1.80	1.92
	3 = Low	.29	.30
SPEED 2 = 45-mi/h (72-km/h)			
AXLE LOAD	1 = High	4.65	4.20
	2 = Medium	1.15	1.18
	3 = Low	.21	.17

Listing for Significance of Differences at α = .05 for all Significance Tests

5.55
4.65
4.20
3.58
1.92
1.80
1.18
1.15
.30
.29
.21
.17

Table 27 shows the effects of axle load, tire pressure, and speed. Again, the axle load levels are the combined results of all axle types at each load level. At the high axle load levels for each tire pressure, speed is not significant. For example, the equivalency factors of 5.55 and 4.65 at tire pressures of 75-psi (515-kPa) and speeds of 5-mi/h (8-km/h) and 45-mi/h (72-km/h), respectively, are not significantly different. This is also true at the 110-psi (760-kPa) level and, in fact, at this level the 4.65 factor is not different than 3.58 and 4.20. However, all these factors at the high axle load levels are larger and significantly different than medium and lower load levels.

In summary, the equivalency factors shown in these three tables change in magnitude as would be expected from engineering experience due to pavement strength, speed, tire pressure, and load. The only unexpected trend as previously discussed is that the tandem axle factors are lower than the tridem axle factors as shown in table 25. As shown across all three tables, the factors are many times not significantly different statistically. Increasing the number of variable levels and ranges by more testing could change these results. Table 26 shows without a doubt the strong, clear, undeniable effect of load regardless of tire pressure and pavement strength except on very thin pavements at low pressures when the factor becomes significantly different. These results show the Hutchinson deflection method model to be clear and strong.

Effect of Uneven Load Distribution on Multiaxles

An additional analysis was performed to examine the effects of using the rear tandem axle to calculate the equivalency factors for the tandem vehicle instead of the front tandem axle. The reason for this additional analysis is that the results for the tandem axle vehicle indicated much lower LEF's than for either the single or tridem axle. The additional analysis is to determine if a different set of tandems would produce different results. The truck was loaded such that an attempt was made to make both sets of tandems have equal weights for all weight levels. This was accomplished on the front set of tandems within a reasonable margin of error; however, it was not possible to load the rear set of tandems such that both wheels of the tandem set had the same load. There was a constant difference of 15 to 20 percent for each load level. For this reason, the front set of tandems was used in the original analysis.

The results of these additional analyses point out an important aspect of the Hutchinson Deflection Method. Figures 49 and 50 show the effects of the different axle types for both the Christison and Hutchinson methods. In the Christison method, a similar trend was obtained regardless of the set of tandem axles used. The tandem LEF's were significantly lower than the single and tridem. The Hutchinson method, however, shows that the unbalanced rear tandem set produced a much higher LEF than the balanced front set of tandems. As can be seen from figure 50, the LEF's were more comparable to the single and tridem values. The reason the Hutchinson method shows the difference is because of the way it accounts for each individual fatigue cycle from the individual wheels of the tandem set as discussed earlier in this report. The Hutchinson method accounts for the much higher level of response from the unbalanced second wheel of the tandem axle group. The Christison method on the other hand does not fully account for the higher response of the second wheel. These observa-

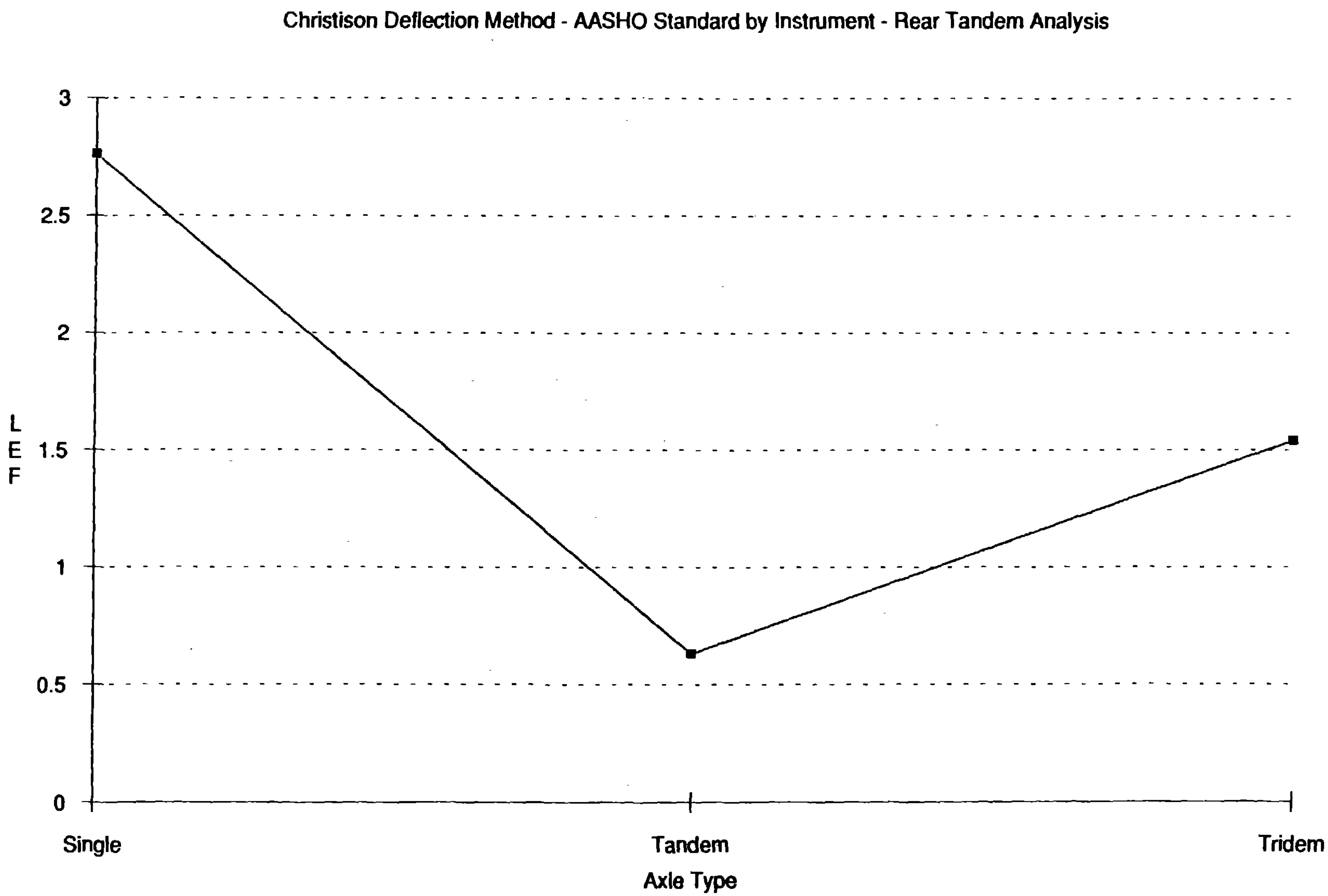


Figure 49. Cell mean plot of the different axle types effect for the Christison deflection method.

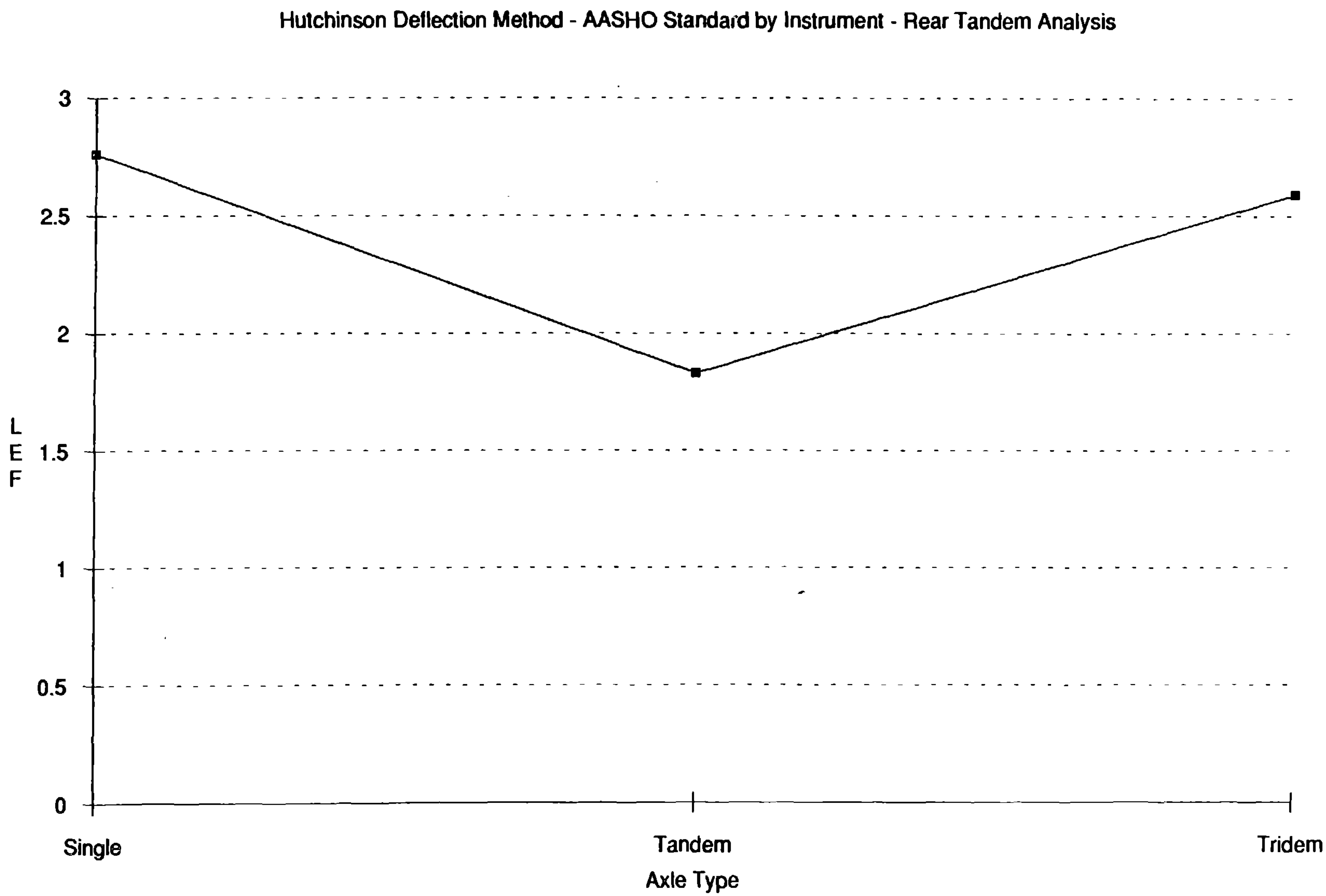


Figure 50. Cell mean plot of the different axle types effect for the Hutchinson deflection method.

tions from the additional analyses are further support for the selection of the Hutchinson deflection method as the most reasonable for continued analyses and considerations in primary response LEF's.

Viability of Deflection Based Load Equivalency Factor Methods

It is apparent from the analysis presented above that the two methods for deflection based load equivalency factor calculation using primary pavement response measurements are viable for estimating load equivalence. If the AASHO Road Test is used as a basis for providing the most accurate estimates of load equivalency factors, the Hutchinson method would be considered the best of the two methods for use in practice in the United States. Since both methods can predict load equivalency factors with reasonable accuracy, the idea of using the methods to develop a full LEF model is viable. Such a model could take into account the effects increasing tire pressure, variable vehicle speeds, different truck and axle configurations, different suspension and dynamic characteristics, or a wide range of other vehicle, pavement structural, or environmental characteristics which in some way influence the factors. The three-way interaction results presented show the relationships that can be developed. This experiment clearly indicates that some of these factors do not need to be considered in such an overall model when estimating load equivalency factors for design or cost allocation purposes. The ramifications of some of these conclusions are discussed in the next section on conclusions and research recommendations. It also appears as will be described in the next section on strain measurements, that based on the results of this project, the deflection based procedures are more reliable and accurate than are the strain based methods.

STRAIN BASED METHODS OF LOAD EQUIVALENCY FACTORS

Two methods were selected for calculation of load equivalency factors using primary pavement strain responses. The first is a method proposed by Christison, which is closely related to the deflection method discussed in the previous section.[2] The other method was proposed by Southgate and uses a strain energy concept for estimation of load equivalency factors.[8] Strain energy is related to tensile strain which was measured on this project and is used to calculate the equivalency factors. A description of both methods is given in detail in appendix A.

Christison's strain based method uses the strain profile of an axle group to calculate the equivalency factor just as was done in the deflection based method.[2] However, since the strain profile almost always rebounds back to or past zero strain from a positive tensile strain after the passage of each wheel of a multiaxle group, there was no way to modify the Christison strain method as was proposed by Hutchinson for the deflection method. Thus, Christison's strain method takes more fully into account the effects of each wheel of a multiple axle group than does his deflection method.

Comparison of Strain Methods to AASHTO Load Equivalency Factors

As with the deflection methods, it is logical to compare the results of the strain based methods to the load equivalency factors developed from the AASHO Road Test. Table 28 shows an indication of the actual load

Table 28. Comparison of strain based primary response LEF methods with AASHTO factors.

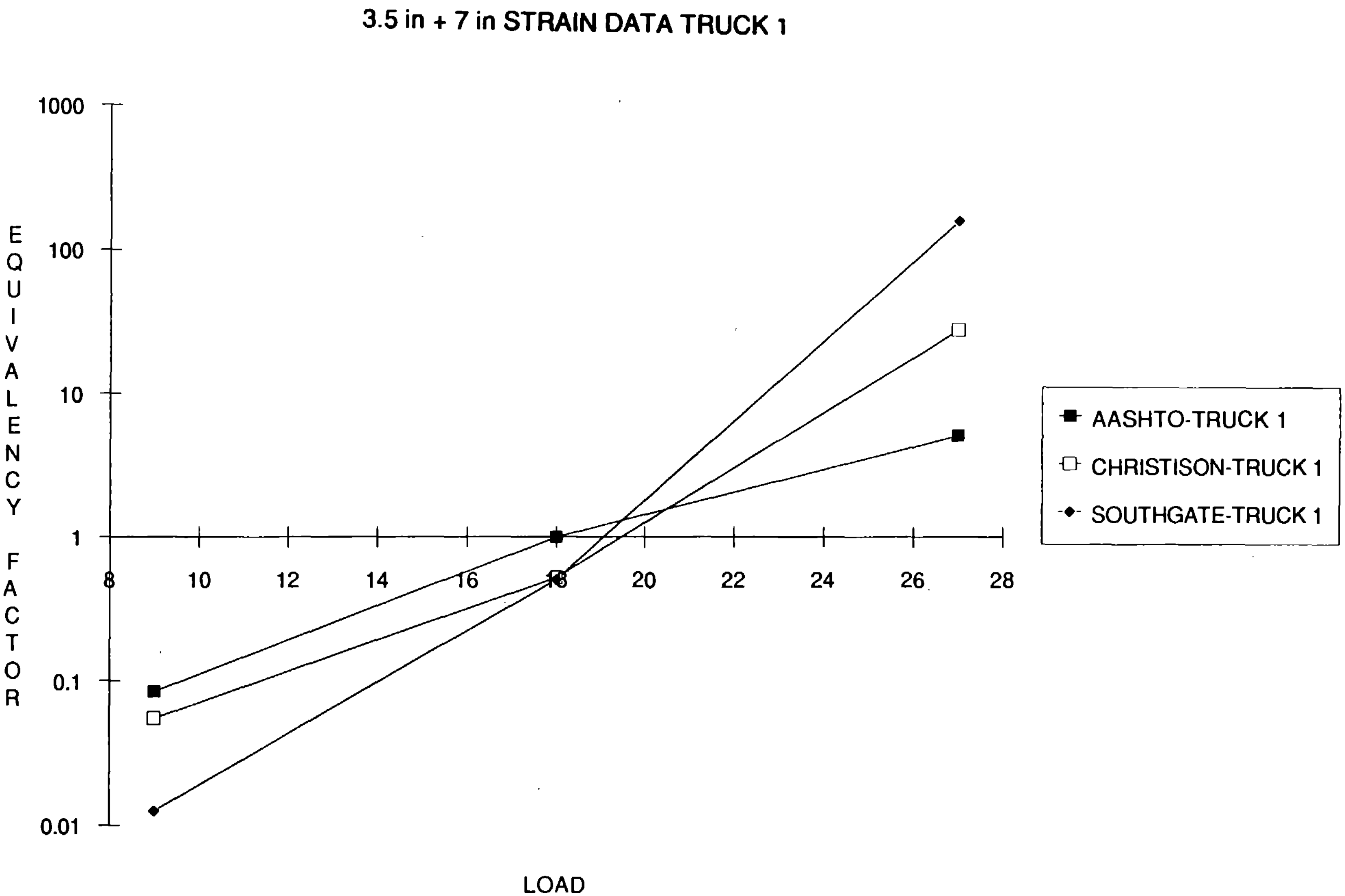
		CHRIST. PAVE	SOUTH. PAVE		% DIFF	%DIFF
TRUCK	LOAD	AVERAGE	AVERAGE	AASHTO	CHRISTISON	SOUTHGATE
1	27	27.85	159.99	5.11	445.45	3033.94
1	18	0.52	0.51	1.00	-48.00	-49.50
1	9	0.06	0.01	0.08	-34.91	-85.21
2	44	1.69	1.89	2.99	-43.48	-36.71
2	32	0.33	0.15	0.89	-63.16	-83.13
2	20	0.09	0.03	0.16	-45.99	-84.57
3	60	7.45	14.57	2.48	200.30	487.50
3	42	7.46	13.97	0.66	1027.84	2013.46
3	24	2.26	2.15	0.08	2590.48	2459.52

equivalency factor values from each pavement section and each instrument at 45-mi/h (72-km/h) vehicle speed and 75-psi (515-kPa) tire pressure. These are the conditions that most closely represent the AASHO Road Test conditions for which the AASHTO factors were developed. Figures 51 through 53 show the plots of the average equivalency factors determined from this study versus the AASHTO factors. It is apparent from examining this data and plots that neither of the strain methods were as good as the deflection methods for estimating AASHO Road Test factors. In fact, some of the strain results are an order of magnitude or more different than the AASHO factors. The variability of the strain results between instruments and pavements is larger than the deflection results and thus may contribute to the differences. It is interesting to note that although the strain methods produce a higher estimate of load equivalency factors for single axle loads over the 18,000-lb (8,172-kg) standard, this may actually represent the reality of what is occurring in terms of strain. Whether this represents what actually occurs in terms of pavement damage is another question. It is possible that when a strain method is used the exponent on the power function commonly known as the fourth power law may be somewhat lower than the 3.8 used on this project in the Christison method. The Southgate method although similar to the Christison method at lower levels of load equivalency factors, tended to indicate very extreme values of LEF in some cases. For single axles, this was as much as an order of magnitude higher than Christison's strain method which in turn was already an order of magnitude higher than the AASHTO method. For tandem axles, however, both primary response methods predicted lower values than the AASHTO method.

This was the same trend that was observed with the deflection based methods and may have something to do with the use of the center set of tandems on the truck rather than the rear tandems. Of the two strain methods evaluated, the Christison method seems to be the more reasonable. When compared to either deflection method discussed previously, however, it does not predict the AASHTO level of equivalency factors nearly as well. It is possible that the Christison strain method can produce an accurate set of load equivalency factors if better control of vehicle lateral position and increased replication of the strain measurements, as discussed earlier in this report, were exercised during testing. However, it is likely that, in general, the strain method will tend to produce higher load equivalency factors than the deflection based methods or the AASHTO factors.

Effects of Experimental Factors on Strain Methods

The strain methods do not provide very good information as to what vehicle or structural parameters most greatly affect the load equivalency factor calculations. The variations of instruments in the strain methods was larger than for the deflection methods and have made it difficult to interpret much useful information. In the Southgate method, even load showed to be significant only at a .25 significance level. This is an extremely poor level of confidence for a factor which is known to so greatly influence load equivalence factors.



1 in=25.4 mm

Figure 51. Plot of strain based equivalency methods versus the AASHTO factors for single axles.

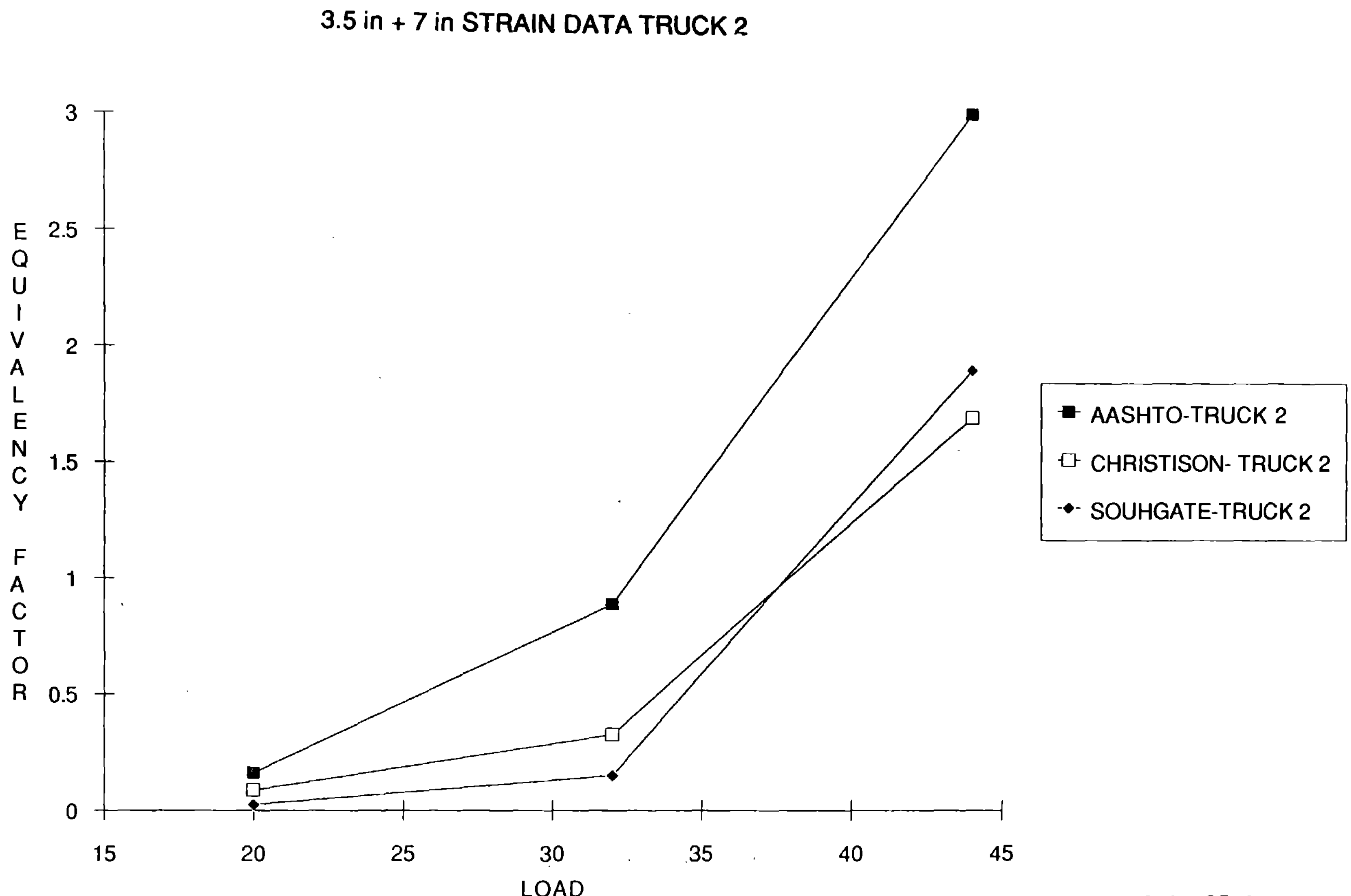


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Figure 52. Plot of strain based equivalency methods versus the AASHTO factors for tandem axles.

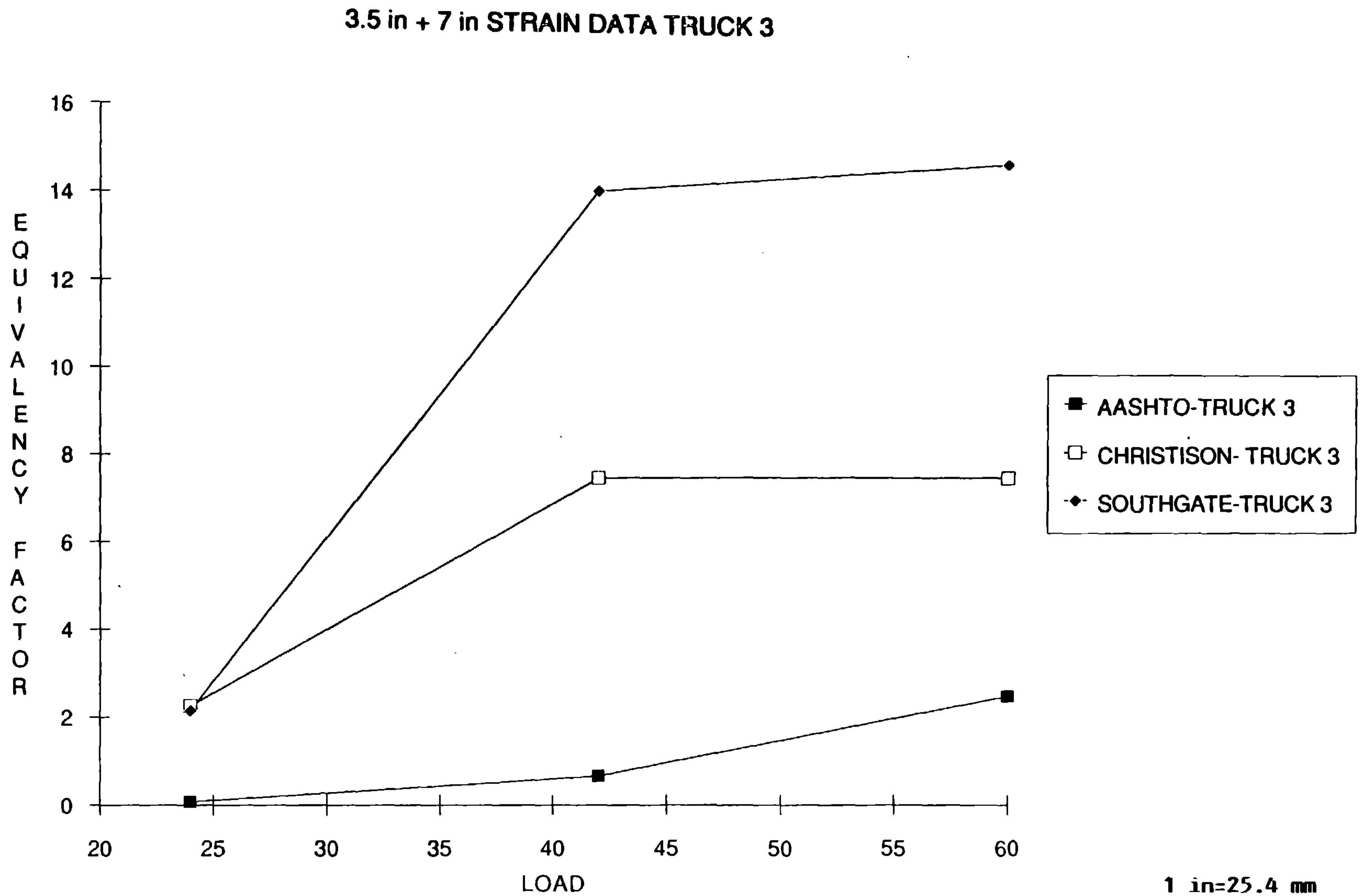


Figure 53. Plot of strain based equivalency methods versus the AASHTO factors for tridem axles.

Christison Strain Based Load Equivalency Factors

The first level corrected ANOVA for the Christison strain method is shown in table 29.

The pooled error mean square is calculated as shown in table 30.

The fact that a fewer number of the error terms pooled for this strain method than for the deflection methods indicates a larger variability of the factor "instruments" for the strain gauges than for the deflection gauges. The pooled error is then used to retest the effects and interactions which are associated with the error terms that were pooled. The resulting final corrected ANOVA for the Christison strain method is shown in table 31.

From this corrected ANOVA the significant factors and interactions are as follows:

At α = .10
- LOAD
- (LOAD) x (AXLE)
- (TP) x (AXLE)
- (PVMT) x (SPD)

At α = .01
- SPD

It is apparent that the strain based Christison method is not as sensitive to the main effects or interactions as either of the deflection methods discussed above. In fact, the main effect "Load" is only significant at the .10 level. It is interesting to note, however, that speed shows to be significant at the .01 level. Speed did not show to be significant, as a main effect, at all in the deflection based methods. This seemed to indicate that the time of loading is significant when strains are being measured. Also, the fact that one of the two factor interactions significant at the .10 level is Pavement x Speed indicates that the thickness of a surface also has an effect on the strain response time. The slow moving vehicle had time to fully develop the strain at the bottom of the asphalt layer than the faster moving vehicle did. A plot of the main effect of speed is shown in figure 54. Figure 55 shows the two-way interaction of Pavement x Speed which shows that these effects are much more pronounced on the thick pavement section than the thin.

Southgate Strain Based Load Equivalency Factors

The first level corrected ANOVA for the Christison strain method is shown in table 32. From this table, several of the error terms can be pooled with the residual error term based on a significance level of .25. The pooled error mean squares are calculated as shown in table 33.

Thus, the final corrected ANOVA using the pooled error term is shown in table 34 for the Southgate strain method.

Table 29. First level corrected ANOVA - Christison strain method.

Christison Strain Equivalency Factor Method
Full Factorial - AASHTO Standard - By Instrument

Corrected ANOVA using the interactions with instruments (INST) to make the F tests

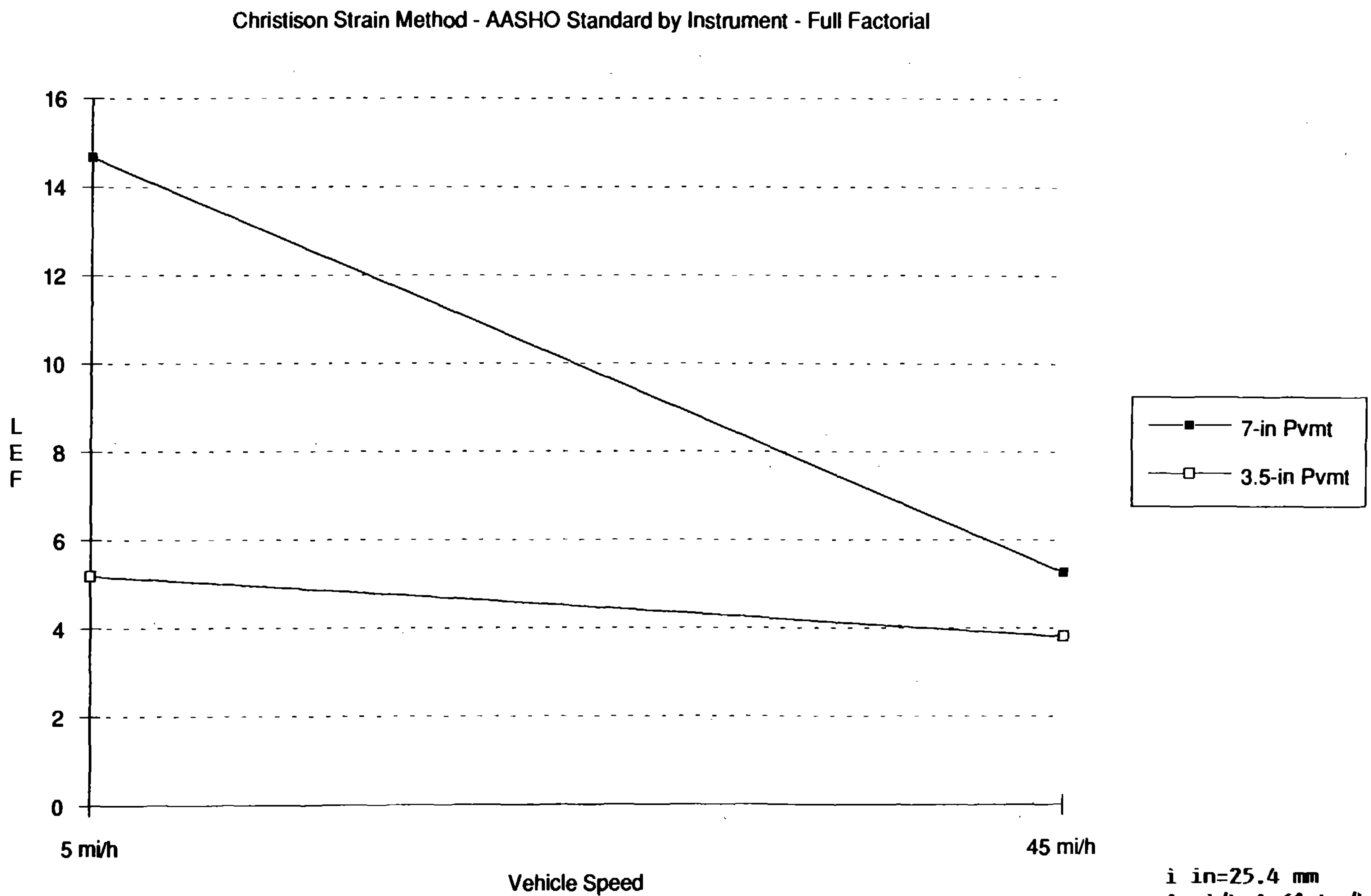
SOURCE	df	MS	F	Sig of F
TEMP	1	1,903	3.02	.25
PVMT	1	1,595	2.53	NS
Error (1)	2	630		
LOAD	2	3,384	5.70	.1
(PVMT)x(LOAD)	2	213	0.36	NS
Error (2)	4	593		
TP	1	163	1.59	NS
(PVMT)x(TP)	1	269	2.62	.25
Error (3)	2	103		
(LOAD)x(TP)	2	602	2.80	.25
(PVMT)x(LOAD)x(TP)	2	22	0.10	NS
Error (4)	4	215		
AXLE	2	1,815	2.19	.25
(PVMT)x(AXLE)	2	167	0.20	NS
Error (5)	4	829		
(LOAD)x(AXLE)	4	1,904	3.51	.1
(PVMT)x(LOAD)x(AXLE)	4	65	0.12	NS
Error(6)	8	543		
(TP)x(AXLE)	2	584	3.12	.1
(PVMT)x(TP)x(AXLE)	2	83	0.44	NS
(LOAD)x(TP)x(AXLE)	4	472	2.52	.25
Error (7)	8	187		
SPD	1	999	6.68	.25
(PVMT)x(SPD)	1	385	2.57	.25
Error (8)	2	150		
(LOAD)x(SPD)	2	441	1.27	NS
(PVMT)x(LOAD)x(SPD)	2	83	0.24	NS
Error(9)	4	347		
(TP)x(SPD)	1	109	1.02	NS
(PVMT)x(TP)x(SPD)	1	53	0.50	NS
(LOAD)x(TP)x(SPD)	2	66	0.62	NS
Error (10)	4	106		
(AXLE)x(SPD)	2	421	1.66	.25
(PVMT)x(AXLE)x(SPD)	2	41	0.16	NS
(LOAD)X(AXLE)x(SPD)	4	626	2.47	.25
(TP)x(AXLE)x(SPD)	2	83	0.33	NS
Error (11)	10	254		
Residual	56	129		

Table 30. Pooled error calculation for Christison strain method.

SOURCE	df	MS	SS
Error (3)	2	102.6	205.2
Error (8)	2	149.7	299.4
Error (10)	4	106.1	424.4
Residual	56	129.0	7224.0
Pooled Error	64	127.4	8153.0

Table 31. Final corrected ANOVA - Christison strain method.

SOURCE	df	MS	F	Significance
TEMP	1	1903.1	3.0	NS
PVMT	1	1594.8	2.5	NS
Error (1)	2	629.8		
LOAD	2	3384.3	5.7	.10
(PVMT) x (LOAD)	2	212.8	.4	NS
Error (2)	4	593.2		
(LOAD) x (TP)	2	602.0	2.8	NS
(PVMT) x (LOAD) x (TP)	2	22.1	.1	NS
Error (4)	4	214.7		
AXLE	2	1814.9	2.2	NS
(PVMT) x (AXLE)	2	167.1	.2	NS
Error (5)	4	829.0		
(LOAD) x (AXLE)	4	1903.7	3.5	.10
(PVMT) x (LOAD) x (AXLE)	4	65.0	.1	NS
Error (6)	8	542.7		
(TP) x (AXLE)	2	584.4	3.1	.10
(PVMT) x (TP) x (AXLE)	2	83.0	.4	NS
(LOAD) x (TP) x (AXLE)	4	471.8	2.5	NS
Error (7)	168	187.1		
(LOAD) X (SPD)	2	441.5	1.3	NS
(PVMT) x (LOAD) x (SPD)	2	83.1	.2	NS
Error (9)	4	346.5		
(AXLE) x (SPD)	2	420.9	1.7	NS
(PVMT) x (AXLE) x (SPD)	2	40.7	.2	NS
(LOAD) x (AXLE) x (SPD)	4	626.1	2.5	NS
(TP) x (AXLE) X (SPD)	2	83.2	.3	NS
Error (11)	10	253.6		
TP	1	163.5	1.3	NS
(PVMT) x (TP)	1	269.0	2.1	NS
SPD	1	999.5	7.9	.01
(PVMT) x (SPD)	1	385.2	3.0	.10
(TP) x (SPD)	1	108.7	< 1	NS
(PVMT) x (TP) x (SPD)	1	53.3	< 1	NS
(LOAD) x (TP) x (SPD)	2	66.2	< 1	NS
Pooled Error	64	127.4		



1 in=25.4 mm
1 mi/h=1.61 km/h

Figure 54. Cell mean plot of vehicle speed main effect for the Christison strain method.

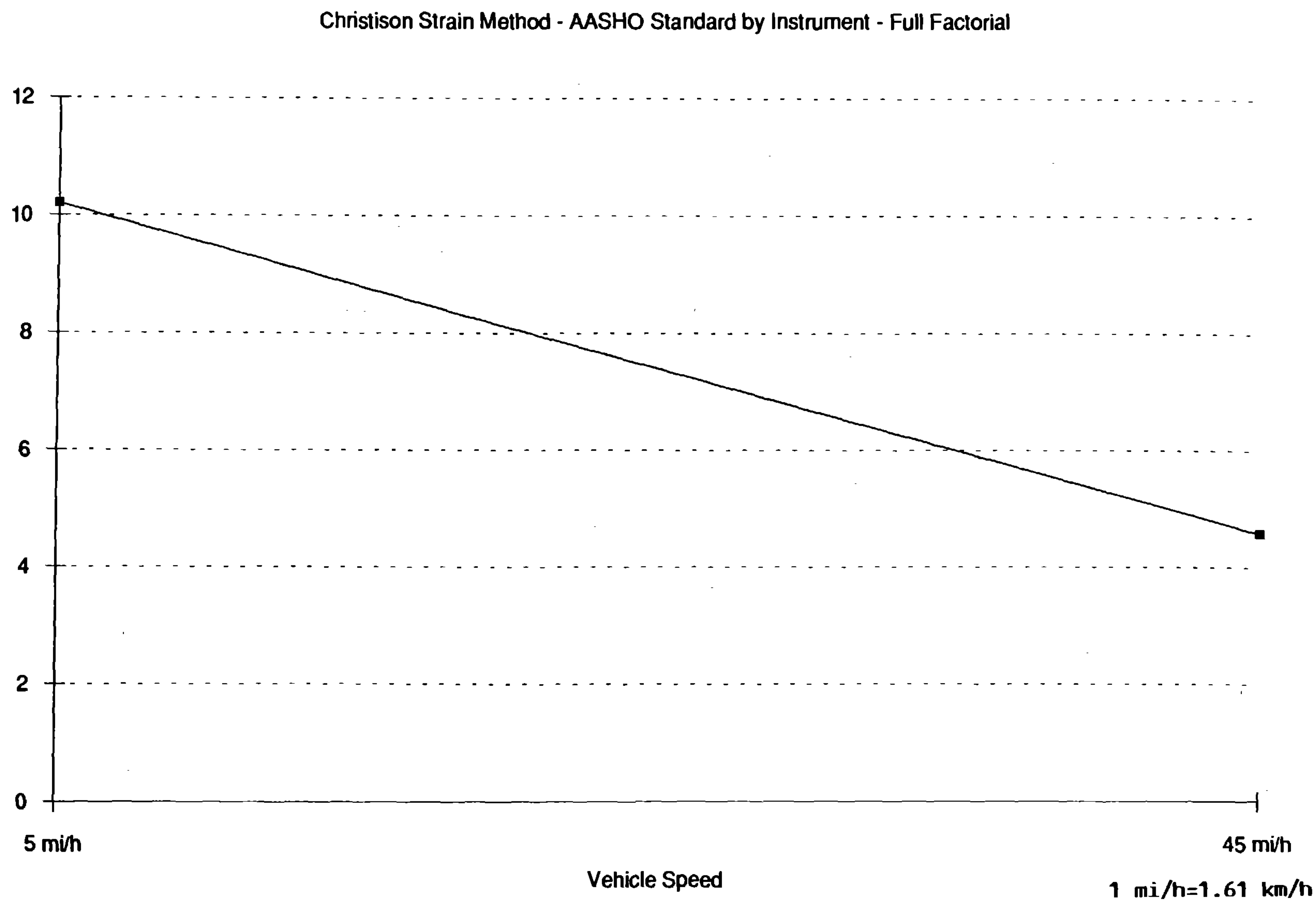


Figure 55. Cell mean plot of vehicle speed x pavement interaction for the Christison strain method.

Table 32. First level corrected ANOVA - Southgate strain method.

Southgate Strain Equivalency Factor Method
Full Factorial - AASHTO Standard - By Instrument

Corrected ANOVA using the interactions with instruments (INST) to make the F tests

SOURCE	df	MS	F	Sig of F
TEMP	1	38,143	0.60	NS
PVMT	1	158,702	2.49	NS
Error (1)	2	63,806		
LOAD	2	248,414	2.99	.25
(PVMT)x(LOAD)	2	31,847	0.38	NS
Error (2)	4	83,172		
TP	1	37,531	1.14	NS
(PVMT)x(TP)	1	24,257	0.74	NS
Error (3)	2	32,829		
(LOAD)x(TP)	2	80,010	1.68	NS
(PVMT)x(LOAD)x(TP)	2	3,090	0.06	NS
Error (4)	4	47,739		
AXLE	2	259,089	2.88	.25
(PVMT)x(AXLE)	2	28,343	0.31	NS
Error (5)	4	90,082		
(LOAD)x(AXLE)	4	175,715	1.86	.25
(PVMT)x(LOAD)x(AXLE)	4	5,787	0.06	NS
Error(6)	8	94,347		
(TP)x(AXLE)	2	87,024	2.24	.25
(PVMT)x(TP)x(AXLE)	2	10,460	0.27	NS
(LOAD)x(TP)x(AXLE)	4	81,110	2.09	.25
Error (7)	8	38,863		
SPD	1	82,368	2.13	NS
(PVMT)x(SPD)	1	43,336	1.12	NS
Error (8)	2	38,733		
(LOAD)x(SPD)	2	79,084	1.11	NS
(PVMT)x(LOAD)x(SPD)	2	14,863	0.21	NS
Error(9)	4	71,039		
(TP)x(SPD)	1	29,095	1.01	NS
(PVMT)x(TP)x(SPD)	1	12,473	0.43	NS
(LOAD)x(TP)x(SPD)	2	27,494	0.96	NS
Error (10)	4	28,676		
(AXLE)x(SPD)	2	85,305	1.78	.25
(PVMT)x(AXLE)x(SPD)	2	6,453	0.13	NS
(LOAD)X(AXLE)x(SPD)	4	113,598	2.37	.25
(TP)x(AXLE)x(SPD)	2	31,520	0.66	NS
Error (11)	10	48,024		
Residual	56	29,602		

Table 33. Pooled error calculation for Southgate strain method.

SOURCE	df	MS	SS
Error (3)	2	32,829	65,658
Error (7)	8	38,863	299.4
Error (8)	2	38,733	77,466
Error (10)	4	28,676	114,704
Residual	56	29,602	1,657,712
Pooled Error	64	30,922	2,226,444

Table 34. Final corrected ANOVA - Southgate strain method.

SOURCE	df	MS	F	Significance
TEMP	1	38,143	< 1	NS
PVMT	1	158,702	2.5	NS
Error (1)	2	63,806		
LOAD	2	248,414	3.0	NS
(PVMT) x (LOAD)	2	31,847	< 1	
Error (2)	4	83,172		
(LOAD) x (TP)	2	80,010	1.7	NS
(PVMT) x (LOAD) x (TP)	2	3,090	< 1	NS
Error (4)	4	47,739		
AXLE	2	259,089	2.9	NS
(PVMT) x (AXLE)	2	28,343	< 1	NS
Error (5)	4	90,082		
(LOAD) x (AXLE)	4	175,715	1.9	NS
(PVMT) x (LOAD) x (AXLE)	4	5,787	< 1	NS
Error (6)	8	94,347		
(LOAD) X (SPD)	2	79,084	1.1	NS
(PVMT) x (LOAD) x (SPD)	2	14,863	< 1	NS
Error (9)	4	71,039		
(AXLE) x (SPD)	2	85,305	1.8	NS
(PVMT) x (AXLE) x (SPD)	2	6,453	< 1	NS
(LOAD) x (AXLE) x (SPD)	4	113,598	2.4	NS
(TP) x (AXLE) X (SPD)	2	31,520	< 1	NS
Error (11)	10	48,024		
TP	1	163.5	1.3	NS
(PVMT) x (TP)	1	269.0	2.1	NS
SPD	1	999.5	7.9	.01
(PVMT) x (SPD)	1	385.2	3.0	.10
(TP) x (SPD)	1	108.7	< 1	NS
(PVMT) x (TP) x (SPD)	1	53.3	< 1	NS
(LOAD) x (TP) x (SPD)	2	66.2	< 1	NS
Pooled Error	64	127.4		

From the final corrected ANOVA, the significant factors and interactions are as follows:

At α = .10
- (TP) x (AXLE)
- (LOAD) x (SPD)

At α = .05
- (LOAD) x (TP) x (AXLE)

The Southgate strain based method for load equivalency factors shows the least sensitivity of all the methods to the factors under study. No factors or interactions were significant at the .01 level. Only one three-factor interaction was significant at the .05 level, and two two-factor interactions at the .10 level. It is apparent that the large variation in strain measurements coupled with the method used to calculate equivalency factors are producing unreliable results. Since the factor "Load" did not show to be significant at all, there are definitely some problems associated with this method. It is possible that better control over the strain measurements and lateral position of the vehicles and increased replication could produce more reasonable results using this method. However, the strain data obtained on this project does not show any reasonable trends. If strains are predicted using computer algorithms as was done when the method was developed by Southgate, reasonable trends could be obtained. This is because the computer generated strain responses do not have the inherent variation associated with the actually measured values.

Viability of Strain Based Load Equivalency Factor Methods

The strain data from this project was too varied and had too many missing cells to provide indications that strain based methods are viable for use in estimating load equivalency factors from primary pavement response measurements. However, if better control and increased replication of strain measurements as described elsewhere in this report is achieved, then the strain methods could become more viable in predicting load equivalency factors.

SUMMARY OF RESULTS DISCUSSION

The results from the experiments run on this project and the detailed data analyses on the data from those experiments are discussed in this section. Quantification of the variability of the instruments which measured pavement response is key to the interpretation of the results. Once the instrument variation is quantified, these error terms can be used to analyze the remainder of the response data in order to determine which factors are significant in estimating primary response load equivalency factors and which LEF method is the most reasonable for use in further studies to develop detailed models of primary response equivalency. Analysis of variance was performed on the data from each of the methods, both strain and deflection based. Comparison of the results of each method to the AASHTO equivalency factors shows that the deflection based methods are far more reasonable in estimating the commonly accepted AASHTO values. The next section presents the conclusions and recommendations from the detailed discussions of the results that were presented in this section.

SECTION 8. CONCLUSIONS AND RESEARCH RECOMMENDATIONS

Research on this project has indicated that primary pavement response based load equivalency factors (LEF's) are a reasonable method to estimate the equivalent damaging effects of various load parameters as compared to a standard loading condition. It is important to relate the load equivalency factors that have been examined throughout this study to actual equivalent damage on pavement structures and to pavement and life.

A useful result of this project is the identification of a method that can estimate reasonable load equivalency factors using direct primary pavement response measurements. This allows many possibilities for research and design. For example, deflection measurements can be recorded on a pavement to calculate a sample of the mix of traffic loading equivalency factors for a major roadway which is under consideration for rehabilitation design. This also opens the possibilities that accurate models can be developed from a large factorial experiment to actually predict the load equivalence factor based on the variation of the significant factors identified in this study. It is evident from the results of this project that a deflection based primary response load equivalency factor method can be employed to predict a reasonable set of meaningful load equivalency factors. Strain methods could be improved with better lateral vehicle control, more replication, and inclusion of all strain gauges in the lateral array existing on the pavement.

The deflection method proposed by Hutchinson, which is a modification of the method originally proposed by Christison as described in the previous sections, seems to be the most viable of the four methods which were analyzed in detail on this project. Additional studies are necessary to evaluate whether a variation of this method or other correlations or transformations of the concepts of primary load equivalency factors should ultimately be used in a primary response load equivalency factor model. The development of such a model will also require collection of more detailed pavement response data at a wider variety of load levels and with increased replication and control of lateral placement of the vehicles relative to the gauges.

An interesting and significant conclusion from this study is that vehicle classification is generally irrelevant in estimating load equivalency factors. This was shown because the pavement response measurements effectively returned to zero after passage of one axle set and before the influence of the next axle set on a vehicle was felt. Therefore, overall vehicle equivalency factors can be obtained by directly adding the axle load equivalency factors for each individual axle set on the vehicle. This validates the method that has been used for pavement design of adding individual axle load equivalency factors.

TEMPERATURE EFFECTS

It is apparent that temperature has a significant effect on the primary pavement response measurements and contributed to the variation of equivalency factor data on this project. This indicates that standard loading data is desirable at a wide variety of temperatures and other conditions in order to correspond with the environmental conditions that are experienced by the load for which the equivalency factor is being calculated. It is probable that if this approach is followed, temperature will no longer be significant in the determination of load equivalency factors. This was shown to be the case by the analysis of the deflection methods using the steering axle response from each run as a surrogate standard since the steering axle load could not be varied. Temperature was not significant in this analysis. It is not known what the overall effect would be on the equivalency factor values or the level of significance of the factors identified in this project. However, the general trends shown by the data on this project would not be expected to change significantly. The general effect would probably be to reduce load equivalency estimates that are extreme, mostly with regard to the strain values, to make them more reasonable. It would tend to have less of an effect on equivalency factor values that are currently reasonable. This is partly because temperature values over 100°F (37.8°C) occurred only a few times during the testing and most likely caused extremely high strains and deflections with regard to the standard loading measurements which were accomplished during cooler temperatures. The LEF values at high temperatures tended to be the outliers in the analysis.

The test case was run using the steering axle of each of the tractor trailer units as the standard load because the steering axle weight did not change significantly as the rear axles were loaded or unloaded. This analysis was described in previous sections and seems to indicate that accounting for temperature variations in the standard load has an affect on the results. The factor "temperature" was not significant even at the .25 level. This supports the theory that temperature effects will tend to cancel out in the ratio of the equivalency factor calculation. This is an extremely useful result for further testing and analysis for a detailed model development. It indicates that temperature does not need to be a factor in the model but that a standard load response is required at the same temperature that the response for the load in question is measured.

Therefore, although the analyses performed on this project provide the information needed to achieve the objective of determining the viability and applicability of primary response load equivalency factors, a more rigorous testing procedure must be undertaken to develop accurate models of load equivalency factor from primary response pavement measurements. An alternative to this approach is to use the testing methods described on this project to measure primary response load equivalency factors directly in individual pavements. A general model, however, would be useful for implementing a system for using such equivalency factors in design, research, or cost allocations purposes.

SIGNIFICANCE OF TEST FACTORS

For the deflection methods analyzed, the factor of load was by far the most significant factor. This underscores the fact that these are <u>load</u> equivalency factors and the general nature of these factors is that the actual load is the primary contributor to the damage caused on the pavement. Small additional contributions from increased or decreased tire pressure and increased or decreased speed are not nearly as evident as load. However, some indications that these factors have influence were apparent in this data. The fact that load shows to be so highly significant emphasizes the general validity of the concept of primary pavement response load equivalency factors.

The factor "pavement" on this project indicates the difference between a pavement with a 3½-in (89-mm) asphalt concrete surface versus a pavement with a 7-in (178-mm) asphalt concrete surface and did not show to be highly significant. This indicates that structural number or pavement thickness may not be a necessary factor in a response based equivalency factor method for flexible pavements. This, of course, has not been tested for any other types of flexible pavement designs or for rigid pavements. This finding, in general, agrees with the AASHTO load equivalency factors because their variation with structural number is actually quite small and may not necessarily be considered statistically significant at a high level of confidence. The fact that the factor "pavement" is not significant in this analysis is quite useful because of the possibilities for general models that apply to a wide range of pavement types and structural capabilities.

For the two deflection based methods, some interactions with the factor "pavement" were evident. This indicates that although pavement alone is not significant, the effects of some parameters such as load and tire pressure effects may be slightly different between pavement thicknesses. These multi-factor interactions, however, were not significant at the highest level, only at the .05 and .10 levels. However, since some significance was shown, future analyses should continue to consider pavement thickness or strength as a parameter for further investigation or verification of the actual influences. Several significant interactions existed with the factor "axle." These include Load x Axle and Axle x Speed interactions, as well as a three-factor interaction of Load x Axle x Speed. This indicates that the damaging effect of the various axle types depends to some degree on the level of load placed on the axle and the speed at which the vehicle is traveling. In most all cases, the slower vehicle speed tended to show more damage potential, probably because the strain or deflection response values could more fully develop when the vehicle passes slowly over the gauge than when it passes it over at 45-mi/h (72-km/h). The single axle seemed to be more sensitive to high load levels than either the tridem or tandem axles. Based on the load levels selected for this analysis, the tandem axle caused by far the least damage potential. A comparison of the combination axle type-axle load parameters among all one-way combinations of these factors indicated the relative equality between axle types and axle loads. Thus, if a somewhat larger set of tandem axle loads that have been selected to fill the low, medium, and high load categories for the tandem axle set, the tandem axles may not have shown to be less damaging than the single or tridem.

An interesting interaction was shown by the deflection methods between axle load and tire pressure. At low and medium load levels, tire pressure had very little effect. However, at high load level, the lower tire pressure seemed to have the most damaging potential. This is contrary to what would be intuitively expected since a higher tire pressure would tend to concentrate the same load on a smaller area of pavement and, thus, would seem to cause more damage. The data on this project, however, does not support this theory.

Using the Hutchinson Deflection Method, the effect of uneven tandem axles and possibly other types of multiaxles can be estimated in terms of elevated LEF's. Therefore, a response based LEF method can be used to account for axle type variations and variations between uneven wheels of a multiaxle group.

The strain methods, although not very reliable because of the variation in strain data, tended to show a main effect of speed. Again, the slow speed had the higher damaging potential. This indicates that speed should be further investigated, especially with regard to strain measurements in a primary response load equivalency factor method.

MODEL DEVELOPMENT RECOMMENDATIONS

This study shows that it should be possible to develop good regression models for general use in prediction of primary response load equivalency factors. A much larger factorial experiment than was possible on this study would be necessary in order to collect a range of data at more levels of axle load. Additional replication within the experiment will also be useful to better quantify the variation shown to be inherent in these types of measurements on this project.

Standard axle loads should be run in conjunction with each load which is under consideration for equivalency factors to get a good estimate of the variation of the standard load with temperature, moisture, and other environmental conditions and parameters. With such a large array of response measurements to the standard load under varying environmental conditions, it is highly likely that a very accurate regression equation can be developed to predict the standard load which would produce an unbiased estimator of the standard load in every equivalency factor calculation made in the process of model development. Thus the main regression equation which will predict load equivalency factor will be relatively free of variation errors in the measurement of the standard load response.

Some method could be devised to accurately vary the steering axle load in additional testing in order to develop equivalency factor models for steering axles. It was very difficult using standard trucks to vary the steering axle load. A special platform built directly above the steering axle on a special truck could be used. This would allow weights to be placed directly over the steering axle in known quantities in order to produce a valid factorial of load for steering axle.

Additional replication, that is the repeat of particular cells in the factorial, would also be useful in the model development effort to obtain more reliability in the data. This would produce, overall, better estimators of a population mean of the standard load response and the response from the loads in question. The replication should be performed in a randomized experiment such that the loading sequence is not set to be as easy on the experimenters as possible. The loading sequence instead should be as random as possible without regard for the difficulty this poses in the experimentation process. Although when the value for one cell is being obtained, multiple repeats of the runs should be performed to get better control of the lateral position of the vehicles and obtain a good average value of all repeats to stand for that one replica observation within the factorial. When the second replicate observation of that factorial is run in a random manner, again multiple repeats of that loading condition should be observed to obtain the best possible average for the second replicate as well. Better control over the lateral position, for example, might be accepting only vehicles that are less than 3-in (76-mm) away from the centerline of the instrumentation instead of 6-in (152-mm) as was used in this analysis. This should be accomplished with the goal to get even more observations in each cell to produce a better estimator of the population mean.

CORRELATION WITH LAYER THEORY

Additional analyses could also be performed to correlate the pavement response measurements on this project, or obtained during detailed model development as described above, with the results from linear elastic or viscoelastic layer theory. If accurate correlations can be obtained, then layer theory models can be used to directly estimate primary response load equivalency factors. It is recommended that evaluation of several different layer theory models over a wide range of input parameters be performed in order to better correlate to the field-measured values. Correlation to the values obtained from the more detailed testing recommended above would be advisable to have the best replication and lateral vehicle control possible in pavement response measurements. If good correlations are obtained, then a regression model may not be necessary for estimating primary response load equivalencies. Accurate modeling of pavement structures with layer theory models will allow direct computation of response based load equivalencies using any of the methods discussed on this project.

UNEVENLY LOADED MULTIAXLE GROUPS

Another useful study relative to primary response load equivalency factors would be the effects of unevenly loaded multiaxle groups. For example, the effect of a tridem axle with one of the three wheels loaded significantly higher than the other two, may produce significantly higher load equivalencies than an evenly loaded tridem. This would be especially true for some of the methods discussed on this project. The difference in an evenly loaded multiaxle versus an unevenly coded multiaxle would be quite significant in terms of pavement damage and, therefore, in load equivalency factor. A small supplementary analysis of this project showed that these effects could be significant.

SUMMARY

This study has produced valuable results with regard to better understanding and interpretation of primary response based truck load equivalency factors. It has been determined that primary response based factors can be a reasonable and quite useful method of estimating the relative damaging effects of various loading conditions on pavement structures. Several methods were identified as being accurate and useful for this purpose. A number of vehicle, pavement, and environmental factor were identified as affecting primary pavement response measurements and load equivalency factors developed from these measurements. It was determined that although some of these factors may affect the primary response measurements, they do not necessarily affect resulting load equivalency factors. These factors include pavement, temperature, and, to some degree, pavement thickness or strength. The reason for this effect is that the variations of response measurements between different levels of these factors tends to cancel out in the load equivalency factor calculation process. This concept is quite useful because of the possibility for development of general primary response load equivalency factor models.

This study has provided the basic information of which methods should be studied in more detail, which loading, pavement, or environmental factors affect load equivalency factor estimation and the level of detail which would be necessary to develop detailed models for predicting load equivalency factors at any level of the important factors. Recommendations were made for additional testing and analyses which would be necessary to develop such detailed models. This approach is highly recommended because of the excellent potential for primary response load equivalency factors identified on this project.

APPENDIX A

SUMMARY OF LOAD EQUIVALENCY FACTOR METHODS

A literature review to identify most of the available primary response truck load equivalency factor methods was undertaken as part of this project. The extensive literature review of existing local and foreign practices has revealed that the most common mechanistic responses used to determine load equivalency factors for pavement design are:

- Maximum vertical strain on top of the subgrade.
- Maximum tensile strain at the bottom of the pavement layer.
- Maximum surface vertical deflection.
- Maximum tensile stress in a concrete pavement.

The focus of this review is on studies that relate pavement performance to structural response parameters. The following reviews are concise summaries of the most relevant structural pavement response-based equivalency systems currently available.

Zube and Forsyth (1965)

Zube et al presented one of the early experimental studies to compare the relative destructive effect of a single-axle flotation-wheel (18.00 by 19.50) and a regular single-axle dual-wheel (10.00 by 20.00) configurations.[13] The two criteria of destructive effect selected were surface pavement deflection and strain at the bottom of the asphalt concrete layer.

Pavement deflection and strain measurements were obtained over eight roadways, representing a relatively wide range of flexible and composite structural section. Sufficient data were accumulated to evaluate the effect of pavement temperature, single-axle load, and tire inflation pressure on pavement deflection and maximum tensile strain. On the average, a 12-kip (5,448-kg) single-axle loading was equivalent with an 18-kip (8,172-kg) single-axle dual-wheel configuration. This equivalency is largely dependent on pavement structure and pavement temperature. To examine the effect of tire pressure on pavement deflection, the inflation pressure of the flotation tire single-axle was reduced from 75- to 55-psi (515- to 380-kPa). Under an 18-kip (8,172-kg) loading, pavement deflection decreased by 10 percent but under a 12-kip (5,448-kg) loading the deflection remained relatively unchanged.

Deacon (1969)

Deacon developed a procedure for the theoretical determination of load equivalency factors for use in those situations where distress is caused by flexural fatigue.[14] Structural pavement response was carried out using a computer program developed by the Chevron Research Company. Several axle and tire configurations, and pavement structures were analyzed assuming a circular tire imprint and a uniform contact pressure. Load equivalency factors, F_i were derived using the maximum principal tensile strains on the underside of the surface layer. The derived expression was:

$$F_i = (e_i/e_b)^{5.5} \tag{15}$$

where:

e_i = the maximum tensile strain for load i
e_b = the maximum tensile strain for the standard load (i.e., 18-kip (8,172-kg) single-axle dual-tires).

The theoretical results of this study are shown in figures 56 and 57. It can be seen that for comparable load magnitudes, single tires are approximately three times as destructive with respect to fatigue as dual tires. For this reason, Deacon recommends to identify and treat single tires separately in equivalency studies. The results in figure 57 indicate that a single axle load is equivalent in destructive effect to a tandem axle load when it has a magnitude equal to 57 percent of the tandem axle load.

Scala (1970)

Scala established equivalent loading factors to compare the effect of repeated loading on a pavement using the total vertical elastic deflection at the surface.[15] He found from the AASHO Road Test results that load equivalency factors are approximately equal to the figure 56 fourth power of the ratio of the actual loads. Accepting that the deflection of a pavement is proportional to the load, the load equivalency factors for a given loading system would also be proportional to the fourth power of the ratio of the deflections under the loads.

Based on deflection, equivalent loads with common axle types were 11.4- to 12-kip (5,176- to 5,448-kg) for single axle single tires, 18-kip (8,172-kg) for single axle dual tires, 29- to 31-kip (13,166- to 14,074-kg) for tandem axle groups with dual tires, and 40.7-kip (18,478-kg) for tridem axles. Therefore, the following equations were suggested.

Single axle single tires

$$F_i = (W_S/12)^4 \tag{16}$$

Tandem axle group with dual tires

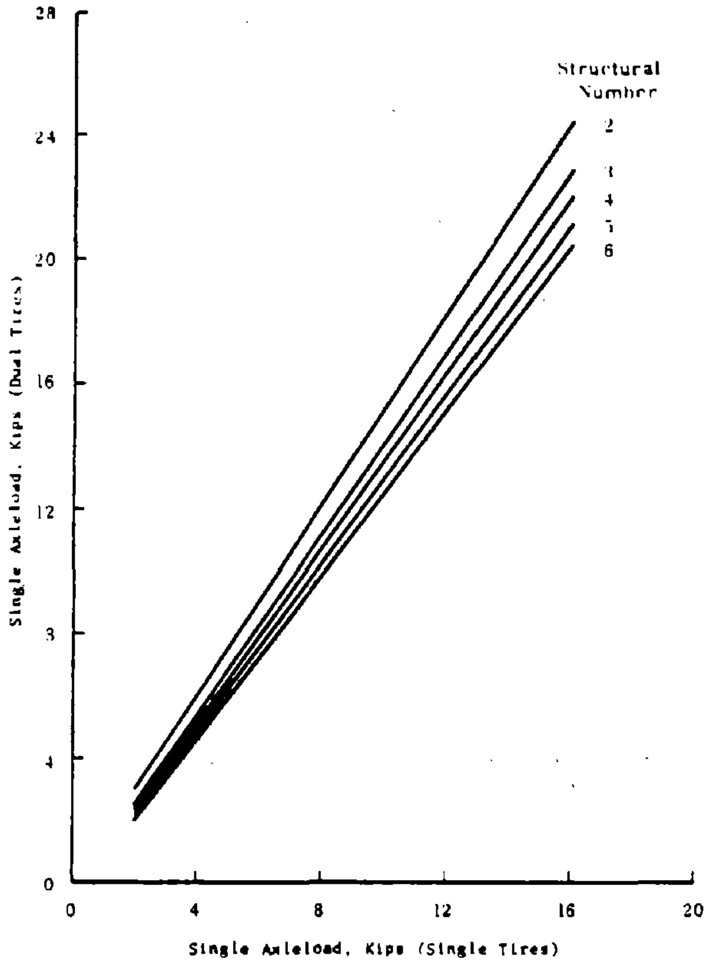
$$F_i = (W_T/30)^4 \tag{17}$$

Triple axle with normal tires

$$F_i = (W_{TR}/40.7)^4 \tag{18}$$

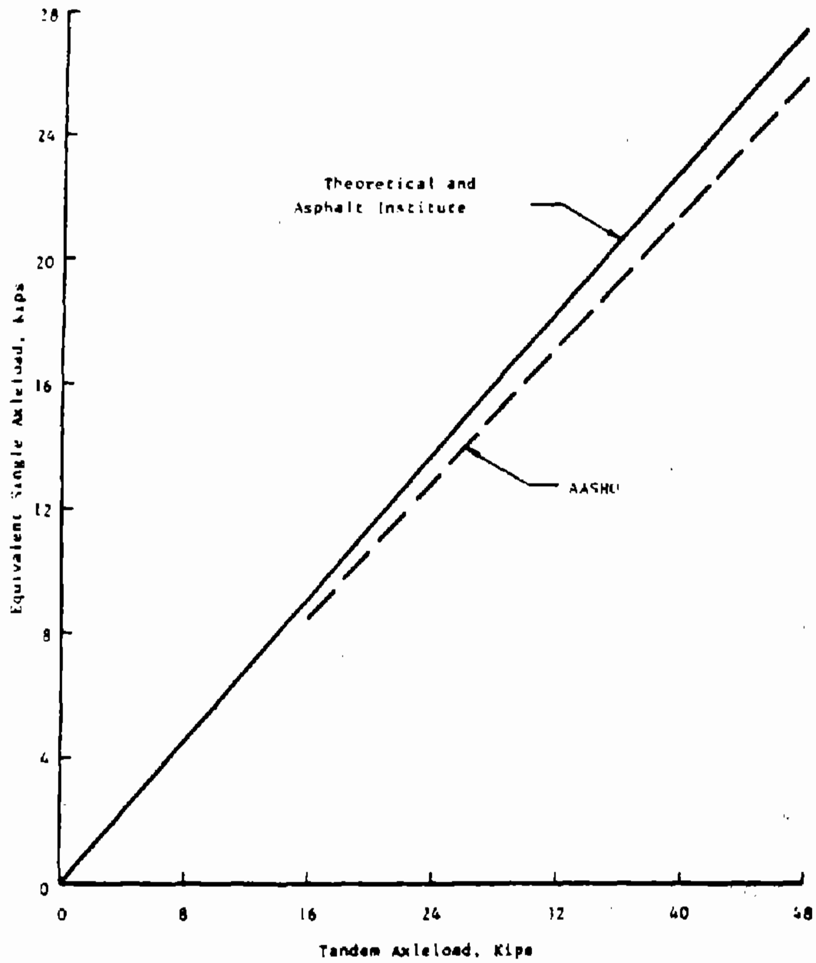
where:

F_i = Equivalency load factors
W_S = Load on the single axle single tires
W_T = Load on the tandem axle system
W_{TR} = Load on the triple axle



1 kip=454 kg

Figure 56. Load equivalency between single and dual tires.[14]



1 kip=454 kg

Figure 57. Load equivalency between single and tandem axles.(14)

Scala mentioned that vehicle speed has quite an impact on surface deflections. Assuming the equivalencies are related to the fourth power of the ratio of deflections, a vehicle with a particular loading traveling at 10-mi/h (16.1-km/h) is approximately eight times (equivalent repetitions) as severe as a vehicle with the same loading at 45-mi/h (72.5-km/h).

Gerard and Harrison (1970)

Gerard et al presented a theoretical analysis of load equivalent factors between dual tandem versus dual, dual-tandem versus single, and dual versus single.[16] Pavement structures are modeled as a two layer system of linearly elastic, isotropic materials, with each layer being homogeneous. First layer comprises the surface and the base, and it is finite, second layer represents the subgrade and it is infinite. Contact area for each wheel is the same irrespective of assembly arrangement.

The variables considered are:

- Vertical displacement on surface.
- Vertical displacement at the interface.
- Vertical stress at interface.
- Vertical strain in lower layer at the interface.
- Maximum stress difference (shear stress) in lower layer at interface.
- Principal tensile strain in upper layer.

It was found that the six criteria used in this analysis fall into three distinct categories:

1. Vertical displacement on the surface and vertical displacement at the interface which gave similar results when the modular ratio was high but were significantly different when the modular ratio is unity. The interface criterion always gives ratios of assembly loads that are less than for the surface criterion.

2. Vertical stress at interface, vertical strain in lower layer at interface, and maximum stress difference in lower layer at interface gave almost identical results that were significantly different than those for either of the displacement criteria. The ratios of the assembly loads were slightly greater for the criteria of maximum stress difference and vertical strain than for the criterion of vertical stress.

3. Principal tensile strain in upper layer which in general gave significantly different values for the ratio of the assembly loads than the other two groups. The change in wheel spacing has a greater relative effect on the ratio of assembly loads for these criterion than for any of the other criteria.

Assuming that the maximum stress difference at the interface is accepted as the criterion for subgrade distress, and principal tensile strain as the criterion for pavement distress, then for any average wheel spacing equivalency factors as shown in table 35 are obtained.

Table 35. Equivalency factors.[16]

	Dual Versus Single		Dual Tandem Versus Single		Dual Tandem Versus Dual
	Shallow Pavement	Deep Pavement	Shallow Pavement	Deep Pavement	Shallow or Deep Pavement
Low Modular Ratio	1.8	1.5	3.6	3.0	1.8 to 1.9
High Modular Ratio	1.5	1.0	3.1	2.0	1.8 to 1.9

Ramsamooj, Majidzadeh and Kauffmann (1972)

Ramsamooj et al applied fracture mechanics to the problem of fatigue cracking and failure of flexible pavements.[17] For this purpose the Pari's law

$$d_c/d_n = AK^4 \quad (19)$$

is used, where:

d_c/d_n = the rate of crack propagation.
K = the stress - intensity - factor.
A = a constant of the material.

From this theoretical relation and the fact that K is proportional to the load, P, the load equivalency factor for single axle loads is found to be proportional to the fourth power of the load. The load equivalency factor for tandem axles depend on the spacing of the axles and the shape of the influence line of K as the loads move across the crack. It can be obtained from the influence lines for K by taking the ratio between the fourth power of the rises and falls of the tandem loading, and the rise of the standard loading as shown in figures 58 and 59.

Jung and Phang (1974)

Jung et al used several Ontario (Canada) flexible pavements and the AASHO Road Test results to derive load equivalency factors in terms of the vertical deflection on top of the subgrade.[5] The theoretical study was done using the Chevron computer program, and also the Odemark's concept of equivalent layer thickness. Deflection calculations using Chevron's computer program were essentially similar to Odemark's method. Thus, due to simplicity, the latter was used to determine load equivalency factors from the AASHO Road Test data.

Correlation regression analyses for AASHO Road Test data resulted in the following load equivalency factor equation:

$$F_i = (W_i/W_s)^6 \times 10^{-0.09(Pi - Ps)} \quad (20)$$

where:

W = subgrade vertical deflection using Odemark's method,
P = axle load, and
i and s subindexes = applied and standard load, respectively.

A plot of this equation is shown in figure 60.

Terrel and Rimstrong (1976)

Terrel et al derived theory-based load equivalency factors considering the effects of wheel load, tire contact pressure and width, thickness and nature of pavement layers, speed of vehicle, and pavement temperature.[18]

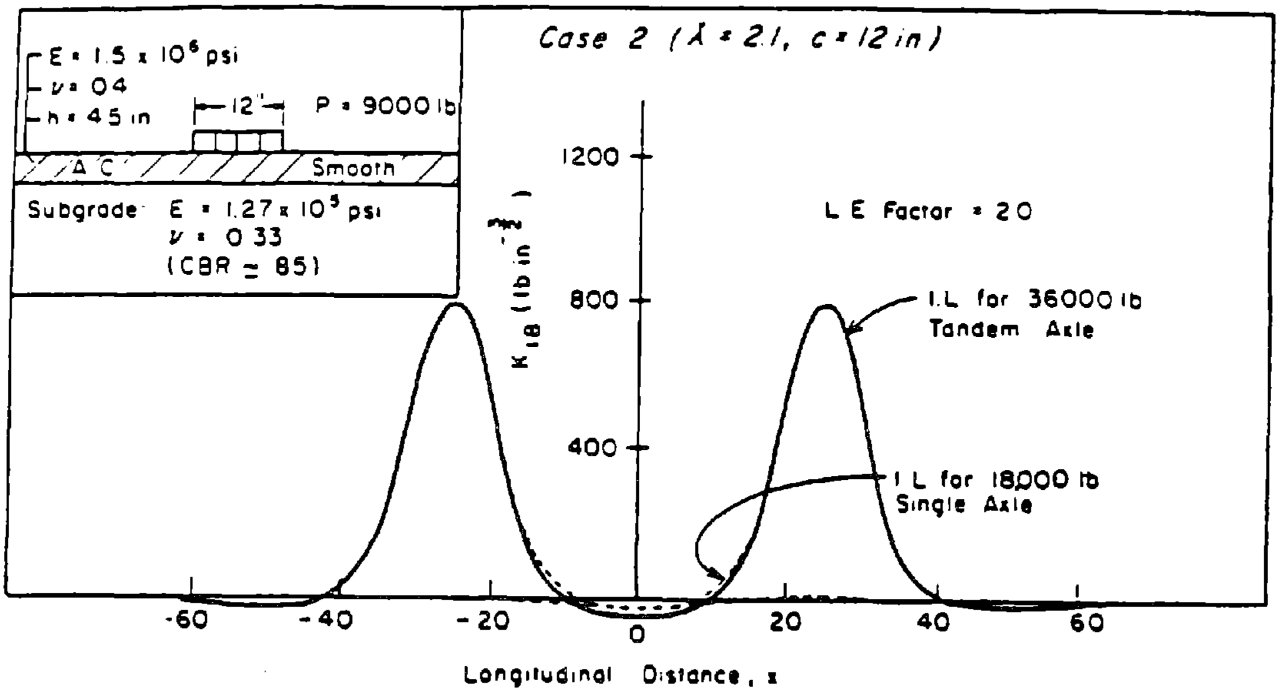


Figure 58. Load equivalency factor for 36-kip (16,344-kg) tandem axle(17)

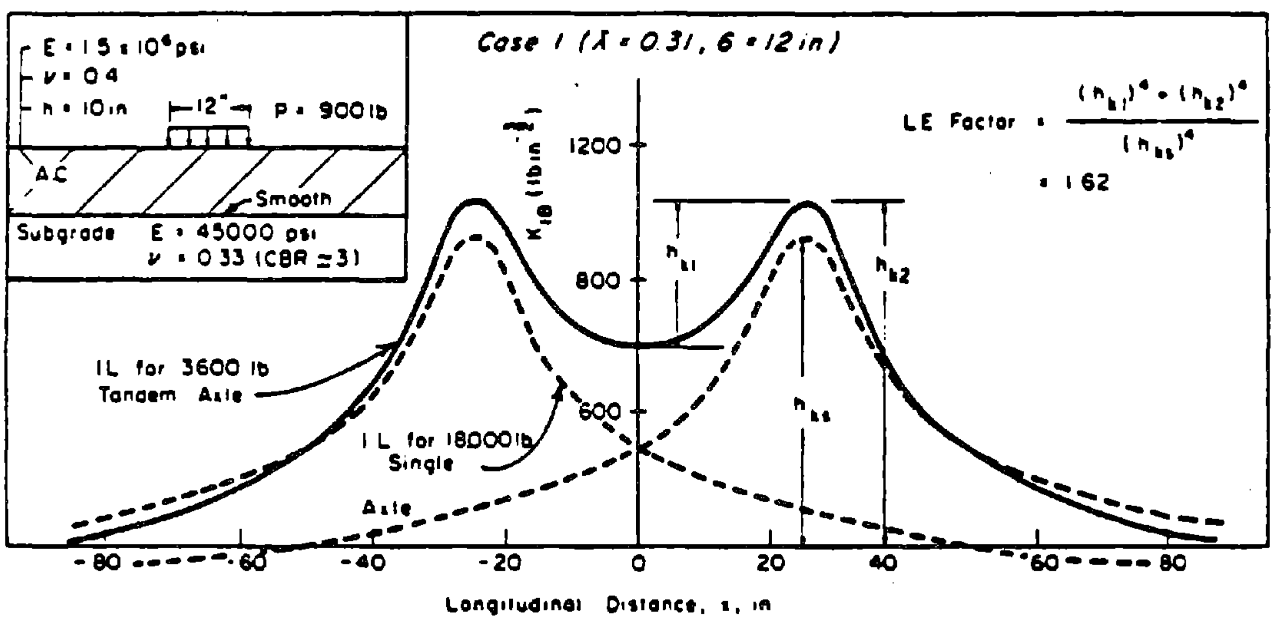
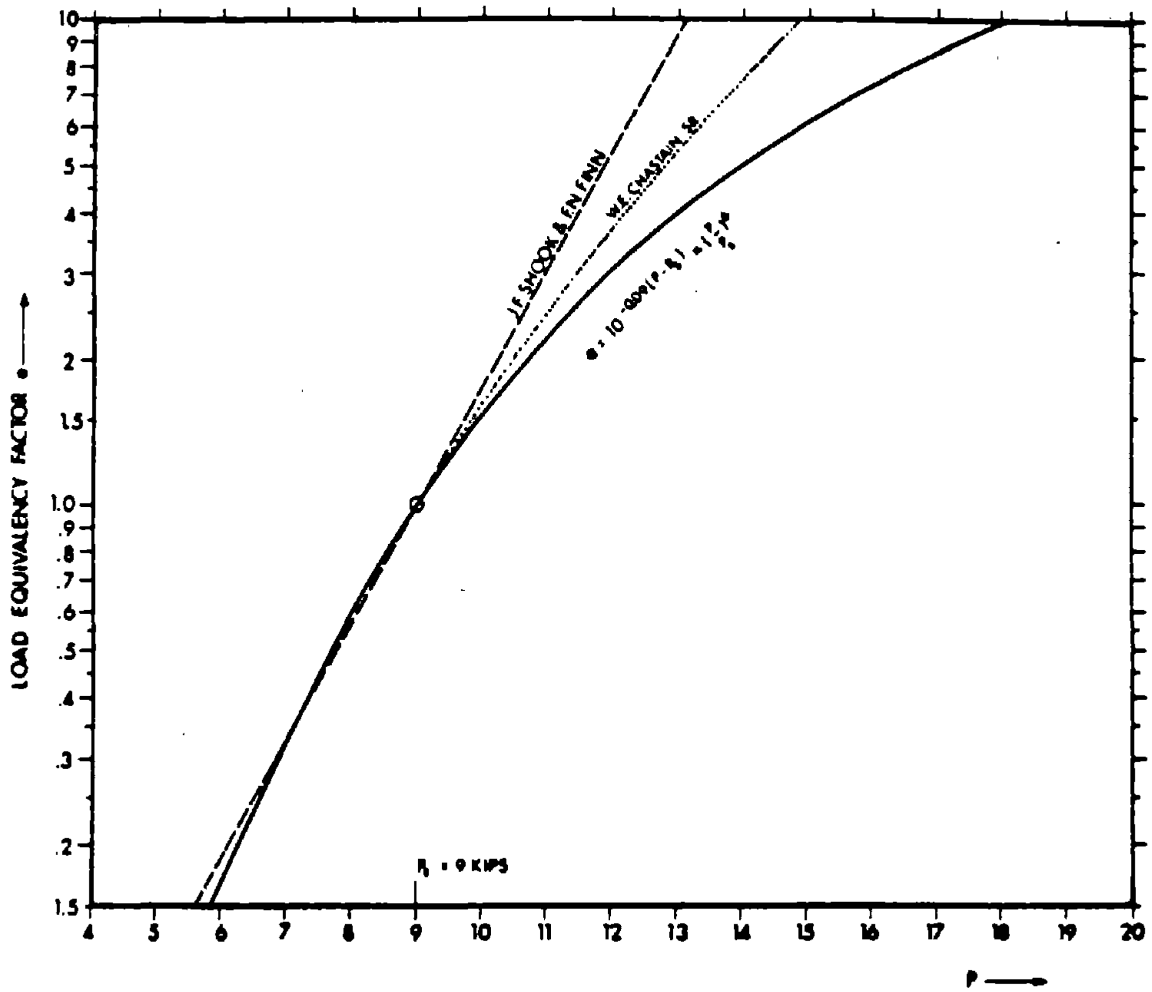


Figure 59. Load equivalency factor for 36-kip (16,344-kg) tandem axle(17)



1 kip=454 kg

Figure 60. Load equivalency factor versus wheel load[5]

The CHEV 5L computer program was used to calculate the radial tensile strain on the bottom of asphalt concrete layers, and the vertical compressive strain on the subgrade.

Using as a reference a truck running at 10-mi/h (16.1-km/h) with an 18-kip (8,172-kg) axle load and 10-in (254-mm) wide dual tires on a pavement with 6-in (152-mm) of asphalt concrete at a temperature of 68.5°F (20.3°C), relationships were developed between load and the corresponding number of repetitions to failure (see figures 61 and 62). Using these figures, equivalency load factors, F_i, are found by:

$$F_i = N_b/N_i \tag{21}$$

where:

N_b = number of load repetitions of the standard load, and
N_i = number of load repetitions of the applied load.

Treybig and Von Quintus (1976)

Treybig et al developed load equivalency factors for triaxle loading using maximum surface deflections, maximum tensile strain at the bottom of the pavement surface, and maximum vertical strain on top of the subgrade.[19] The structural responses were found using the ELSYM5 computer program. The equivalency factors were found assuming that the relation between the maximum structural response and the load equivalency factors is unique regardless of the type of axle configurations. It was found that the maximum compressive strain on top of the subgrade gave the least amount of error in estimating equivalencies as compared to extrapolated equivalency factors for load and axle configurations outside the boundaries of the AASHO Road Test.

Nordic Cooperative Research Project (1977)

In the Nordic Cooperative Research Project the applicability in the Nordic countries of the AASHO Road Test results was examined.[20] Load equivalency factors, F_1, were determined as a function of the axle loads.

$$F_i = (P_i/P_s)^n \tag{22}$$

where:

P_i = applied axle load,
P_s = standard load, and
n = constant that takes different values depending on the type of subgrade and the structural pavement response considered.

The computed values of n shown in table 36 are the result of an extensive finite element analysis of several pavement thicknesses and base moduli, two types of subgrade (clay and sand), and two structural pavement responses (vertical subgrade strain and horizontal strain in the lower interface of the asphalt layer).

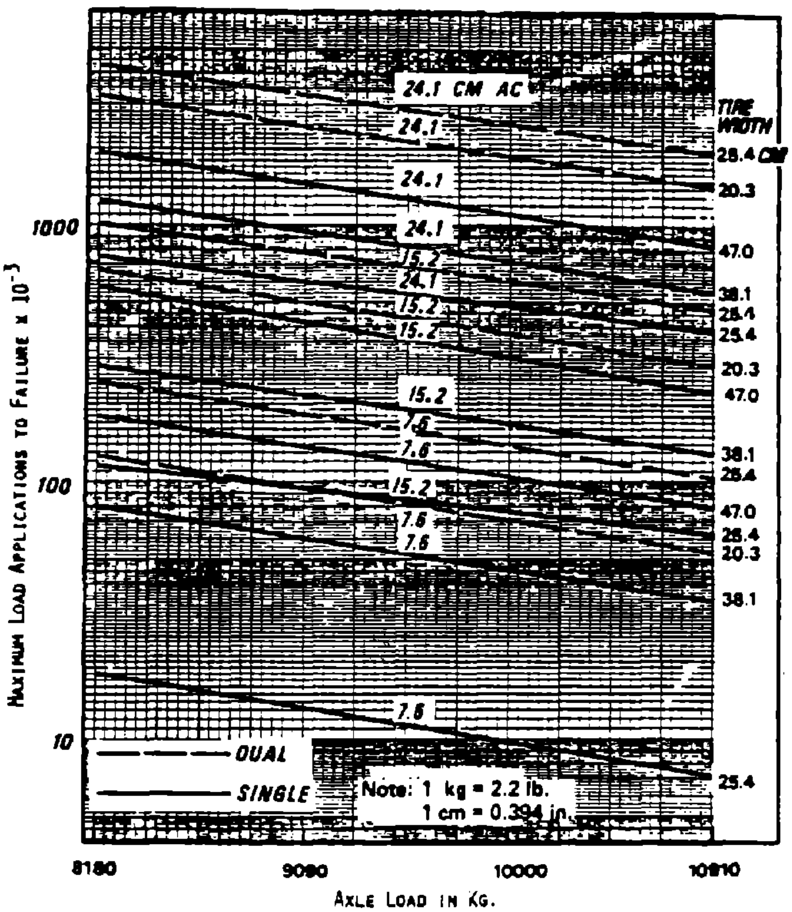


Figure 61. Relative pavement life to failure for a range of loads and pavements based on fatigue of AC layer.[18]

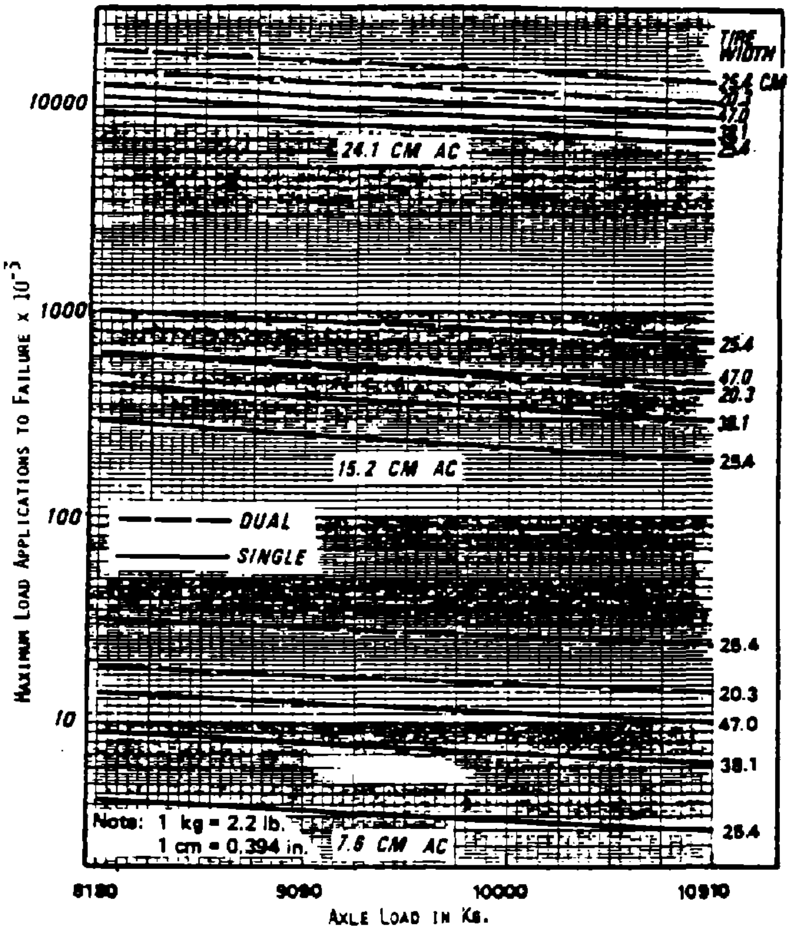


Figure 62. Relative pavement life to failure for a range of loads and pavements based on fatigue of AC layer.[18]

Table 36. Mean and standard deviation of n.

Criterion	Vertical Strain		Horizontal Strain	
Subgrade Type	n	Std.Dev.	n	Std.Dev.
Clay	3.98	.31	3.77	.25
Sand	3.05	.51	3.28	.31

Kirwan, Snaith and Glynn (1977)

Kirwan et al using a DEFPAV, a finite element computer program, developed load equivalency factors, F_i, in terms of the applied, P_i, and the standard, P_s, axle loads:[21]

$$F_i = (P_i/P_s)^{5.1} \tag{23}$$

The exponent of 5.1 was derived by computing the rut depth on a pavement structure (the Nottingham test pit) under a number of different wheel loads. The standard wheel load used was 9-kip (4,100-Kg).

Christison (1978)

Christison reported on early experiments at the Alberta Research Council instrumented pavement site.[22] The site allows recording of longitudinal strains at the bottom of the asphalt concrete, pavement surface deflections, and pavement temperatures at various depths within the pavement structure under moving vehicle loads. Load equivalency factors are calculated on the basis of the measured pavement response as follows:

Single axle loads:

$$F_i = (e_i/e_b)^{3.0} \tag{24}$$

$$F_i = (d_i/d_b)^{3.0} \tag{25}$$

Tandem axle loads:

$$F_i = (e_i/e_b)^{3.0} + (Ke_i/e_b)^{3.0} \tag{26}$$

$$F_i = (d_i/d_b)^{3.0} + (\quad d/d_b)^{3.0} \tag{27}$$

where:

e_i = Tensile strain caused by the single axle load or the leading axle on a tandem axle load

e_b = Strain caused by the standard 18-kip (8,160-kg) single axle dual tire load

K = Average ratio of strains recorded under the second axle to those under the leading axle

d_i = Surface deflection under a single axle load or leading axle on a tandem axle load

d_b = Surface deflection under the 18-kip (8,160-kg) single axle dual tire load

= Difference between maximum deflections under the second axle and the minimum deflection between axles.

The standard axle of the Benkelman Beam truck was used as the reference axle load, (i.e., single axle carrying a load of 18,000-lb (8,160-kg) on dual 10.00 by 20.00 tires inflated to 80-psi (550-kPa).

Pavement response parameters were found to depend on temperature, vehicle speed, and lateral placement with respect to the sensors. Response ratios obtained by successive runs lying within a .98-in (25-mm) range from the sensors were averaged to account for the variation in lateral vehicle placement. In order to eliminate the effect of temperature and vehicle speed, each pass of the axle load to be evaluated was followed by the reference axle load at the same speed. Nevertheless, the effect of pavement temperature and vehicle speed on pavement response ratios and in turn on equivalency factors is quite substantial. Load equivalency factors based on strain were found to be more sensitive to pavement temperature. For a standard axle, for example, load equivalency with respect to strain was nine times higher at 3.1-mi/h (5-km/h) than at normal highway speeds, while load equivalency with respect to deflection was 50 times higher at 77°F (25°C) than at 40°F (4.4°C).

The results presented next refer to pavement tensile strain and surface deflection ratios under single axles with dual tires, single conventional tires and single wide-base tires. For a given axle and tire configuration, response ratios were regressed versus the load carried to allow interpolation for a variety of loads.

The load range that was tested on the dual tire axles varied from 12,600- to 26,300-lb (5,720- to 11,940-kg). Tires ranged in size from 10.00 by 20.00 to 12.00 by 22.50. The following regression equations were developed for the pavement response ratios.

$$\frac{e_{TEN}(L)}{e_{TEN}(80)} = -0.344+0.0174(L) \tag{28}$$

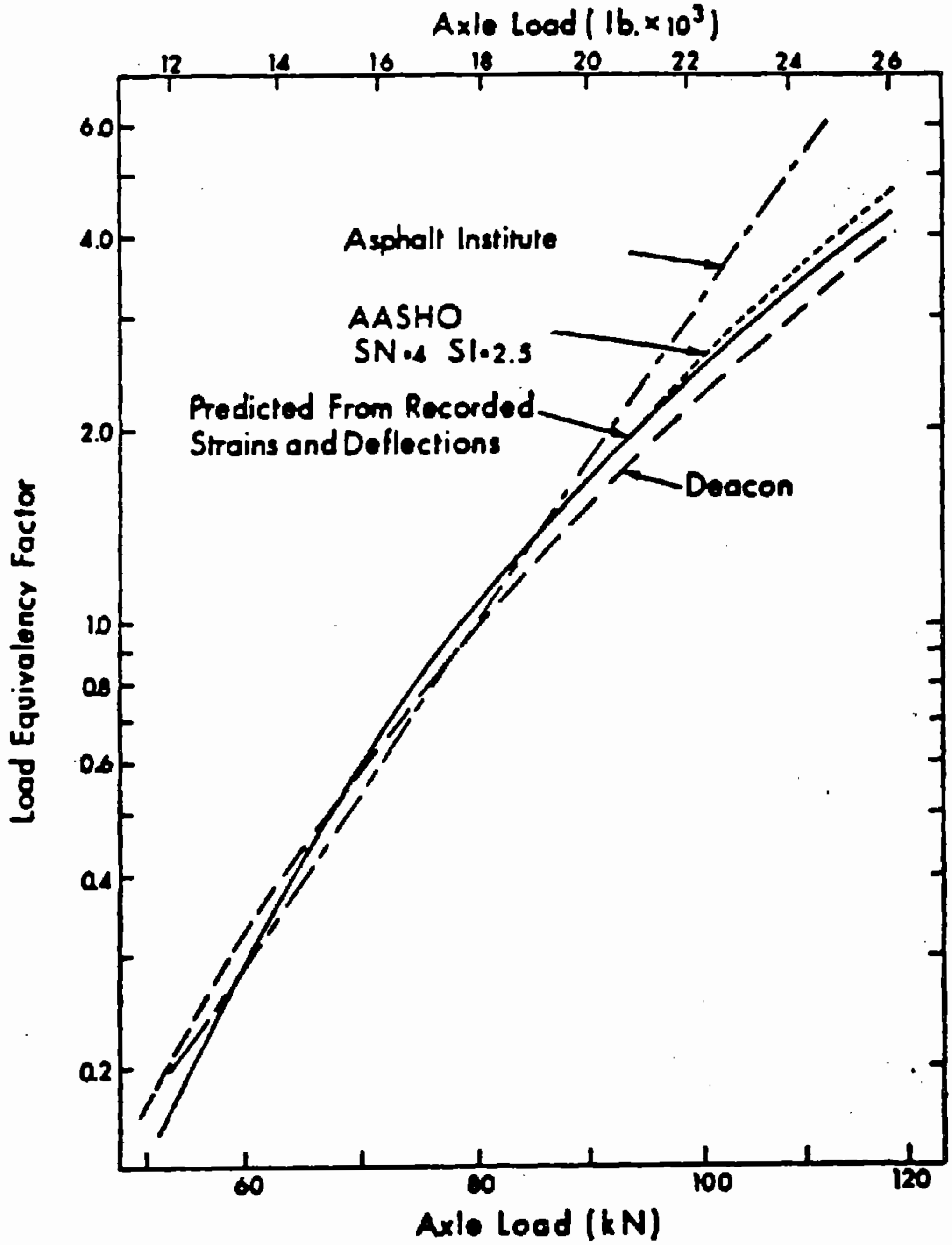
$$\frac{d(L)}{d(80)} = -0.380+0.0172(L) \tag{29}$$

where e_{TEN} is the tensile strain at the bottom of the asphalt concrete and d is the pavement surface deflection under a load L, (kN). The fit of these equations is typically very good, (i.e., r^2 in the order of 0.97). For the range of loads tested, the strain and deflection ratios were similar and therefore, load equivalency factors were computed using the average of strain and deflection ratios for each load level, (figure 63). The calculated load equivalency factors were found to be in good agreement with empirical equivalency factors derived from the AASHO Road Test.[23]

The load range that was tested for single axles on conventional single tires ranged from 2,000- to 12,000-lb (908- to 5,448-kg). Tire sizes of 10.00 by 20.00, 12.00 by 0.00, 11.00 by 20.00 and 12.00 by 22.50 were tested with inflation pressures ranging from 55- to 80-psi (.039- to .056-kg/mm^2). The following regression equations were developed for the pavement response parameters.

$$\frac{e_{TEN}(L)}{e_{TEN}(80)} = -0.53+0.0199(L) \tag{30}$$

$$\frac{d(L)}{d(80)} = -0.040+0.0207(L) \tag{31}$$



1000 lb=1 kN
1 lb=0.454 kg

Figure 63. Load equivalency factors for single axles with dual tires.[22]

The linear relationships imply that possible differences in the magnitude of the recorded pavement responses due to variation in inflation pressure were masked by load effects. Setting the response ratios equal to 1.00, the load L that would cause equal response, (i.e., and imply equal damage) with the reference load can be calculated. Thus, a 12,000-lb (5,448-kg) axle load on single tires was found equivalent with the 18,000-lb (8,172-kg) reference axle load. The load equivalency between single and dual tires for a range of loads can be seen in figure 64. These results agree with findings by Deacon.[14]

The load range that was tested for single axles on single wide-base tires was from 13,850- to 19,000-lb (6,270- to 8,640-kg). Both bias-ply and radial 18.00 by 22.5 tires were tested inflated at 87- to- 85-psi (601- and 587-kPa), respectively. Prior to testing, tire imprints were obtained by raising the wheel, painting and lowering the tire on paper. It was shown that the imprint area increases for increasing load or decreasing tire pressure and that for comparable loads and inflation pressures, the area under the bias-ply tire is roughly 10 percent lower than under the radial tire. It was decided, however, to neglect the effect of tire imprint characteristics on the recorded pavement response parameters. As a result, pavement response ratios and equivalency factors were evaluated using the combined response measurements from both tire types. The results of the regression analysis are given by the following equations:

$$\frac{e_{TEN}(L)}{e_{TEN}(80)} = 0.310+0.00925(L) \tag{32}$$

$$\frac{d(L)}{d(80)} = 0.485+0.00804(L) \tag{33}$$

The study concludes that for the range of loads considered and independently of the postulated criterion, the potential damaging effect of wide-base tires is lower than that associated with conventional single tires but higher than that associated with conventional dual tires. It was also found that interfacial strains and surface deflections caused by the standard 18-kip (8,160-kg) axle load increased with service life, increasing asphaltic concrete temperature and decreasing vehicle velocity.

Von Quintus (1978)

Von Quintus derived equivalency factors outside the boundaries of the AASHO Road Test results by establishing a relationship between a structural pavement response (e.g., strain, stress, deflection) and AASHO equivalency factors.[24] The response variables selected were surface deflection, tensile stress or strain at the bottom of the surface layer, and compressive strain at the top of the subgrade. Using ELSYM5 and SLAB-49, the structural responses were obtained for both flexible and rigid pavements. Three mathematical relationships, described below, were considered in relating performance equivalencies to each of the mechanistic variables. All relationships are hinged to AASHO data in that the AASHO single axle equivalency factors were used to calibrate the equations.

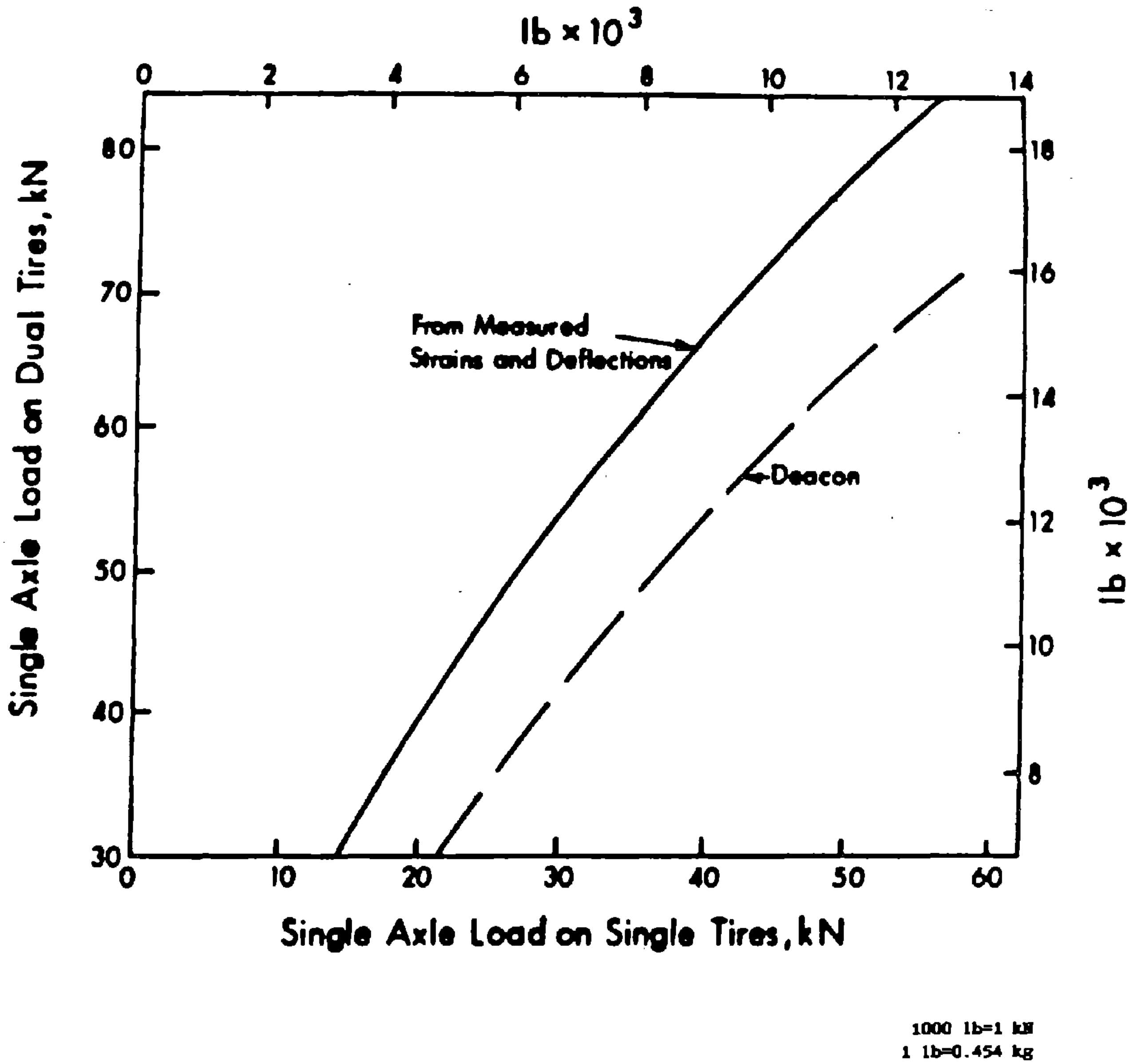


Figure 64. Load equivalency; single versus dual tires.(22)

Ratio method:

$$F_T(2X) = [RV_T(2X)/RV_s(X)]F_s(X) \tag{34}$$

Exponential method:

$$F_i(X) = [RV_i(X)/RV_s(18)]^B \tag{35}$$

Curvature method:

$$F_i(X) = [RV_i(X)/RV_s(18)]^B + [\quad RV_i(X)/RV_s(18)]^B \tag{36}$$

where:

$F_T(2X)$ = Predicted equivalency factor for a tandem axle load of 2X
$F_s(X)$ = AASHO equivalency factor for a single axle load of X
$RV_T(2X)$ = Maximum response variable under a tandem axle load of 2X
$RV_s(X)$ = Maximum response variable under a single axle load of X
$F_i(X)$ = Equivalency factor for an axle configuration i of load X
$RV_s(18)$ = Maximum response variable for an 18-kip (8,172-kg) single axle load
$RV_i(X)$ = Difference in magnitude between response variables under and between axle loads
B = Experimentally determined constant

Performance equivalency factors were predicted for flexible pavements using the above criteria and response variables. It was found that the curvature method should be used for the asphaltic concrete tensile strain or the subgrade compressive strain, and the ratio method should be used for surface deflections.

Equivalency factors for rigid pavements were not predicted within a reasonable accuracy for the AASHO conditions. They were dependent to some degree on the model and loading configurations used to simulate in-field conditions.

Christison and Shields (1980)

Christison et al reported results from additional testing at the Alberta Research Council instrumented pavement site.(25) A variety of bias-ply and radial wide-base tires were tested as outlined below:

1. Single axles on 18.00 by 22.5 dual tires with loads ranging from 14- to 28-kip (6,356- to 12,712-kg) and single axles on 16.50 by 22.5 dual tires with loads ranging from 12-kip- to 21-kip (5,448- to 9,534-kg).

2. Tandem axles on 18.00 by 22.50 single tires with loads ranging from 25- to 39-kip (11,350- to 17,706-kg).

3. Single axles on 10.00 by 20.00 dual tires with loads ranging from 15- to 24-kip (6,810- to 10,896-kg).

Throughout the analysis, a single axle load of 18-kip (8,172-kg) on dual 10.00 by 20.00 tires was used as a reference as in the previous study.(22) For a given load, average values of strain and deflection ratios, (Response ratios, Rr), were calculated as a function of the axle load, L(kN), and used as the criterion for calculating axle load equivalencies.

With respect to ply type, the results differed for the tires with 18.00 by 22.50 and 10.00 by 22.50 dimensions. For the wide-base tires, there was no difference in response ratio between the bias-ply and the radial type. For the 10.00 by 20.00 size, bias-ply and radial type yielded the following different response ratios, respectively.

$$Rr = 0.179+0.0102(L) \qquad (37)$$
$$Rr = 0.190+0.0093(L) \qquad (38)$$

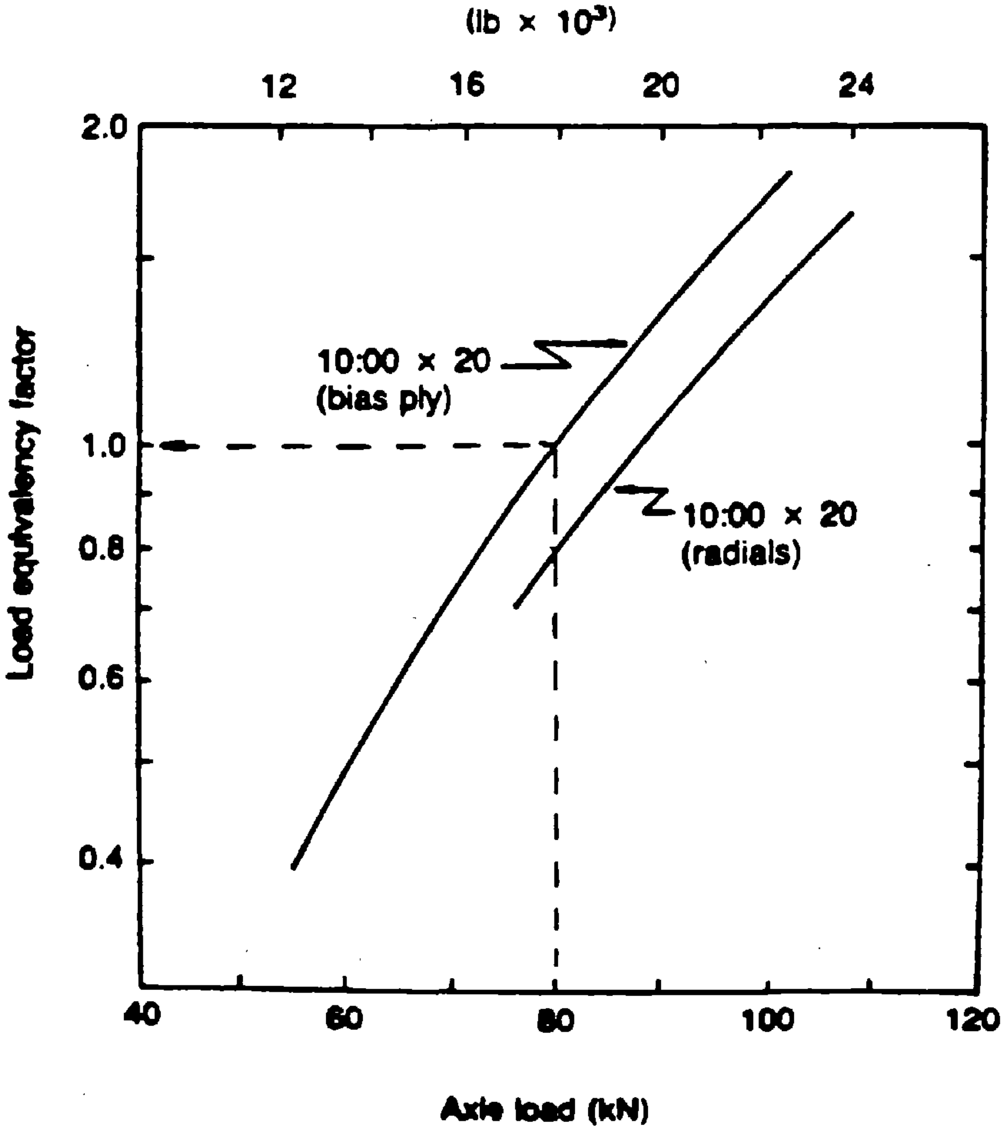
These two equations suggest an 8 percent higher response ratio for the bias-ply tires which is translated into a 25 percent higher pavement damage, (figure 65).

Wide-base tires were further evaluated with respect to tire size. The 16.00 by 22.50 tires were shown to be slightly more damaging in comparison to the 18.00 by 22.5 tires for similar loads. This increase in load equivalency with decreasing tire width is consistent with other analytical and experimental work and can be explained by the increased contact pressure of the tire imprint.

The relative damaging effect of single axles on wide-base tires and conventional dual tires was estimated by combining information for the relationships illustrated in figures 63 and 64. Load equivalencies for single axles at the legal load limit were compared for conventional dual tires and wide-base tires, that is (i.e., 20,000- and 22,000-lb (9,080- and 9,988-kg), respectively. It was found that depending on tire type and size, wide-base tires are 1.2 to 1.8 times more damaging than conventional duals. The same comparison is illustrated in figure 66 over a variety of axle loads.

Wang and Anderson (1981)

Wang et al using a procedure similar to Treybig et al and Von Quintus determined load equivalency factors for triaxle loading for flexible pavements.[26, 19, 24] The AASHO Road Test results were used to calibrate the results from the mechanistic approach. Using the Bitumen-Structures-Analysis-in-Roads (BISAR) computer program, the maximum subgrade compressive strains were calculated. These values were then related with AASHO load equivalency factors in logarithmic coordinates for single- and tandem-axle loadings, resulting in two parallel lines. Using these parallel lines and the point corresponding to a load equivalency factor of approximately 55,000 repetitions of 76-kip (34,504-kg) triaxle loading, the figure 65 load equivalency factors of various triaxle loading entities were obtained as shown in figure 67 and 68.



1000 lb=1 kN
1 lb=0.454 kg

Figure 65. Conventional duals; effect of ply type on load equivalency.[25]

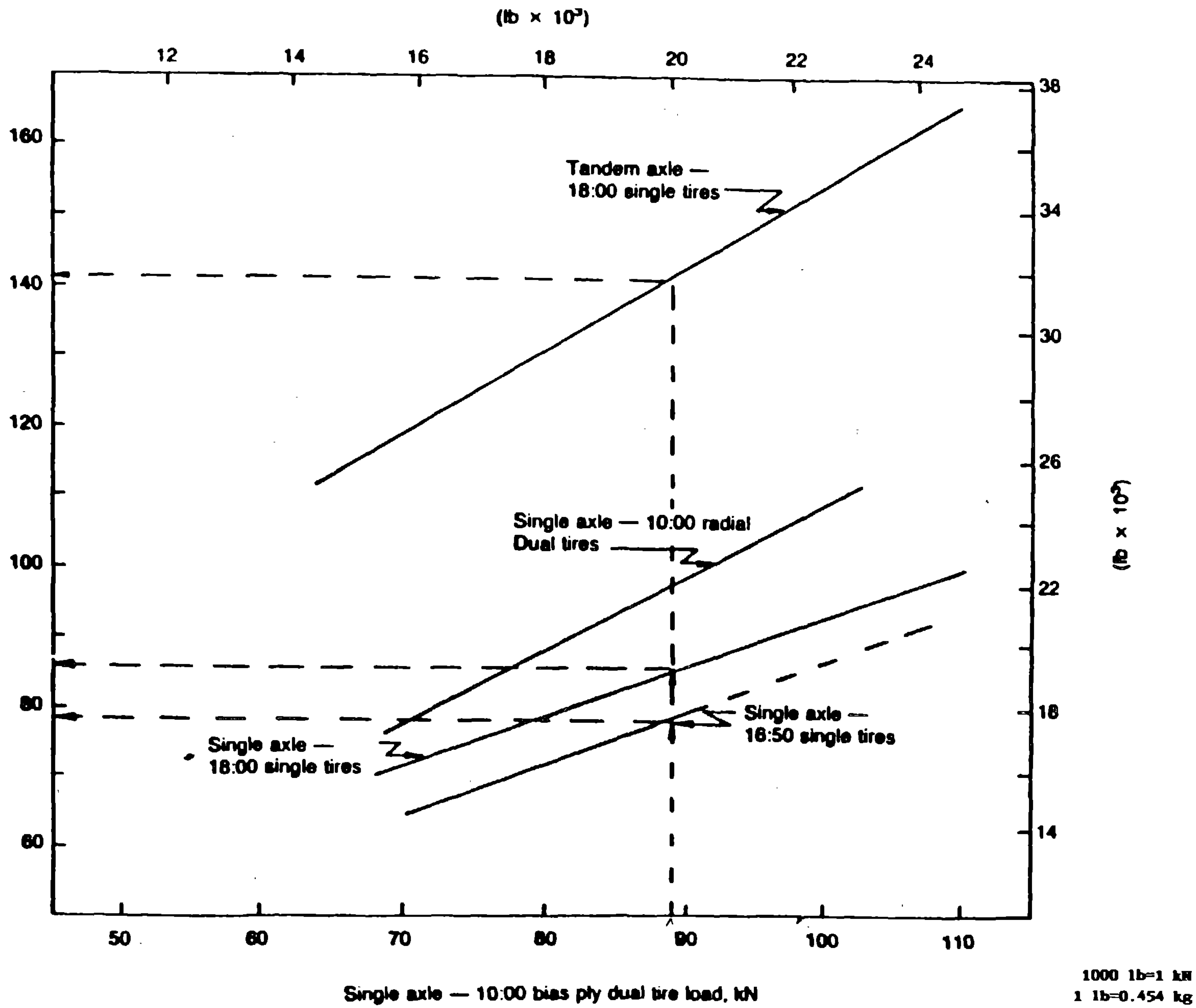


Figure 66. Load equivalencies for various tire configurations.[25]

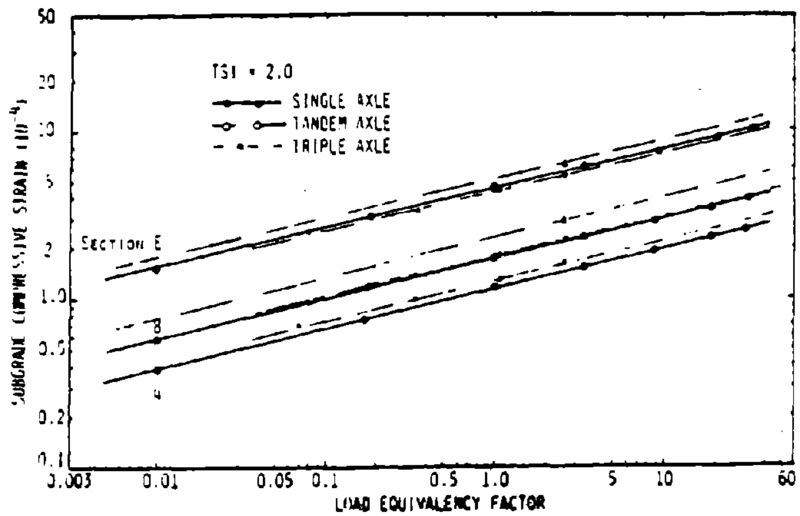


Figure 67. Subgrade compressive strain versus load equivalency factor for sections 4, B, and E.[26]

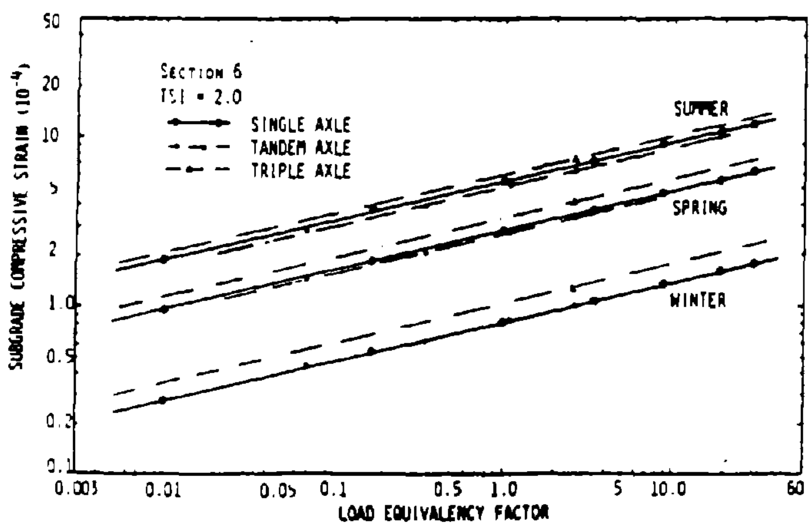


Figure 68. Subgrade compressive strain versus load equivalency factor for section 6.[26]

Tayabji, Ball and Okamoto (1983)

Tayabji et al conducted an experimental and theoretical study of load equivalency factors for tridem-axle loading on rigid pavements.[27]

Strains and deflections were measured at the five sites on I-90 in Minnesota. The applied loads were a 20-kip (9,080-kg) single-axle, a 34-kip (15,436-kg) tandem-axle, a 42-kip (19,068-kg) tandem-axle, and a 42-kip (19,068-kg) tridem-axle. Theoretical analysis was also conducted using a finite element program. Calculated edge strain profiles are shown in figure 69.

The results of this study indicate that a tridem-axle can be considered as equivalent to a single-axle weighing about 50 percent of the tridem-axle and to a tandem-axle weighing about 80 percent of the tridem-axle. Load equivalency factors are shown in figure 70 and table 37.

Southgate and Deen, (1984)

Southgate et al introduced a strain energy approach to the asphalt concrete fatigue problem.[7] Using the strain energy, W, of a body, an expression for the "work strain", e_w, was found to be

$$e_w = (2W/E)^{0.5} \tag{39}$$

where, E is Young's modulus of elasticity.

To apply conventional concepts of load equivalency factor, work strain was related to tensile strain at the bottom of the asphalt concrete, e_a, through regression, (equation 40). The expression used for the load equivalency calculations related the number of standard axle load repetitions, N, to work strain, (equation 41). The Chevron n-layer computer program was used for stress analysis.

$$\log(e_a) = 1.1483\log(e_w) - 0.1638 \tag{40}$$

$$\log(N) = -6.4636\log(e_w) - 17.3081 \tag{41}$$

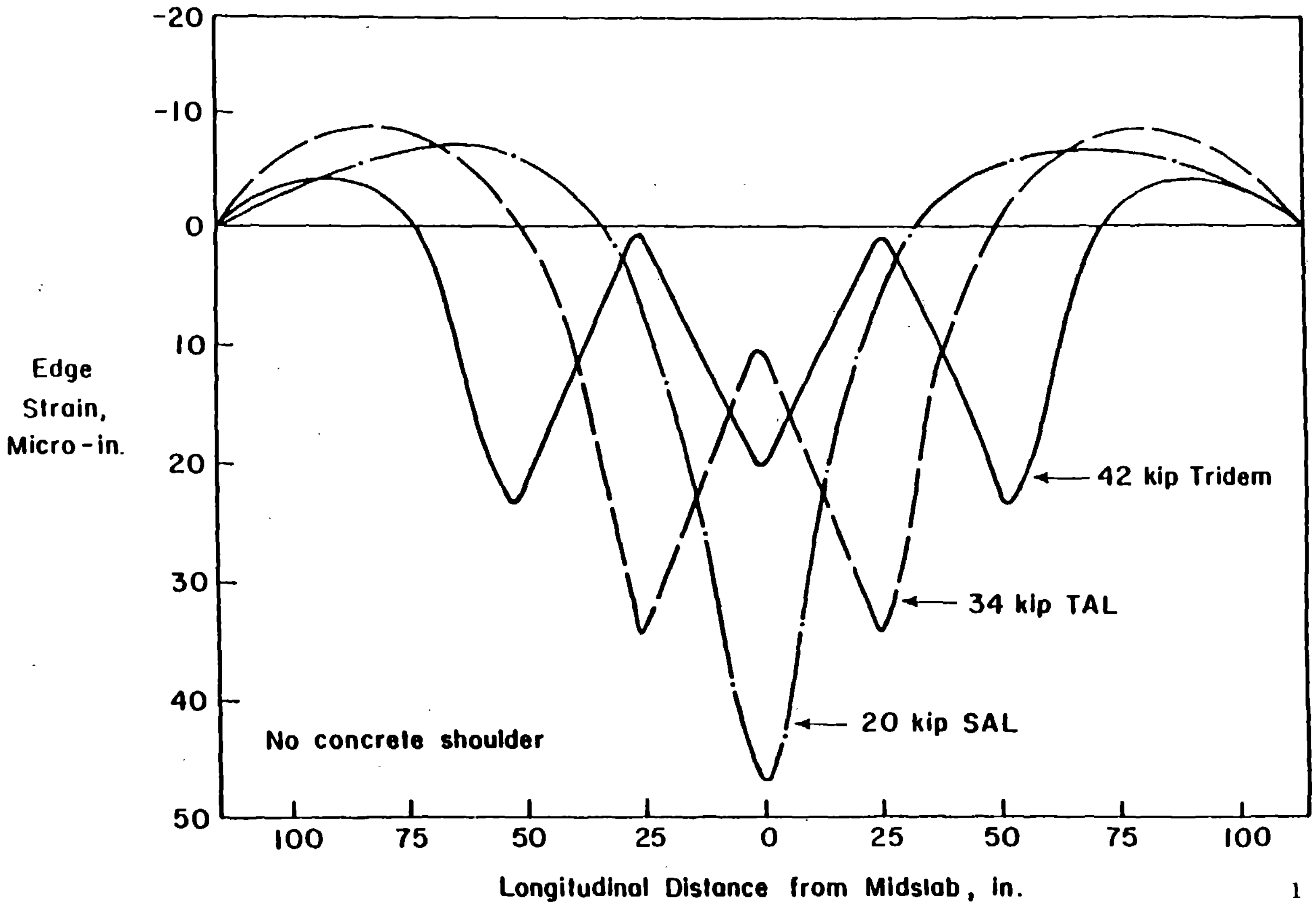
The load equivalency factor, F_i, was defined as

$$F_i = N_{18}/N_L \tag{42}$$

where:

N_{18} = repetitions calculated by equation 41 in which the work strain is that due to an 18-kip (8,160-kg) four-tired single axle load and

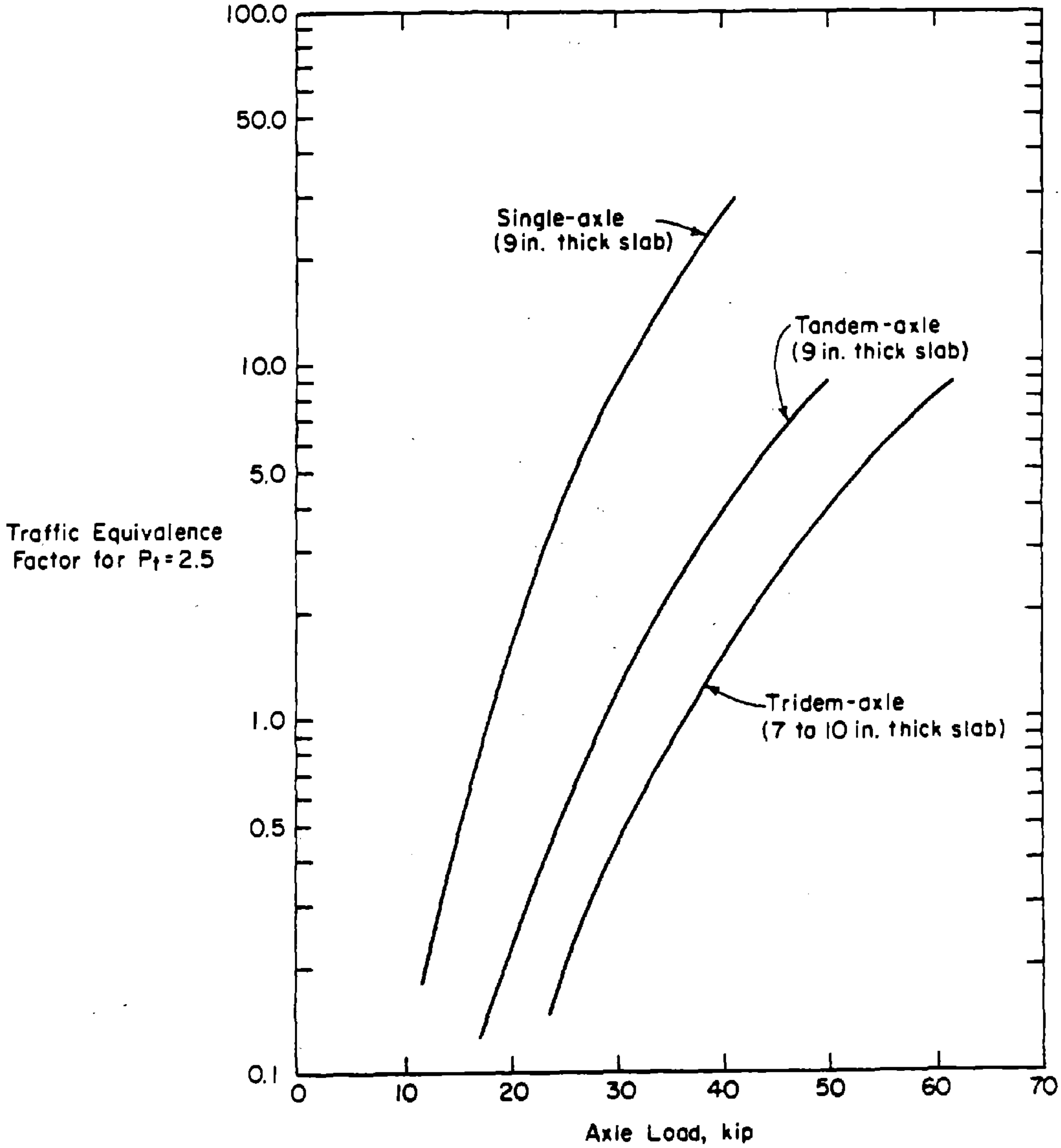
N_L = repetitions calculated by equation 41 in which the work strain is that due to the total load on the axle or group of axles.



1 kip=454 kg
1 in=25.4 mm

Note: Calculated maximum edge strain for 42 kip TAL is 42 micro-in.

Figure 69. Calculated edge strain profiles.[11]



1 kip=454 kg
1 in=25.4 mm

Figure 70. Comparison of traffic equivalence factors.[27]

Table 37. Traffic equivalence factors for tridem-axles on rigid pavement, p_t = 2.5.[27]

Tridem-Axle Load, kip	Traffic Equivalence Factor
30	0.43
32	0.55
34	0.70
36	0.91
38	1.20
40	1.44
42	1.68
44	2.16
46	2.64
48	3.12
50	3.77
52	4.32
54	5.04
56	6.00
58	7.20
60	8.06

p_t = terminal pavement serviceability index

1 kip=454 kg

Following the procedure described above, load equivalency factors were derived for various axle configurations as shown in figure 71. Using regression analyses, a relationship between the load equivalency factor, F_i, and the axle load, A_i (kip), was found to be:

$$\log F_i = a + b \log A_i + c (\log A_i)^2 \tag{43}$$

in which the regression coefficients a, b and c take the values shown in table 38.

The main purpose of the study was to determine the effects of uneven load distributions on the axles of tandems and tridems groups. It was found that the load equivalency factor from equation 43 should be adjusted by a multiplicative factor, MF, defined as:

$$\log (MF) = 0.0018635439 + 0.0242188935R - 0.0000906996R^2 \tag{44}$$

for tandem axle groups, where:

$$R = \frac{|(\text{Axle Load No. 1} - \text{Axle Load No. 2})|}{(\text{Axle Load No. 1} - \text{Axle Load No. 2})} \times 100$$

and

$$\log (MF) = a + b\,(\text{Ratio}) + c\,(\text{Ratio})^2 \tag{45}$$

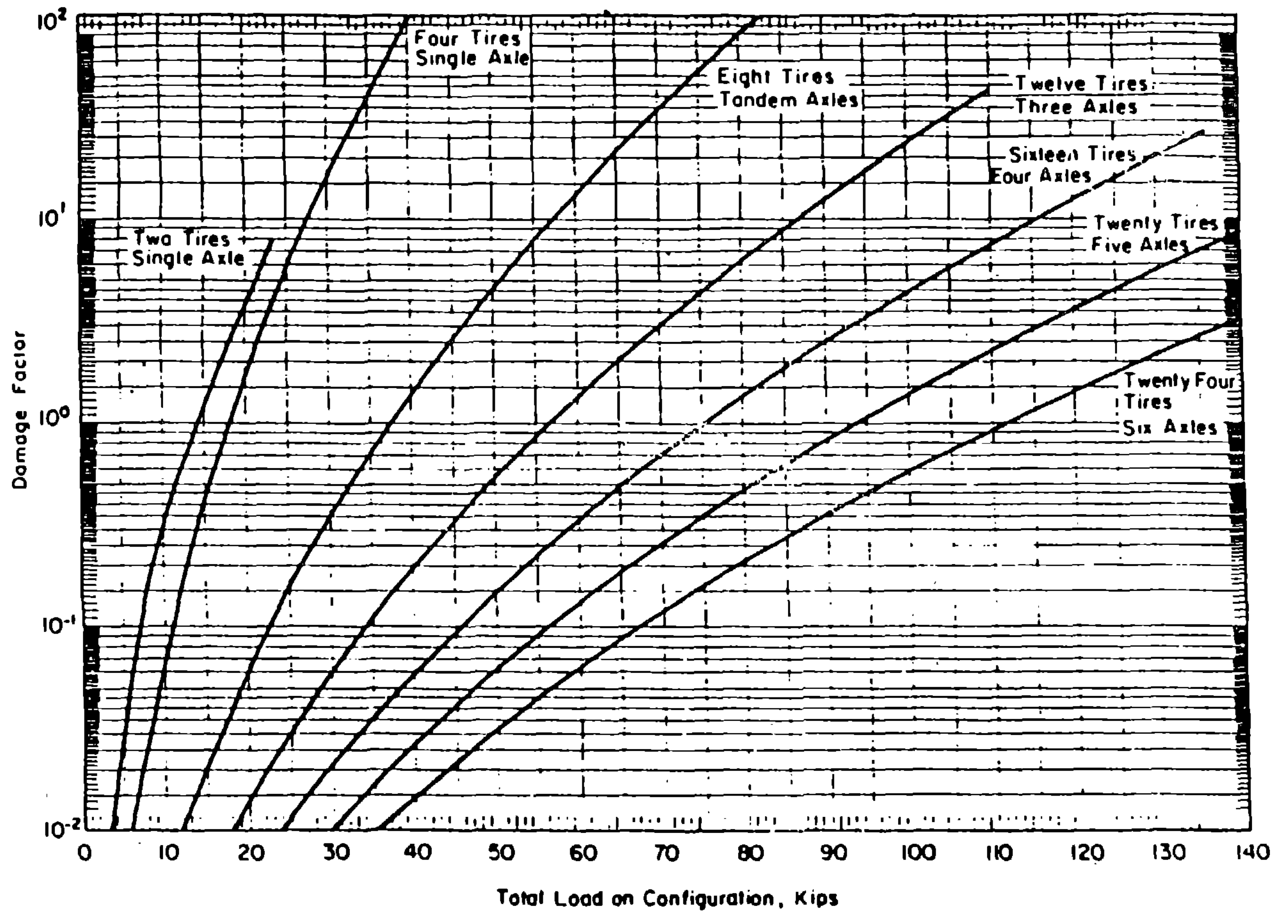
for tridem axle groups, where:

Ratio = (M - L)/I
M = Maximum axle load, kip
I = Intermediate axle load, kip
L = Least axle load, kip, and
a,b,c = regression coefficients (see table 39)

The conclusion of this study was that unevenly distributed loads on the axles within a tandem is very significant, especially for the tridem case.

<u>Gorge (1984)</u>

Gorge reported on preliminary results of a study carried out jointly by the Universities of Munich and Hannover and the vehicle manufacturer M.A.N. of W. Germany.[28] The study examined the impact of dynamic vehicle loading on pavement damage as indicated by the pavement response parameters measured in the pavement structure. One of the pavement sections was equipped with pairs of longitudinal and transverse strain gauges, placed 1.6-in (40-mm) below the surface of a 5.5-in (140-mm) thick asphalt concrete layer along a length of 59-ft (18-m). Another section was equipped with longitudinal strain gauges only, placed 0.4-in (10-mm) below the asphalt concrete surface spaced every 1.08-ft (.33-m) along a 85-ft (26-m) length. Also, a rigid concrete pavement (Federal Autobahn A7) was tested for fatigue damage.



1 kip=454 kg

Figure 71. Relationship between load equivalency and total load on the axle group and evenly distributed on all axles.[17]

Table 38. Regression coefficients to calculate damage factors for various axle configurations(7)

$$\text{Log(Damage Factor)} = a + b(\text{Log(Load)}) + c(\text{Log(load)})^2$$

AXLE CONFIGURATION	COEFFICIENTS a	b	c
Two-Tired Single Front Axle	-3.540112	2.728860	0.289133
Four-Tired Single Rear Axle	-3.439501	0.423747	1.846657
Eight-Tired Tandem Axle	-2.979479	-1.265144	2.007989
Twelve-Tired Tridem Axle	-2.740987	-1.873428	1.964442
Sixteen-Tired Quad Axle	-2.589482	-2.224981	1.923512
Twenty-Tired Quint Axle	-2.264324	-2.666882	1.937472
Twenty-four Tired Sextet Axle	-2.084883	-2.900445	1.913994

Table 39. Coefficents from regression analyses of unequal load distribution on individual axles of tridem axle group.[7]

$$\log(\text{Multiplying Factor}) = a + b(\text{Ratio}) + c(\text{Ratio})^2$$

in which Ratio = (M − L) / I
M = Maximum Axleload, kips,
I = Intermediate Axleload, kips,
L = Least Axleload, kips, and
a,b,c = coefficients

Load Pattern: 1. L,I,M 2. M,I,L 3. M,E,E 4. E,E,M	
Constant a	0.468782731
Coefficient b	1.093207072
Coefficient c	-0.1503124207
Standard Error of Estimate	0.073149
Correlation Coefficient, R	0.96024
F Ratio	1183.4
Load Pattern: 1. I,L,M 2. M,L,I 3. E,L,E	
Constant a	-0.1161216122
Coefficient b	1.507954095
Coefficient c	0.377814882
Standard Error of Estimate	0.069341
Correlation Coefficient, R	0.92765
F Ratio	326.9
Load Pattern: 1. L,M,I 2. I,M,L 3. E,M,E	
Constant a	-0.0235937584
Coefficient b	1.283412872
Coefficient c	-0.2187655038
Standard Error of Estimate	0.088165
Correlation Coefficient, R	0.92395
F Ratio	710.7
Load Pattern: 1. L,E,E 2. E,E,L	
Constant a	0.0004399421
Coefficient b	0.8053052125
Coefficient c	0.2363591702
Standard Error of Estimate	0.05634
Correlation Coefficient, R	0.96827
F Ratio	1037.4
Load Pattern: All Patterns Above	
Constant a	-0.198429071
Coefficient b	1.20191282
Coefficient c	-0.1746353238
Standard Error of Estimate	0.09792
Correlation Coefficient, R	0.9240
F Ratio	2085.4

Load equivalencies were derived on the basis of asphalt concrete fatigue life as ratios of "equivalent strain." Expressions for equivalent strain were derived to account for the two-dimensional strain state measured as opposed to the one-dimensional strain state in routine fatigue testing. Single axles with tire sizes of 10.00R22.5, 11.00R22.5 and 13.00R22.5 in single and dual configurations were tested. The variables considered were axle load, varying from 11- to 29-kip (4,994- to 13,166-kg), inflation pressure, pavement temperature, vehicle speed and lateral placement. It was shown that the higher the inflation pressure, the higher the equivalent strain.

To implement these findings into estimates of pavement damage, the 4th power law was modified to account for the effect of tire type, tire inflation pressure, and pavement roughness induced load variation, to obtain

$$v = (n_1 n_2 n_3 Q)^4 \tag{46}$$

where, v is the "dynamic load stress factor" corresponding to a load, n_1 expresses the influence of tire configuration, (i.e., equal to 1.0 and 0.9 for single and dual tires, respectively), n_2 expresses the effect of tire imprint contact pressure, p, with respect to a reference tire pressure of 100-psi (690-kPa), n_3 expresses the effect of dynamic load variation, and Q is the static axle load. Expressions for n_2 and n_3 are as follows:

$$n_2 = 0.0737p+0.490 \text{ (single axle)} \tag{47}$$

$$n_2 = 0.0317p+0.780 \text{ (tandem axle)} \tag{48}$$

$$n_3^4 = 1+6C^2 \tag{49}$$

where, p is the cold tire inflation pressure in N/mm^2 and C is the coefficient of variation of the dynamic load.

Some of the main conclusions of this study are:

- At the same axle load, dual tires reduce the road fatigue when compared to single tires.
- The higher the inflation pressure the higher the equivalent strain, but it is not very significant for flexible pavements. For flexible pavements an increase of 70 percent on inflation pressure (same axle load) caused only a 7 percent increase of strain.
- For both flexible and rigid pavements the strain related to road fatigue decreases with increasing speed, up to 25-mi/h (40-km/hr) then it remains practically constant.
- Contact pressure plays an important role on the rutting of flexible pavements.
- It is possible to produce heavier vehicle units having heavier axle loads, and to operate them without an increase in the road fatigue.

Battiato, Camomilla, Malgarini and Scapaticci, (1984)

Battiato et al reported findings from the experimental test site at Nardo, Italy.[6] The site is equipped with a number of strain gauges, vehicle speed sensors, temperature sensors and lateral vehicle position sensors. The strain gauges are positioned transversely with respect to the road center-line at 1.95-in (50-mm) intervals to ensure that in a single vehicle run a tire would pass directly over one of the sensors, (i.e., the tolerance was +-.98-in (+-25-mm)).

Preliminary experiments studied the effect of vehicle speed and pavement temperature on the absolute strain values induced from the steering axles of the vehicles. It was shown that the effect of vehicle speed and pavement temperature on strain can be quite substantial. To eliminate these effects strain ratios were calculated with respect to the strain caused by the 14-kip (6,350-kg) steering axle of each of the vehicles tested. It was found that the effect of speed on strain ratios is negligible and that all sensors respond "in the same manner" to variations in temperature. It was decided therefore to treat strain ratios as a whole without differentiating with respect to temperature, vehicle speed or sensor longitudinal position.

Load equivalencies were calculated with respect to a 12-ton (10,896-kg), load on a single axle with dual conventional tires. These results were then correlated with the axle load to obtain load equivalency factors, F_i, in terms of applied axle load

$$F_i = c\ W^a \tag{50}$$

It was found that the exponent a does not follow the fourth power law, instead it depends on the axle type and has a maximum value of 3.0.

Southgate and Deen (1985)

Southgate et al presents in addition to the "Work Strain" equations developed in adjustment factor equations to account for the spacing between two axles of a tandem group, and for the varying tire contact pressure[8, 7]

$$\log(adj) = -1.589745844 + 1.505262618 \log x - 0.3373568476 (\log x)^2 \tag{51}$$

$$\log(adj) = A + B \log p + C (\log p)^2 \tag{52}$$

where:

adj = adjustment factor to modify the load equivalency factor from equation 43
x = spacing between two axles of a tandem group, in.
p = tire contact pressure, psi
A,B,C = regression coefficients (see table 40)

Table 40. Regression coefficients to calculate adjustment factors for varying tire pressures and axle configurations for equally distributed tire loads.[8]

$$\log(\text{Factor}) = A + B*\log(\text{TCP}) + C*(\log(\text{TCP}))^2$$

Where TCP = Tire Contact Pressure

THICKNESS OF ASPHALTIC CONCRETE (inches)	COEFFICIENTS A	B	C
		FOUR-TIRED SINGLE AXLE	
3	-2.464465	0.576804	0.420942
4	-1.962926	0.591459	0.263080
5	-1.637979	0.612273	0.154626
6	-1.414034	0.635424	0.075089
7	-1.253049	0.659304	0.014209
8	-1.136604	0.683179	-0.033811
9	-1.049978	0.706696	-0.072534
10	-0.985633	0.729684	-0.104286
		EIGHT-TIRED TANDEM AXLES	
3	-2.573477	0.647141	0.414958
4	-2.221240	0.803333	0.224419
5	-1.889261	0.810996	0.116696
6	-1.579889	0.763381	0.054667
7	-1.291573	0.668360	0.020454
8	-1.022015	0.550498	0.004322
9	-0.760984	0.419143	0.000498
10	-0.538517	0.279885	0.005342
		TWELVE-TIRED TRIDEM AXLES	
3	-2.640784	0.686070	0.413835
4	-2.224371	0.777724	0.239410
5	-1.829065	0.730261	0.147497
6	-1.461152	0.614593	0.100533
7	-1.116870	0.462852	0.080565
8	-0.794540	0.291453	0.077889
9	-0.491654	0.109482	0.086793
10	-0.283964	-0.077749	0.103706

Yao (1985)

Yao presented a theoretical and experimental study on plain concrete pavements in China P.R.[29] Using regression analyses, a load equivalency factor, F_i, was found in terms of the applied axle load, P_i, and the standard single axle load, P_s, assumed to be 22.5-kip (10,215-kg).

$$F_i = c \ (P_i/P_s)^{16} \tag{53}$$

where

C = 1.0 for single axle load
= 0.23 for interior loading - tandem axle load
= 3.8 for transverse edge loading - tandem axle load

Sharp, Sweatman and Potter (1986)

Sharp et al carried out an experimental study to evaluate the relative damaging effect of triple axles with conventional dual and wide-base tires.[30] The study was motivated by the spreading use of wide-base tires in six-axle semitrailer tank trucks used across Australia for fuel distribution.

Load equivalency was determined on the basis of pavement surface deflection. Surface deflections were measured at pavement sites instrumented with DCDT's, (Direct Current Displacement Transducers), and TPIs (Transverse Position Indicators). Two six-axle semitrailer trucks were used for testing with tire sizes ranging from 11.00R22.5 to 18.00R22.5 all inflated at the same pressure of 100-psi (.070-kg/mm^2) under cold conditions. Deflection testing took place only in two sites, one with conventional asphalt concrete, 3/6/8-in (75/150/200-mm), and one with a spray seal over 14-in (350-mm) of granular base. The methodology followed was to calculate displacement ratios under the load in question and a reference load. The Benkelman Beam axle was used as the reference load, (i.e., 18-kip (8,160-kg) on dual 10.00 by 20 tires inflated at 80-psi (.056-kg/mm^2)). Subsequently, regression equations were developed between deflection ratios and the corresponding loads. Different loads were considered equivalent if they induced identical deflection ratios.

During testing, vehicle lateral placement was monitored with a TPI, specially developed for the purposes of the study. To ensure that the true maximum strain values were obtained, curves were fitted to the deflection versus lateral placement data. It was also observed that deflection readings for multiple axles increased from the leading to the last axle. The maximum deflection for the axle group was calculated as the "pooled" average of the peak deflection values of the individual axles.

The findings of the study are briefly outlined as follows:

1. For a given load, single tires with nominal widths of 15- to 18-in (375- to 450-mm) produced higher deflections than dual tires with widths of 11-in (275-mm).

2. For single axles, a load of 13,000-lb (5,902-kg) on single wide-base tires was considered equivalent to a load of 18,000-lb (8,160-kg) on conventional dual tires.

3. For tandem axle groups, a load of 24,000-lb (10,896-kg) on wide-base tires was found equivalent to a load of 30,000-lb (13,620-kg) on dual tires on spray seal pavements.

4. For triaxle groups, a load of 37,500-lb (17,025-kg) on single wide-base tires was considered equivalent to a load of 41,000-lb (18,614-kg) on dual tires.

5. Pavement deflection decreases with speed but the ratio (to the standard) decreases, particularly for wide single tires.

Hudson, Seeds, Finn and Carmichael (1986)

Hudson et al presented a theoretical study to develop new load equivalency factors for ADOT.[9] The following parameters were considered in the analysis:

1. Load (lb): 4,000, 10,000, 18,000, 30,000, 50,000.
   (kg: 1,816, 4,450, 8,172, 13,620, 22,700).
2. Tire Pressure (psi): 75, 110, 145.
   (kPa: 515, 760, 1,000).
3. Modulus of Roadbed Soil (psi): 4,000, 12,000, 20,000.
   (kg/mm^2: 2.8, 8.4, 14.1).
4. Subbase/Base Thickness (in): 4/4, 6/8, 8/12.
   (mm: 102/102, 152/203, 203/305).
5. AC thickness (in): 0, 3, 6.
   (mm: 0, 76, 152).
6. Axle type: Single axle single tire, single axle dual tire, tandem axle, and tridem axle.

Separate damage models for both single and tandem axle loads were obtained using the following mechanistic responses:

- Maximum asphalt concrete tensile strain, ϵ_{AC}.
- Maximum asphalt concrete tensile stress, σ_{AC} (psi).
- Maximum asphalt concrete shear strain, γ_{AC}.
- Maximum asphalt concrete shear stress, τ_{AC} (psi).
- Maximum vertical strain on roadbed soil, ϵ_{RS}.

The critical asphalt concrete mechanistic responses were calculated using ELSYM5, an elastic layer theory based computer program.

Given the damage models shown in tables 62 and 63, the technique for generating 18-kip (8,172-kg) single axle equivalence factors for a variety of conditions is relatively simple. An equivalence factor is a ratio of the relative damage between a given loading condition (x/c/p), and a standard 18-kip (8,172-kg) single axle load. (Note: "x" refers to the load magnitude, "c" to the load configuration and "p" to the tire pressure.)

Table 41. Initial single axle damage models resulting from DAMOD-4 computer analysis.[9]

Form of Damage Model

$$\log(N_f) = a_0 + a_1 * \log(R) + a_2 * \log(E_{AC})$$

Mechanistic Response Considered	Symbol (R)	Optimum Coefficients a_0	a_1	a_2	Coefficient of Determination (r^2)
Asphalt Concrete Tensile Strain	ε_{AC}	6.89	-6.21	-3.97	0.599
Asphalt Concrete Tensile Stress	σ_{AC}	4.68	-6.40	2.80	0.615
Asphalt Concrete Shear Strain	γ_{AC}	8.96	-6.43	-4.20	0.584
Asphalt Concrete Shear Stress	τ_{AC}	6.69	-6.28	2.10	0.562
Vertical Strain on Roadbed Soil	ε_{RS}	(Model not possible)			-

Table 42. Single axle damage models resulting from DAMOD-4 computer analysis on data without frozen-winter effects.[9]

Form of Damage Model

$$\log(N_f) = a_0 + a_1 * \log(R) + a_2 * \log(E_{AC})$$

Mechanistic Response Considered	Symbol (R)	Optimum Coefficients a_0	a_1	a_2	Coefficient of Determination (r^2)
Asphalt Concrete Tensile Strain	ε_{AC}	3.25	-7.50	-4.10	0.834
Asphalt Concrete Tensile Stress	σ_{AC}	2.69	-7.47	3.60	0.841
Asphalt Concrete Shear Strain	γ_{AC}	6.61	-7.72	-4.50	0.829
Asphalt Concrete Shear Stress	τ_{AC}	3.85	-7.62	3.10	0.819
Vertical Strain on Roadbed Soil	ε_{RS}	-7.75	-4.28	-	0.723

The equivalence factor for load (x/c/p), therefore, may be calculated as the ratio of the allowable 18-kip (8,172-kg) single axle load applications to the allowable applications for load (x/c/p):

$$e_{x/c/p} = \frac{(N_f)_{18/1/75}}{(N_f)_{x/c/p}} \tag{54}$$

$(N_f)_{18/1/75}$ is calculated for the selected structural and soil support conditions using the single axle damage model with a standard 75-psi (515-kPa) tire pressure, 18-kip (8,172-kg) single axle as the load. $(N_f)_{x/c/p}$ is calculated (for the same structural and soil support conditions) using the appropriate single or tandem axle damage model along with the load magnitude (x) and tire pressure (p) corresponding to load (x/c/p). Two sets of damage models were used in the development. For 3- and 6-in (76- and 152-mm) surface thicknesses, the set of models with tensile strain at the bottom of the asphalt layer as the response parameter was used. For thin surface treatments the models with vertical strain at the subgrade as response parameter was used. Figure 72 illustrates the equivalence factor development process. The following example is provided to demonstrate the technique.

Suppose we have a pavement structure consisting of 3-in (76.2-mm) of asphalt concrete, 6-in (152.4-mm) of base and 8-in (203.2-mm) of subbase in a weak roadbed soil environment (E_{RS} = 4,000-psi (2.81-kg/mm^2)). Suppose also that we want to calculate the equivalence factor for a 30-kip (13,620-kg) tandem axle having a 110-psi (.077-kg/mm^2) tire pressure. Assuming reasonable subbase, base and asphalt concrete moduli of 8,000-psi, 12,000-psi and 450,000-psi, (5.6-, 8.4-, and 316.6-kg/mm^2) respectively, the critical asphalt concrete tensile strains that would be calculated using an elastic layer theory based computer program (e.g., ELSYM5) are 5.111×10^{-4} for the standard 18-kip (8,172-kg) single axle and 5.179×10^{-4} for the 30-kip (13,620-kg) tandem axle. $(N_f)_{18/1/75}$ determined using the single axle damage model is 57,270 and $(N_f)_{30/2/110}$ from the tandem axle model is 1,844. Thus, the tandem axle equivalence factor for these conditions is:

$$e_{30/2/110} = 57,270/1,844 = 31 \tag{55}$$

Unfortunately, since tridem axle loads were not considered in the AASHO Road Test experiment, it was not possible to develop a damage model based on tridem axle loads. Nevertheless, the mechanistic nature of the figure 72 damage models used to generate the single and tandem axle load equivalence factors made it essential that some compatible set of load equivalence factors be established for tridem axle loads. Five different options were identified to determine the factors. All options depend on some extrapolation of the single and tandem axle load equivalence factors:

<u>Option 1</u>: Use tandem axle equivalence factors for tridem axle loads. Of all the options, this was the least attractive because it is too conservative in that it does not give any benefit to having another axle to distribute the load to.

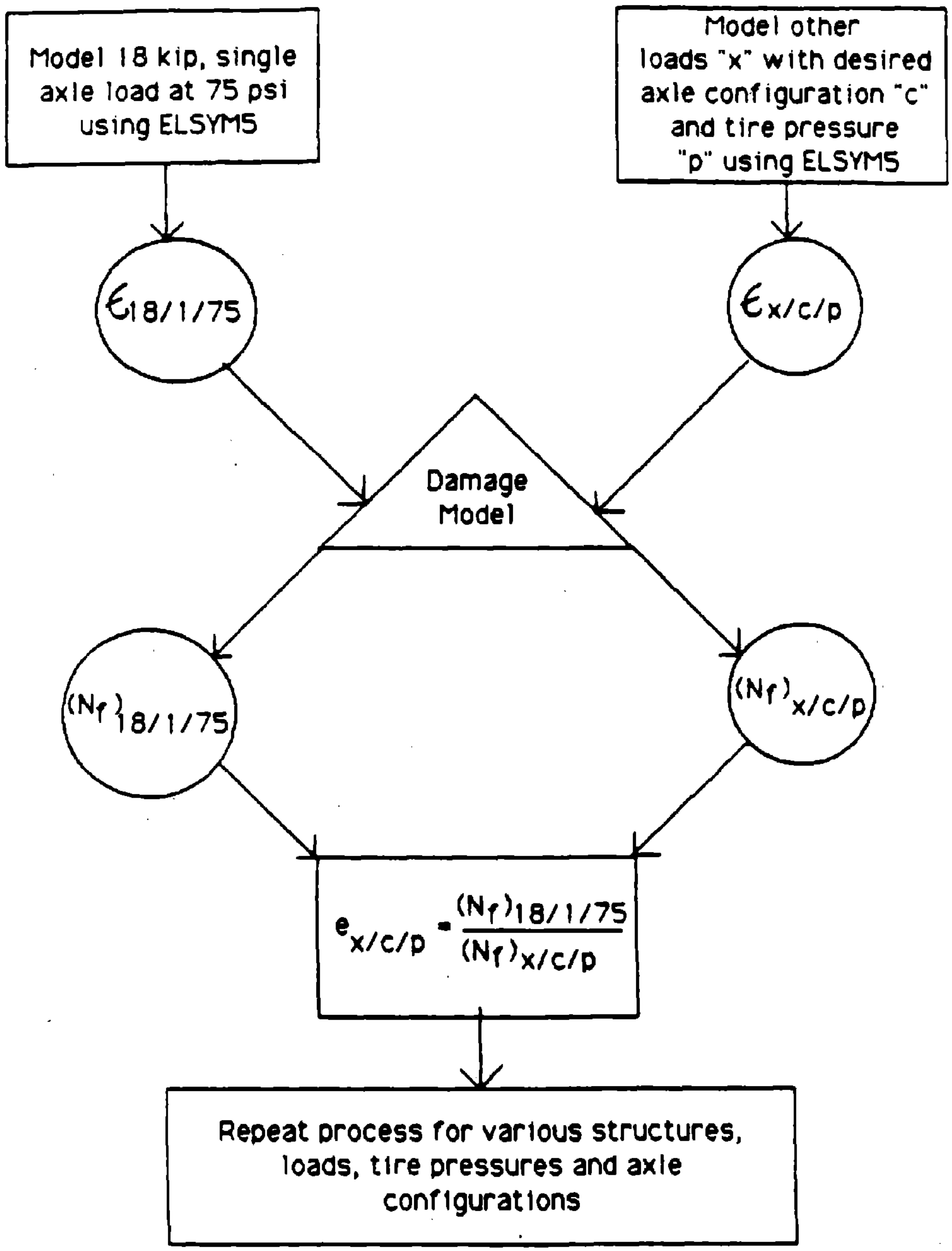


Figure 72. Illustration of equivalence factor development process.[9]

Option 2: Determine the tandem axle load equivalence factor for two-thirds the tridem axle load and increase by 50 percent to account for the third axle:

$$e_{x/3/p} = 1.5 * e_{.666x/2/p} \tag{56}$$

As an example for a roadbed soil modulus of 4,000-psi (2.8-kg/mm^2) and a pavement structure consisting of an 8-in/8,000-psi (203-mm/5.6-kg/mm^2) subbase, 6-in/12,000-psi (152-mm/8.4-kg/mm^2) base and a 3-in/450,000-psi (76-mm/316.6-kg/mm^2) asphalt concrete surface, the equivalence factor for a 30-kip (13,620-kg) tridem axle load with 75-psi (515-kPa) tire pressure, $e_{30/3/75}$, would be 1.5 times the equivalence factor for two-thirds the tandem axle load 20-kip (9,080-kg)):

$$e_{30/3/75} = 1.5 * e_{20/2/75} = 1.5 * (0.309) = 0.464 \tag{57}$$

Option 3: Determine the single and tandem axle load equivalence factors for one-third and two-thirds the load, respectively, then add the two together:

$$e_{x/3/p} = e_{.333x/1/p} + e_{.666x/2/p} \tag{58}$$

Using the 30-kip (13,620-kg) tridem axle load as an example:

$$e_{30/3/75} = e_{10/1/75} + e_{20/2/75} = 0.064 + 0.309 = 0.373 \tag{59}$$

Option 4: Determine the ratio of the tandem axle to the single axle load equivalence factor and assume that the ratio is the same as the ratio of the tridem axle to the tandem axle load equivalence factor:

$$e_{x/3/p} = e_{x/2/p} * \frac{e_{x/2/p}}{e_{x/1/p}} \tag{60}$$

Again using the 30-kip (13,620-kg) tridem axle as an example:

$$e_{30/3/75} = e_{30/2/75} * \frac{e30/2/75}{e30/1/75} = 1.592 * \frac{1.592}{17.00} = 0.149 \tag{61}$$

Option 5: Determine the ratio of the actual tandem axle equivalence factor to the expected tandem axle equivalence factor obtained from two single axles having half the tandem axle load. Then, multiply this ratio by the expected tridem axle load equivalence factor obtained from 1.5 tandem axles having two-thirds the load:

$$e_{x/3/p} = \frac{e_{x/2/p}}{(2*e_{.5x/1/p})} * 1.5*e_{.666x/2/p}) \tag{62}$$

The solution for the 30-kip (13,620-kg) tridem axle load would, in this case, be:

$$e_{30/3/75} = \frac{e_{30/2/75}}{(2*e_{15/1/75})} * (1.5*e_{20/2/75}) \tag{63}$$

$$= \frac{1.592}{(2*0.380)} * (1.5 * 0.309)$$

$$= 0.972$$

Figure 73 provides a plot of equivalence factor versus load for the five options. Based on an examination of these results, Option 5 was selected as the best model for estimating tridem axle load equivalency factors for Arizona.

Christison (1986)

Christison reports on load equivalency factors developed using field data collected at 14 sites across Canada during the summer of 1985 (see table 43).[2] The approach used to derive the equivalency factors, F_i, is similar to the method Christison used in 1978.[22] The exponent of 3.0 used in 1978 has been estimated as 3.8 for this study leading to the following equations:

$$F_i = \sum_{i=1}^{n} (S_i/S_b)^{3.8} \qquad \text{Single \& multiple axles} \tag{64}$$

$$F_i = (D_i/D_b)_{n-1}^{3.8} \qquad \text{Single axles} \tag{65}$$

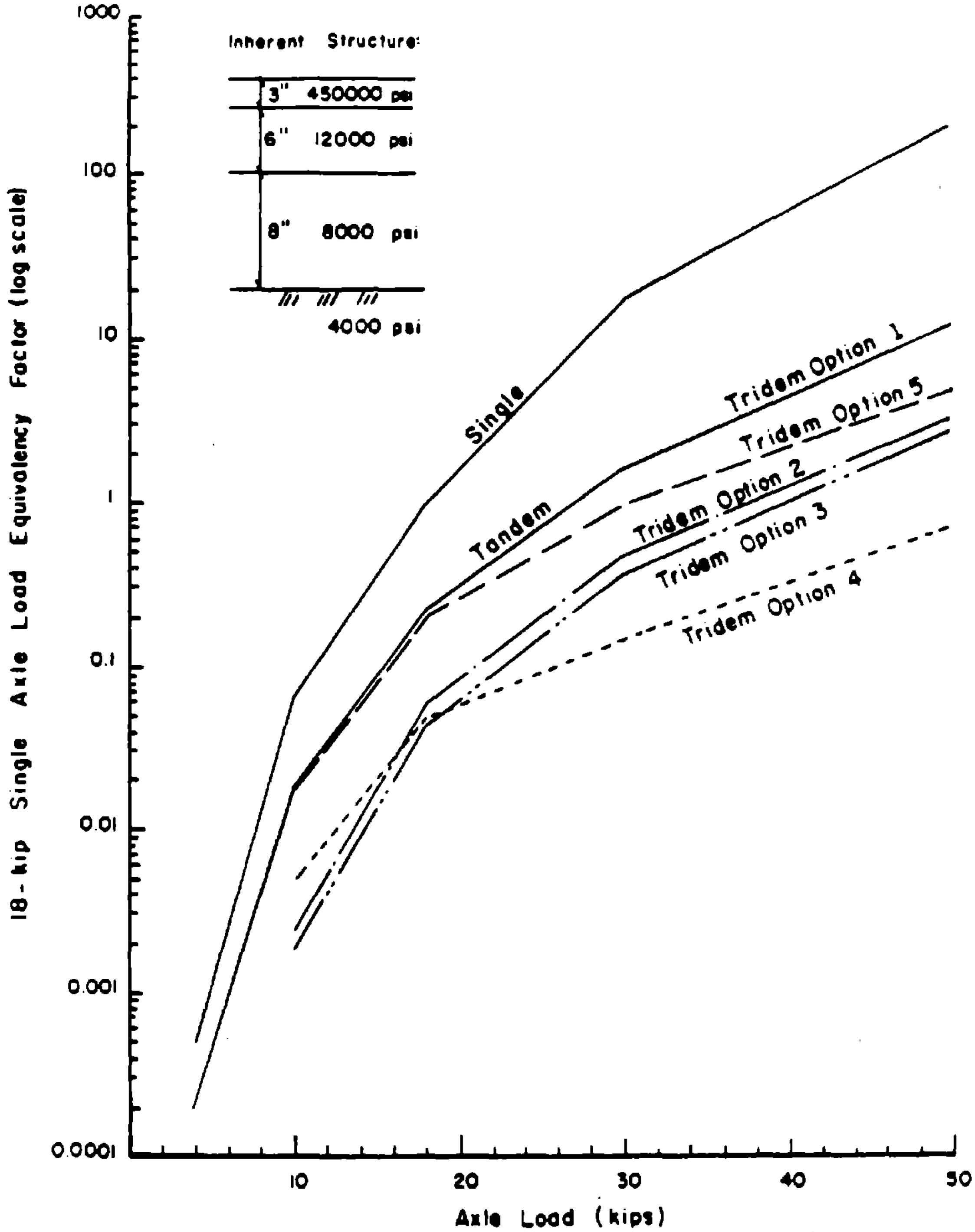
$$F_i = (D_i/D_b)^{3.8} + (\sum_{i=1} {}_i/D_b)^{3.8} \qquad \text{Multiple axles} \tag{66}$$

where:

S_i = Longitudinal interfacial tensile strain recorded under the applied axle load (see figure 74) or leading axle of the axle group under consideration.
S_b = Longitudinal interfacial tensile strain recorded under the standard load.
D_i = Pavement surface deflections caused by the applied single axle load or the leading axle of the axle group under consideration.
$_i$ = Difference in magnitude between the maximum deflection recorded under each succeeding axle and the minimum residual deflection preceding the axle (see figure 75).
D_b = Pavement surface deflection caused by the standard 9-kip (4072-Kg) single axle-dual tire load of the Benkelman Beam vehicle.
n = Number of axles in the axle group.

Measured values were used to substitute in the above equations to obtain equivalency factors. Using least squares regression analyses, gross weight versus equivalency factor relationships were developed. The general form of these relationships were:

$$F_i = k(W_i)^C \tag{67}$$



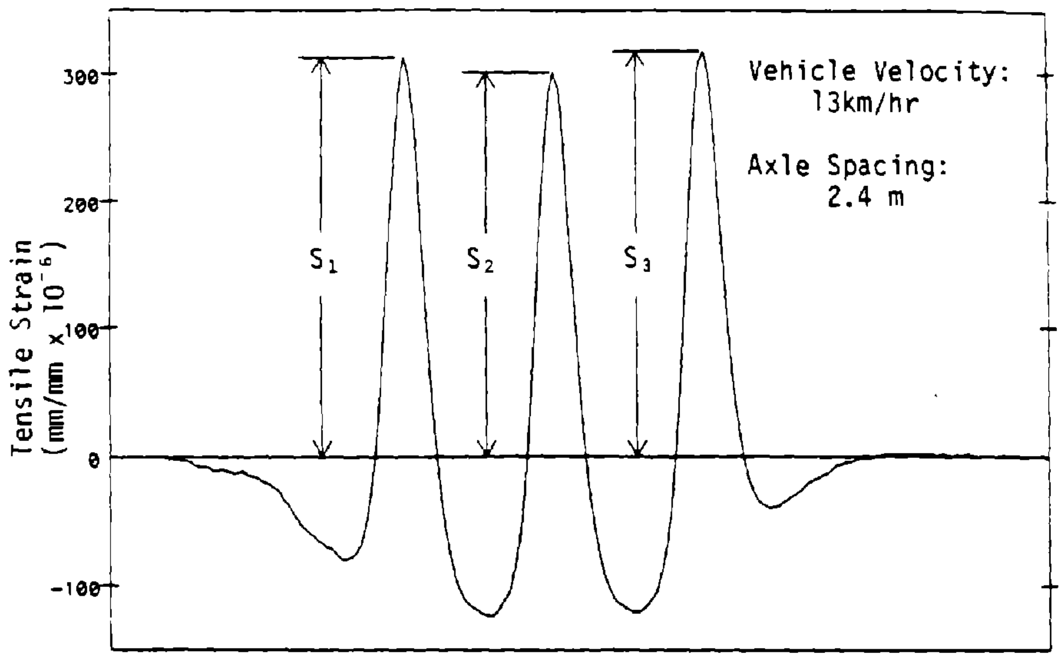
1 kip=545 kg
1 psi=6.9 kPa

Figure 73. Plot of 18-kip (8,160-kg) single axle load equivalence factor versus axle load illustrating five tridem axle equivalance factor options.(9)

Table 43. Pavement test sites.[2]

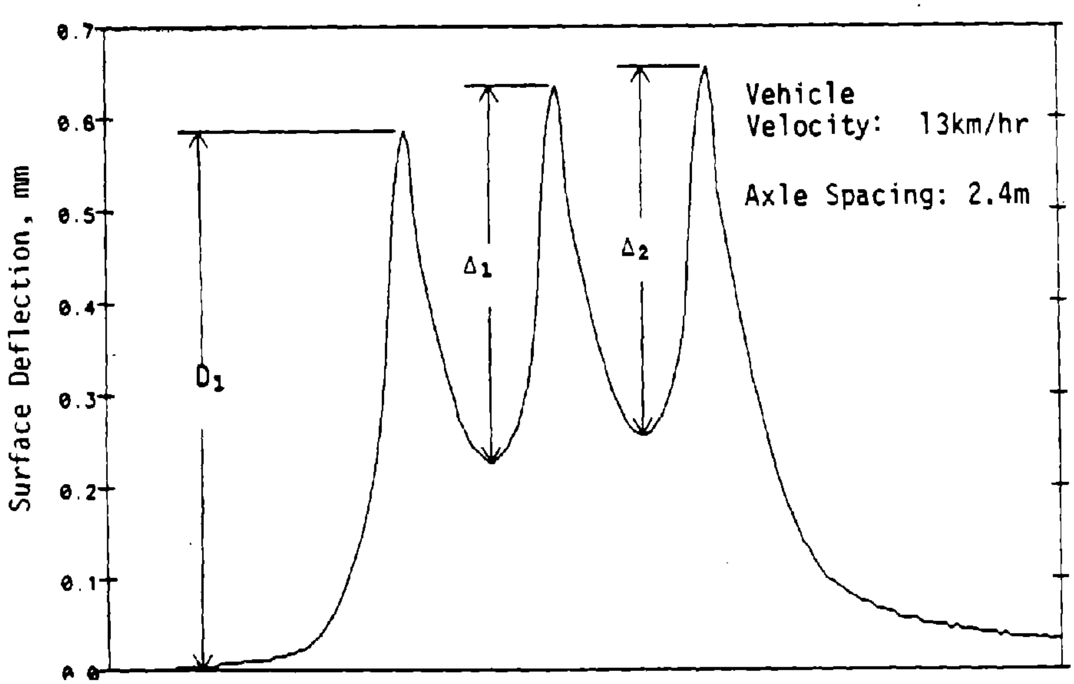
Site No.	Province	Location	Structure			
			A.C. Thick.(mm)	Base Thick.(mm)-Material	Sub-Base Thick.(mm)-Material	Subgrade Material
1	New Brunswick	Hwy.15 - 10 km. E. of Moncton	225	76 - Crushed rock	460 - Crushed sandstone	Silty-sand
2	Nova Scotia	Hwy.102 - 6 km. S. of Truro	160	275 - Granular	200 - Granular	Gravelly-clay
3A	Quebec	Hwy.40 - 55 km. W. of Quebec City	135	200 - Crushed limestone	625 - Granite sand	Granitic-gravel
3B	Quebec	Hwy.40 - 55 km. W. of Quebec City	130	375 - Crushed limestone	450 - Granitic sand	Granitic-gravel
4	Quebec	Rte.363 - 73 km. W. of Quebec City	56	150 - Granitic gneiss	450 - Granitic sand	Clay
5	Quebec	Rte.363 - 73 km. W. of Quebec City	56	200 - Granitic gneiss	550 - Granitic sand	Clay
6	Ontario	Hwy.7-Peterborough Bypass	110	150 - Granular A	350 - Granular C	Silty-sand
7	Ontario	Hwy.403 - 19 km. W. of Brantford	170	200 - Granular A	250 - Granular B	Sand
8	Ontario	Hwy.55 - 8 km. E. of St. Catharines	190	300 - Granular A	90 - Old road	Clay
9	Alberta	Hwy.21 - 8 km. N. of Three Hills	136	170 - Cement Stab. Sand	- -	Clay
10	Alberta	Hwy.21 - 8 km. N. of Three Hills	136	250 - Granular	- -	Clay
11	British Columbia	Hwy.97 - 110 km. W. of Chetwynd	75	145-Asphalt bnd.gran. 200 - Granular	610 - Granular 1000 - Shot rock	Peat/Silty Sand
12	British Columbia	Hwy.97 - 112 km. W. of Chetwynd	85	155-Asphalt bnd.gran. 210 - Granular	610 - Granular 975 - Silty gravel	Silty-sand
13	British Columbia	Hwy.16 - 16 km. N.W. of Tete Jaune Cache	100	545 - Granular	50 - Clay and sand 450 - Pit run gravel	Clay

1 in=25.4 mm



1 in=25.4 mm
1 ft=0.305 m
1 mi/h=1.61 km/h

Figure 74. Longitudinal interfacial tensile strain profile triaxle configuration.[2]



1 in=25.4 mm
1 ft=0.305 m
1 mi/h=1.61 km/h

Figure 75. Surface deflection profile - triaxle configuration.[2]

where:

W_i = gross weight in Kg * 1000
k and C = constants

The values of the constants k and C are not only response dependent but also site dependent. If deflections are considered the values vary from 2.207 to 3.02 for C, and 0.00023 to 0.0040 for k. If strains are considered the values vary from 1.2318 to 3.405 for C, and from 0.000153 to 0.1149 for k.

Some of the conclusions from this study are:

- For a given tandem load with axle spacing between 5-ft (1.5-m) to 6-ft (1.8-m) the magnitude of the equivalent single axle load is 58 percent.
- For the range of axle spacing included in the study [4-ft (1.2-m) to 6-ft (1.8-m)], the influence of variations of axle spacing on potential pavement damage is dependent on the pavement response criteria (i.e., deflection or strain).
- The pavement structure has more effect on equivalency factors based on strain ratios than on equivalency factors based on deflection ratios. The magnitude of the strain ratios tended to decrease with increasing asphalt concrete thickness, T (mm),:

$$\log F_i = 0.578 + 0.0155\,T\,(\log W_i) - 0.0669\,T \quad (68)$$

Hutchinson, Haas, Meyer, Hadipour and Papagiannakis (1987)

Hutchinson et al, using the data collected at 14 sites across Canada, developed load equivalency factors for different axle loads, axle groups and vehicles.[10, 31, 2]

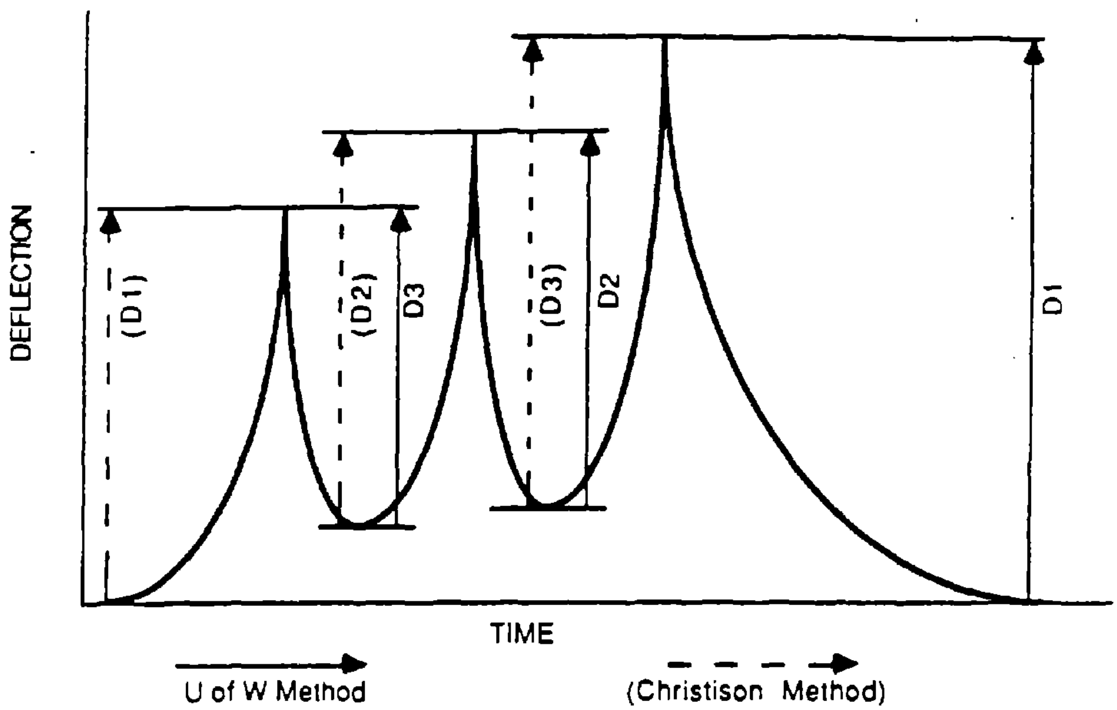
The main difference between this study and Christison is the use of cycle counting to accumulate the damage induced in flexible pavements by the passage of different axle groups (see figure 76).[2] The load equivalency factor in terms of surface deflections is given by:

$$F_i = (D_{1i}/D_s)^{3.8} + (D_{2i}/D_s)^{3.8} + (D_{3i}/D_s)^{3.8} \quad (69)$$

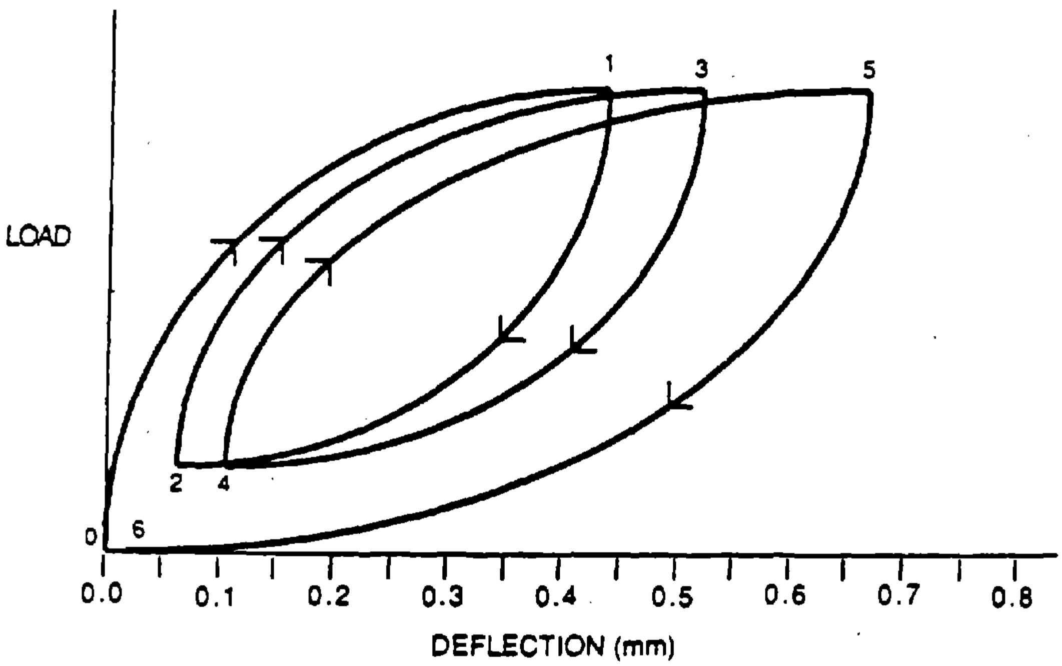
where:

F_i = The load equivalency factor for candidate axle group i.
D_{1i} = The deflection observed under axle group i for the largest load - deflection cycle.
D_{2i} = The deflection observed under axle group i for the second largest load - deflection cycle.
D_{3i} = The deflection observed under axle group i for the third largest load - deflection cycle.
D_s = The deflection observed under the standard axle load.

Regression equations between load equivalency factors and load data were obtained using the load equivalency factors found using the above



a. Variation in pavement surface deflection under tridem.



1 in=25.4 mm

b. Hypothesized load-deflection history for tridem pass.

Figure 76. Load and surface deflection for tridem.[10]

equation. For a 5-ft (1.5-m) axle spacing in a tandem axle group the equation became:

$$F_i = AL^B \tag{70}$$

where:

L = Load in metric tonnes.
A and B = Parameters estimated from regression analysis (see table 44).

Other factors such as pavement temperatures and test speeds were also considered. Non-linear regression equations for site six for tandems and for tridems were obtained:

Tandem

$$F_i = 0.0002703\ L_i^{2.3909}\ T^{0.6867}\ V_i^{-0.04979} \tag{71}$$

Tridem

$$F_i = 0.0003278\ L_i^{2.1291}\ T^{0.6700}\ V_i^{-0.06135} \tag{72}$$

where:

F_i = Load equivalency factor for axle group i.
L_i = Load in metric tonnes on axle group i.
T = Pavement temperature in degrees Celsius.
V_i = Vehicle velocity in km/h.

Some of the most important conclusions of this study are:

- Load equivalency factors from observed pavement responses is sensitive to the method used to isolate and count damage cycles under multiple axle groups as well as to the exponent used for the cumulative damage function.

- Load equivalency factors are significantly affected by temperature and vehicle speed.

Rilett and Hutchinson (1988)

Rilett et al reported load equivalency factors for single, tandem and tridem axle groups.[32] The functions were developed from truck loading test data collected across Canada in 1985 by the Canroad Transportation Research Corporation.[31, 2] The damage accumulation was based on the cycle counting method and the analysis procedure was similar to Hutchinson.[10] The exhaustive statistical analysis of the data base revealed that the load on the axle groups dominated the regression equations.

A regression analysis of the pooled data for all tandems resulted in significant exponents for load (L_i, in 1000s of Kg) and axle spacing (x in meters) but with load dominating the regression equation:

$$F_i = 0.0013563 * L_i^{2.698} * x^{-0.396} \tag{73}$$

Table 44. Parameters of LEF versus load functions.[10]

Site Number	A	B	R^2
1	0.000501	3.0778	0.65
3A	0.007509	2.0449	0.82
3B	0.004321	2.2247	0.90
4	0.000642	2.9002	0.86
5	0.001703	2.5499	0.73
6	0.002396	2.4049	0.92
7	0.001348	2.5437	0.86
8	0.000142	3.3166	0.90
9	0.000267	3.3176	0.89
10	0.000935	2.7683	0.94
11	0.001072	2.6680	0.91
13	0.006174	2.0556	0.79

The load equivalency factor, F_i, for the single axles had a rather low explanatory power, probably due to the narrow range of loads tested:

$$F_i = 0.0153598 * L_i^{2.159} \tag{74}$$

Analyses of the pooled data for the tridems resulted in statistically significant exponents for load (L_i, in 1000s Kg), axle spacing (x, in m), structural number SN and speed (V_i, in km/hr).

$$F_i = 0.0008276 * L_i^{2.669} * x^{-0.168} * (SN)^{-0.251} * V^{0.074} \tag{75}$$

Comparisons of the load equivalency factors developed in this study with those of AASHTO (figure 77) revealed that tandem and tridem load equivalency factors were higher than AASHTO.

Majidzadeh and Ilves (1988)

Majidzadeh et al presents a critique on the damage accumulation method used by Christison.[33, 2] For simplicity the exponent on equations was changed from 3.8 to 4.0.

$$F_i = \mathop{\quad}_{i=1}^{n} (D_i/D_s)^{4.0} \tag{76}$$

$$F_i = (D_i/D_s)^{4.0} + \mathop{\quad}_{i=1}^{n-1} (\quad _i/D_s)^{4.0} \tag{77}$$

A theoretical study using the above equations for different axle spacings (but with the same load) for the response curve presented in figure 78 was carried out. The load equivalency factors are plotted in figure 79 as a function of the axle separation. It will be noted from figure 79 that the discrepancy in the load equivalency factors calculated from equation 73 is exactly 1 (at axle spacing of 4-ft (1.2-m)). This arises because the shape of the response curve has 2 peaks at separation just greater than 4-ft (1.2-m), with the magnitude of the first peak remaining constant at 1 as axle separation increases.

Because of this discrepancy and discrepancies found among single, triple and tandem axles, it seems that a completely satisfactory scheme for defining primary response load equivalency factors cannot be obtained from measuring peak deflections.

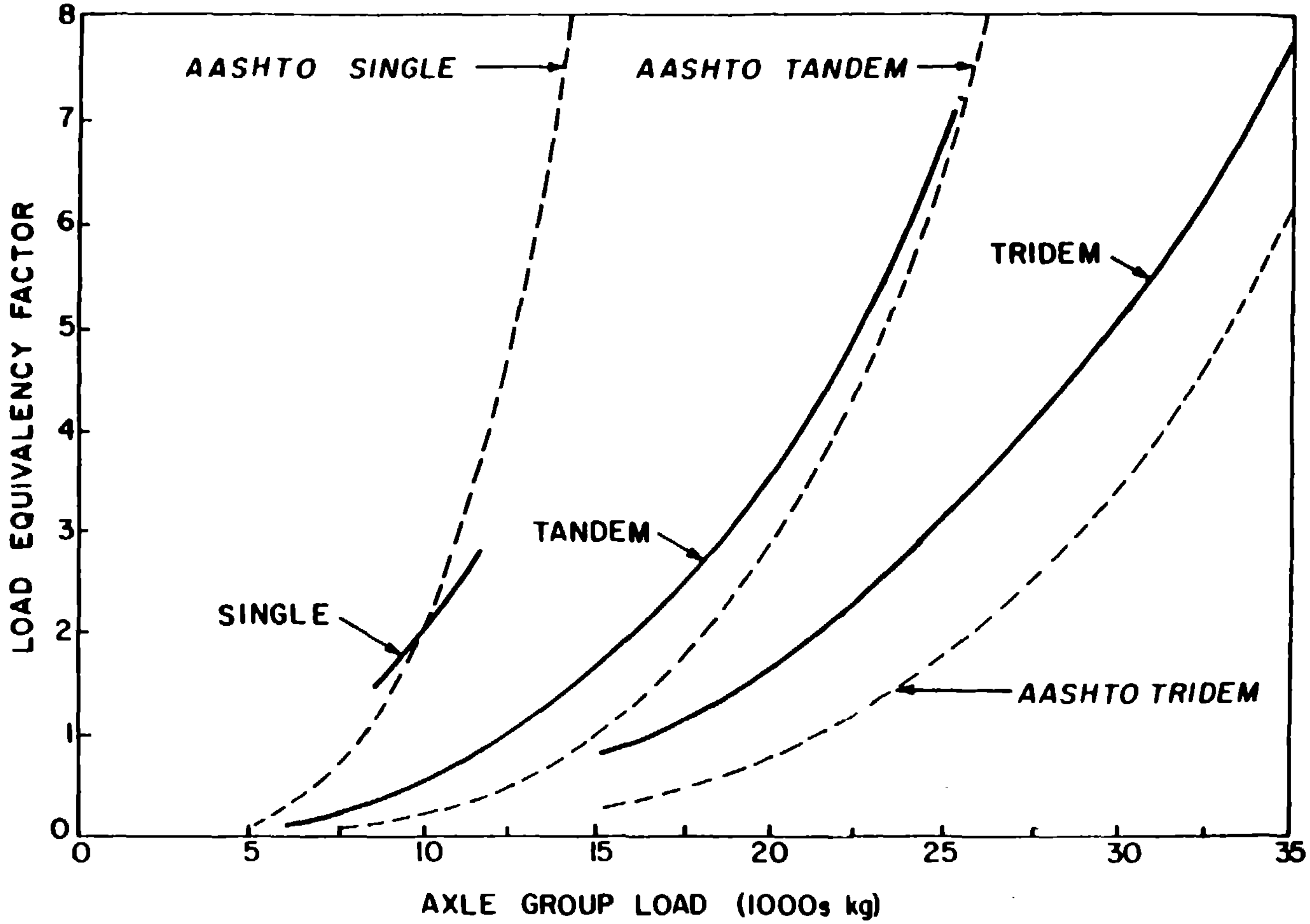


Figure 77. Comparison of LEF functions with AASHTO functions.(32)

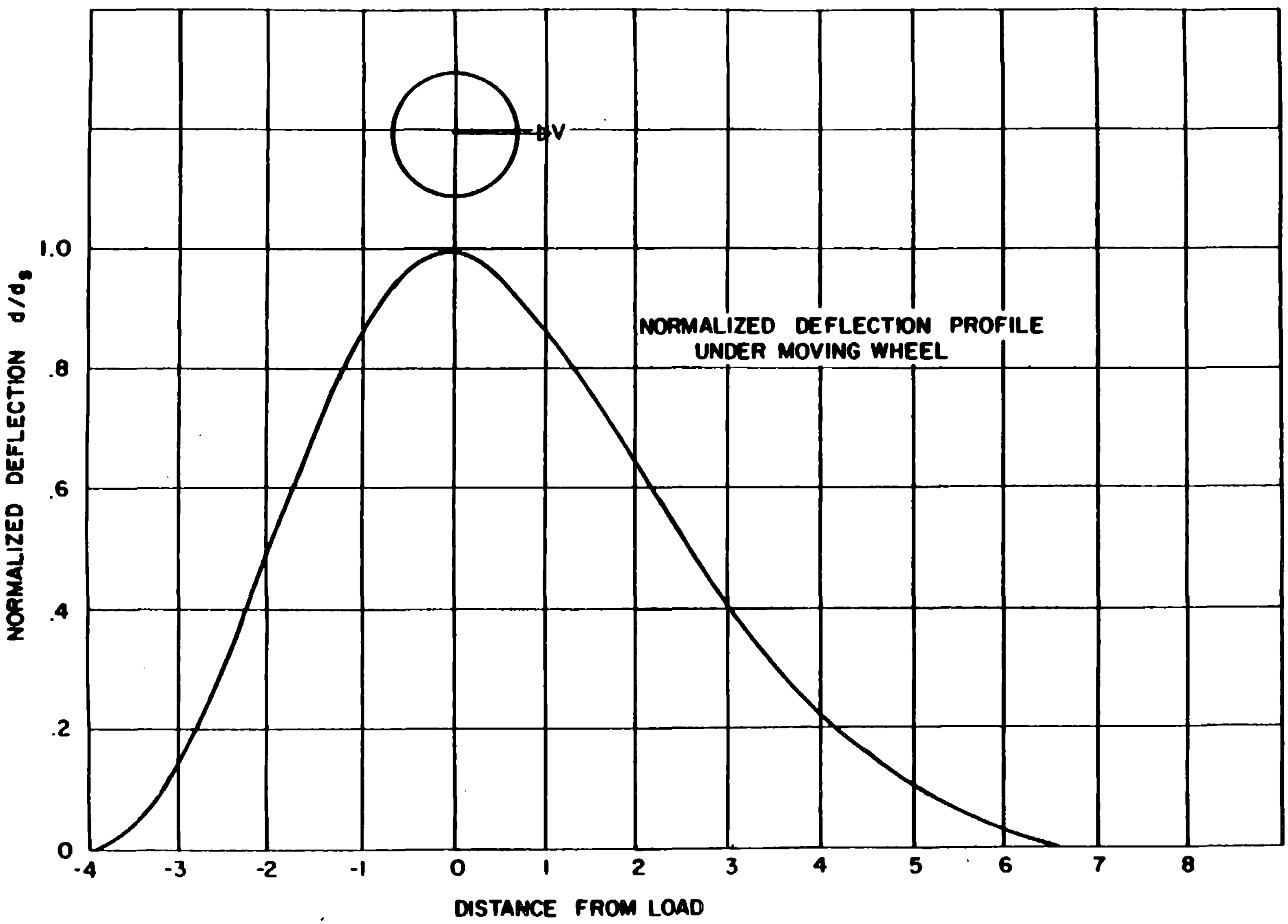


Figure 78. Assumed deflection profile.[33]

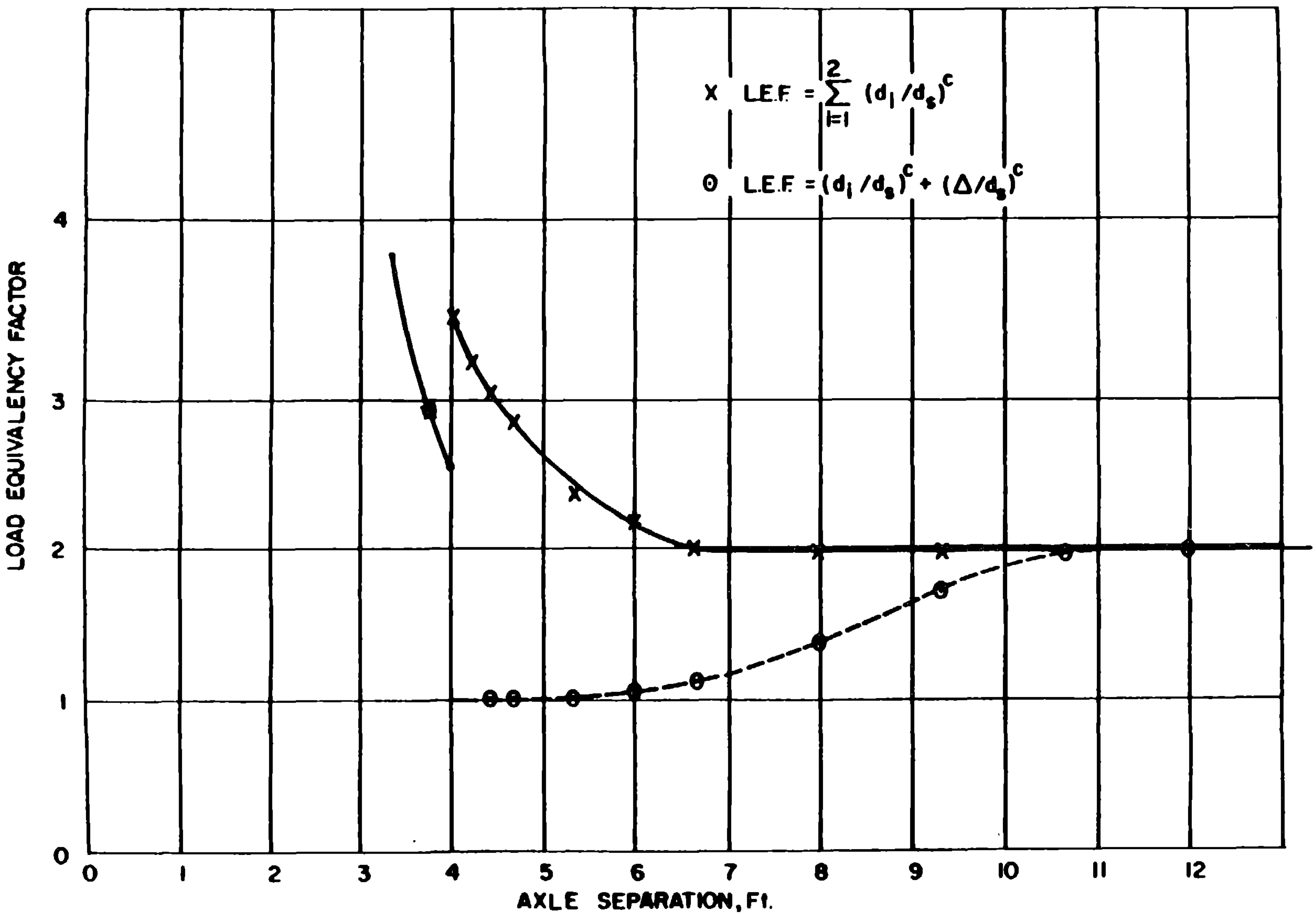


Figure 79. Load equivalency factor for various axle spacing.(33)

APPENDIX B

THEORETICAL DISCUSSION OF STRESS AND LOAD EQUIVALENCY

STRESS EQUIVALENCY

Load equivalency is a special case of stress equivalency whose three basic dimensions (figure 80) are stress level (S), distress level (D), and the number of stress applications (N) that a pavement has received at a fixed stress level when a specific level of distress has been observed. The term stress is used here in a generic sense and refers to the pavement state under a well-defined loading condition that is applied to a well-defined pavement structure (including roadbed) under well-defined ambient conditions. Since stress is not an observable variable, elevation of the pavement's stress state must be done in terms of pavement responses (R) to the loading conditions. Observable responses are strains, deflections, and deformations within the pavement structure. Thus, the stress axis (S) in figure 80 must be replaced by a response axis (R) whenever the system represents observable data.

For any specific type of distress, the symbol D* may be used to denote the level of D at which corrective action (e.g., rehabilitation) is needed to restore the pavement to an acceptable service level. For some analytical developments it is useful to transform D to a unitless "damage ratio," D/D*, and thus transform the (0, D*) range to a (0, 1) range.

Any functional relationship among S, N, and D can be represented by a three dimensional surface in figure 80. When D = D*, the corresponding S-N* plane contains a two-dimensional trace of the surface that may be called a S-N* curve for the conditions represented.

If some particular stress level (S_o) is defined to be a standard stress level, then N_o* is the number of applications at level S_o that have been received by the pavement when D* is observed. At any other stress level (S_x), the number of applications to D* is N_x*, and is defined to be equivalent to N_o* since the pavement is at the same distress level for either set of applications. The ratio of N_o* to N_x* is defined to be the stress equivalence factor (SEF_x) for converting N_x applications at level S_x to an equivalent number (EN_o) of standard applications at level S_o. Thus,

$$EN_o = (SEF_x)\ N_x = (N_o^*/N_x^*)\ N_x \tag{78}$$

where (by definition) N_x applications at level S_x are equivalent to EN_o standard applications at level S_o.

If a series of applications (N_1, N_2, ...) is applied at corresponding stress levels (S_1, S_2, ...), and if each N_x is converted to its equivalent number of standard applications by equation 78, then the total number of equivalent applications for x = 1, 2, ..., is

$$EN_o = \Sigma_x\ (SEF_x)\ N_x = \Sigma_x\ (N_o^*/N_x^*)\ Nx = N_o\ \Sigma_x\ (N_x/N_x^*) \tag{79}$$

Terms within the right-hand summation of equation 79 are generally called cycle ratios. It can be seen that when this summation is unity (Miner's

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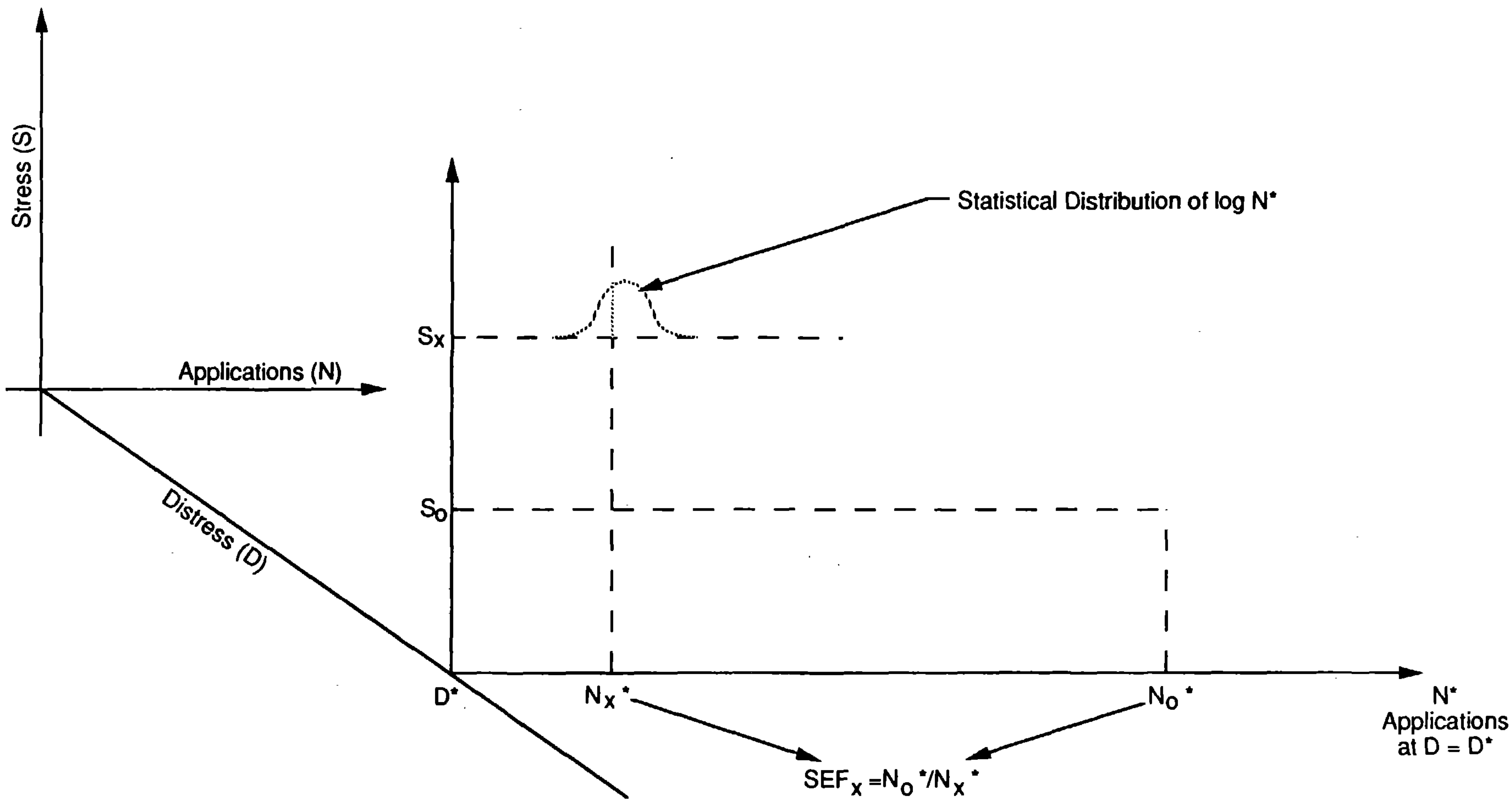

Figure 80. Stress-distress-applications relationships.

rule) then $EN_o = N_o*$ and it is expected that distress level D* will have been reached. Thus Miner's rule can be viewed as a consequence of stress equivalency definitions, and there is no mathematical distinction between Miner's rule and stress equivalency concepts.

A general goal for research on pavement performance and pavement-vehicle interactions is to derive functional relationships among the variables implied by figure 80, including the use of equation (79) for combining mixed-stress applications. Examples include predictive functions for particular forms of distress (D) and for pavement life (EN_o*). For fixed levels of stress (S) and distress (D*), the statistical uncertainty in N (or EN_o) is relatively large and cannot be ignored in the derivation of prediction functions or load equivalence factors. Many studies of fatigue damage and other forms of pavement distress have shown that the distribution of N* (at a fixed stress level) among "homogeneous" specimens (or pavement section S) covers at least one order of magnitude (i.e., one base 10 log cycle).

A common approach to statistical uncertainty in N* is to assume that the frequency distribution of N* is log-normal and therefore has a normal distribution relative to log N*. Thus only one parameter, the standard deviation of log N*, is needed to characterize the statistical uncertainty of S-D-N relationships.

Alongside statistical uncertainty is uncertainty about the mathematical form of any S-D-N relationship. There are generally several competing mathematical models for fitting any particular set of S versus N* data, and different models will inevitably lead to different stress equivalency factors, even when all are derived from the same base. In short, the deterministic part of SEF's is affected by the relative validity of the mathematical model that is used; the unpredictable part of SEF's is affected by the high degree of statistical variation in the observed number of applications to "failure", i.e., N*.

In combination, both types of uncertainty can affect SEF's to a much greater degree than (say) second order changes in load suspensions. Much research is needed to determine the sensitivity of SEF's to the relative effects of a large variety of structural and vehicular stress determinants.

It is useful to transform figure 80 into figure 81 where the S and N axes are now replaced by log S and log N axes. Only the D = D* plane is shown in figure 81, and the S versus N* curve of figure 80 is now a log S versus log N* curve. In these coordinates the statistical distribution of log N* at each stress level may be assumed to be normal, perhaps with nearly equal standard deviations at all stress levels.

For the AASHO Road Test data, the standard deviations (s) for the log N* are about 0.30 and 0.15 for flexible and rigid pavements, respectively. Thus the s scatter of log N* values cover more than one log cycle for flexible pavements and more than half of one log cycle for rigid pavements.

Since, as shown in figure 81, the logarithm of a stress equivalency factor is given by

$$\log SEF_x = \log N*_o - \log N_x* \qquad (80)$$

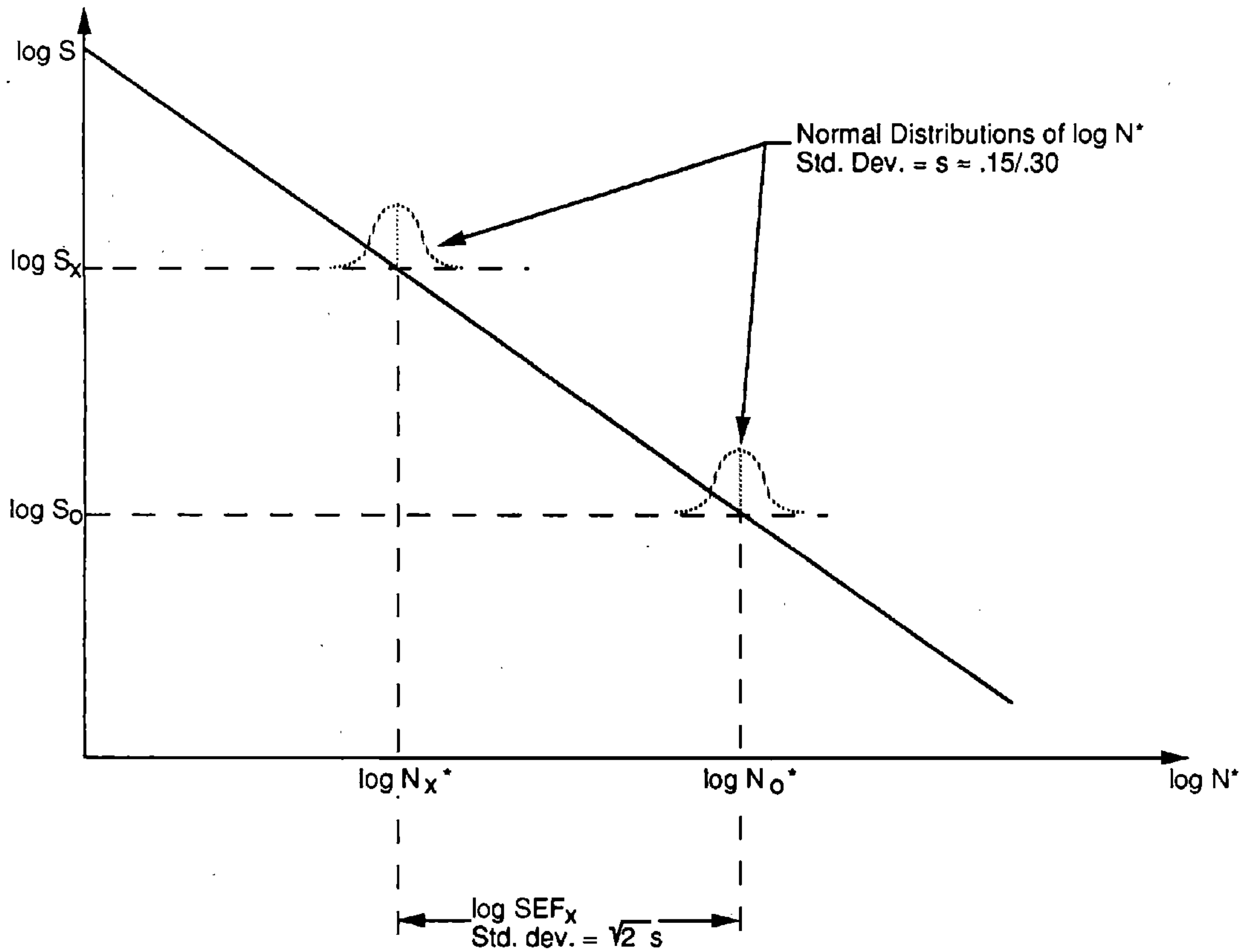

Figure 81. Log S vs. Log N at D = D*.

the statistical variance of log SEF_x is the sum of the variances of log N_o* and log N_x*. If both variances are equal, the standard deviation of log SEF_x is the square root of 2 times the common standard deviation (s) of log N_o* and log N_x*. Thus for the AASHO Road Test observations, log SEF_x has a standard deviation of around 0.42 for flexible pavements and around .21 for rigid pavements. If any SEF_x is based on the mean of n observed values for N_o* and N_x*, the standard deviation for log SEF_x is reduced by a factor of $1/\sqrt{n}$, and the 95 percent confidence band for the "true" log SEF_x has half-width of about $2\sqrt{2}\, s/\sqrt{n}$.

In the AASHO Road Test data it is possible to find several sets of n = 6 test sections that have approximately the same structures at two different axle loadings. Thus log SEF_x's based on Road Test observations (rather than calculated from prediction functions,) have confidence intervals whose half-width is around 0.35 for flexible pavements and around 0.17 for rigid pavements.

If, for example, the observed mean SEF_x is 10.0 (log = 1) at one stress level, and 0.10 (log = -1) at another stress level, the corresponding confidence limits for log SEF_x are 1.0 ± .17 for rigid pavements. The antilogs of these limits for log SEF are 1.0 ± .35 and -1.0 ± .35 for flexible pavements or 1.0 ± .17 and -1.0 ± .17 for rigid pavements. The antilogs of these limits give ranges of 4.5 to 22.4 (flexible) or 6.8 to 14.8 (rigid) when SEF_x = 10.0, and ranges of .04 to 0.22 (flexible) or 0.07 to 0.15 when SEF_x = 0.10.

The practical impact of the foregoing numerical estimates is that only one or perhaps two significant digits can be ascribed to any SEF that is based on actual observations from the AASHO Road Test. It is true that SEF derived from S versus N* relationships will have somewhat more accuracy, but only if the "correct" mathematical model has been fit to the observed data. As previously stated, changes in models can lead to SEF variations at least as large as those associated with the statistical uncertainty of any SEF.

A number of models that have been used for S-N*-D functions lead to relationships in which log N is a decreasing linear function of log S, or alternatively, in which N* is a power function of S. For all such models, $N^* = 10^A S^{-B}$, and

$$\log N_o^* = A - B \log S_o \pm cs \tag{81}$$

at the standard stress level, and

$$\log N_x^* = A - B \log S_x \pm cs \tag{82}$$

at any other stress level.

The last term in equation 81 or 82 is the product of an assumed (common) standard deviation (s) in log N* and a confidence interval factor, c.

From the definition of SEF_x, and from statistical considerations, equations 81 and 82 lead to:

$$\log (SEF_x) = (\log N_o^* - \log N_x^*) = B \log (S_x/S_o) \pm \sqrt{2}cs \tag{83}$$

or

$$SEF_x = (S_x/S_o)^B \ 10^{\pm\sqrt{2cs}} \tag{84}$$

Equation 84 shows that the deterministic part of stress equivalency factors is a power function of the stress ratio, S_x/S_o, whenever the log S versus log N* curve of figure 81 is linear. A somewhat looser conclusion can be stated as follows: stress equivalency factors (N_o*/N_x*) are (approximate) power functions of stress ratios (S_x/S_o) for any range of stress levels over which log N* decreases (more or less) linearly with increasing log S. Thus the so-called "power law" for equivalency factors is valid to the degree that the linearity condition is met.

Another practical consequence of equation 84 is that all multiplicative components of S_o and S_x cancel out and therefore do not affect stress equivalence factors. Such components might include a number of structural, environmental, and loading characteristics, depending upon the mathematical model that is used for the stress function. Another type of cancellation occurs if changes in loading factors bring about the <u>same percentage change</u> in both S and S_x.

It has not been brought out in the present literature which loading factors belong to this category. For example, vehicle suspension factors, tire factors, and dynamic factors that fall in this category have no effect on equivalence factors that are computed via equation 84.

LOAD EQUIVALENCY

To move from stress equivalency to load equivalency it is necessary to assume that all stress determinants (S') that are independent of loading conditions are at specified fixed levels for a series of different loading conditions (L_0, L_1, L_2, ...) where L is a well-defined standard loading condition. In this case, $N_o*(S')$ and $N_x*(S')$ denote the respective number of loading applications to D = D* for loading conditions L_o and L_x, given that non-loading determinants of stress are at S'. The <u>loading equivalence</u> factor for converting applications under L_x to equivalent L_o applications is thus defined by

$$LEF_x\ (S') = N_o*(S')\ /\ N_x*(S') \tag{85}$$

and may change with the structural and/or environmental conditions that are denoted by S'.

The full definition of L_o and L_x must cover all loading factors that affect stress, including static wheel load (WL), tire parameters, vehicle speed and placement, and vehicle dynamics. The symbol L' will be used to represent the set of all loading factor that do not include WL.

It appears reasonable to assume that stress level is a power function of static wheel load when all remaining stress determinants (including other loading factors) are at specified fixed levels. It is thus assumed that

$$S_x = K(S',L')\ (WL_x)^{P(S',L')} \tag{86}$$

or

$$\log S_x = \log K(S',L') + [P(S',L')] \log (WL_x) \tag{87}$$

where all non-loading factors (S') and all loading factors other than WL (i.e., L') are contained in the K and P functions of S' and L'. It is perhaps noteworthy that P can be a constant, or even unity, for stress indicators such as deflection or strain.

Since equation 87 shows that $\log S_x$ is a linear transformation of $\log (WL_x)$ and vice versa, the log S versus log N* curve of figure 81 can be transformed explicitly to a log (WL) versus log N* curve as shown in figure 82. It must be recognized that log N* may now depend not only on WL but also on the levels at which all other stress determinants (i.e., S' and L') are specified. Wheel load equivalence factors can be defined by

$$WLEF_x = N_o^*(S',L')/N_x^*(S',L') \tag{88}$$

or

$$\log WLEF_x = [\log N_o^* (S',L') - [\log N_x^*(S',L')] \tag{89}$$

For the case where $\log N_x^*$ is a decreasing linear function of $\log S_x$, equations 83, 87, and 89 give

$$\begin{aligned}[\log N_o^* (S',L') - \log N_x(S',L')] &= [BP(S',L')] \log (WL_x/WL_o) \pm \sqrt{2cs} \\ &= C(S',L') \log (WL_x/WL_o) \pm \sqrt{2cs}\end{aligned} \tag{90}$$

or

$$WLEF_x = (WL_x/WL_o)^{C(S',L')} 10^{\pm \sqrt{2cs}} \tag{91}$$

Equation 91 is the so-called wheel load (or axle load) "power law" for load equivalency but research has not yet revealed just how the exponent C depends on one or another of the non-wheel load stress determinants. Also, it is not clear that the denominator of equation 88 must contain the L' term. The standard loading condition can justifiably be defined at fixed values of all loading factors (L') such as tire pressure and speed. To estimate the effects of these parameters on relative pavement damage on this study, it was decided to vary the loading factors, L', in the numerator but not the denominator of equation 88 to produce the following:

$$WLEF_x = \frac{N_o^* (S',L')}{N_x^*(S')} \quad \text{(at fixed levels of L')} \tag{92}$$

GENERALIZED LOAD EQUIVALENCE FACTORS

The simplest generalization for equation 91 is to assume that C varies over a relatively narrow range (say 2 to 6) for structures, environments, and distress variables of main interest. If this is the case, then the median value of $C = 4$ would make equation 91 a "fourth power law" and might serve many practical purposes where WLEF precision is not required.

Again for the linear case, certain non-wheel load factors may be absent from the exponent C in equation 91 and thus have no effect on the load

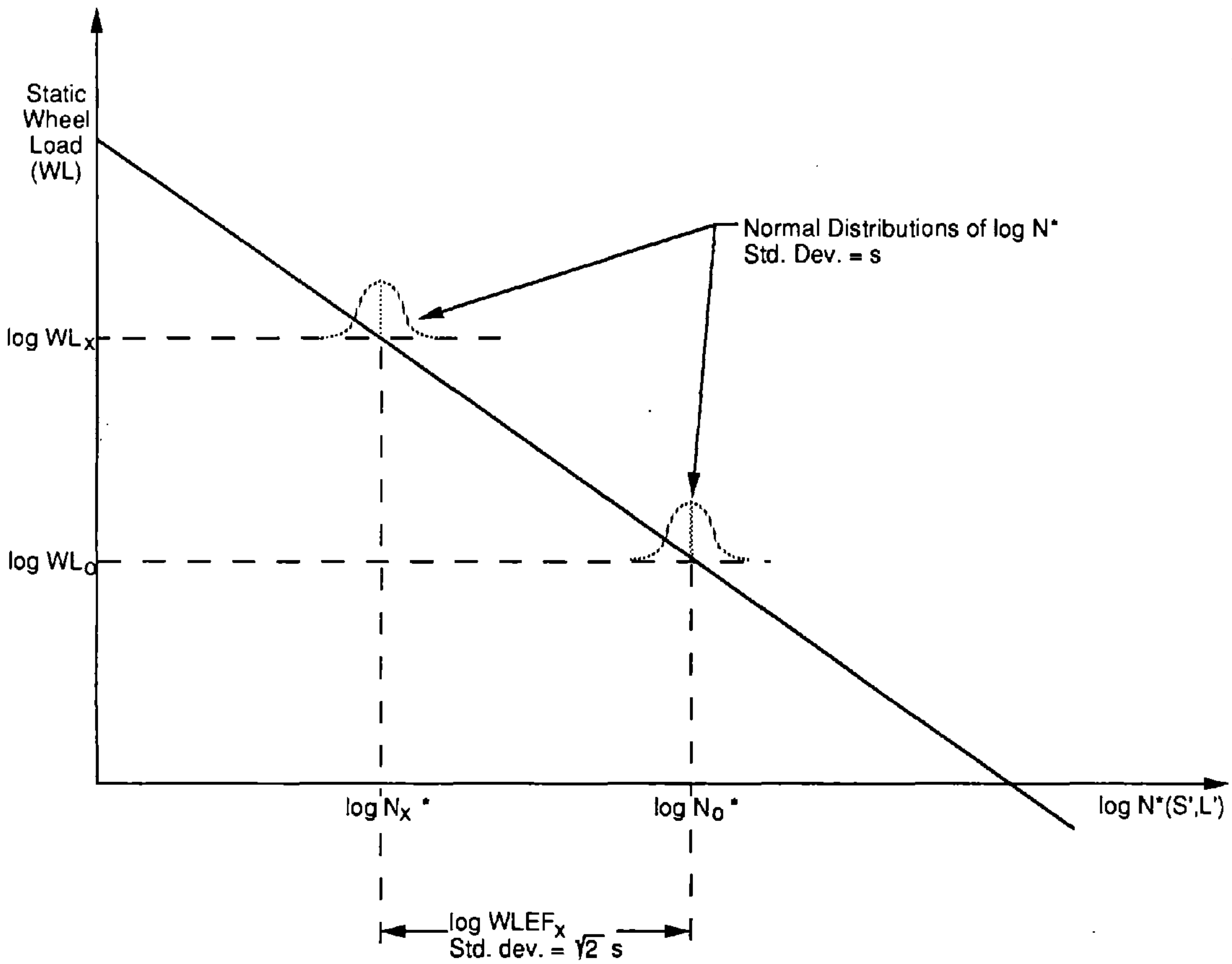

Figure 82. Log WL vs. Log N at D = D*.

equivalence factors. Put another way, all stress determinants must be taken into account for the derivation of S-N-D relationships, but many determinants may disappear or be negligible in the corresponding SEF and WLEF functions.

Graphs for equation 91 will be straight lines in the log WLEF versus log WL plane as shown in figure 83. The central line has slope of $C = 4$. Dashed lines on either side of the central line correspond to 95 percent of the scatter that would be expected among individual observations of WLEF at any particular wheel load level when $c = 2$ and $s = 0.2$ in equation 90. Results from analytical studies of S-N-D relationships and associated equivalence functions strive to account properly for the effects of all contributing factors, but a number of these effects may be ignored or generalized when the results are used to develop pavement design algorithms. One reason for this relaxation is that only relatively crude estimates for loading factors can be known at the pavement design stage.

It appears that a still higher level of generalization for equivalence factors might be useful in certain applications to pavement/vehicle economics. This approach seems especially warranted by the fact that over any substantial period of time, any given transport vehicle may have travelled over several pavement types that exhibit several types and degrees of distress, and during wide variations in ambient conditions, perhaps from hour to hour. Thus precise research results and accurate measurements might lead to (say) at least one thousand different load equivalence factors for any given vehicle. For economic studies it may therefore be more appropriate to determine mean values or envelopes of equivalence factors that are to be ascribed to particular classes of vehicles.

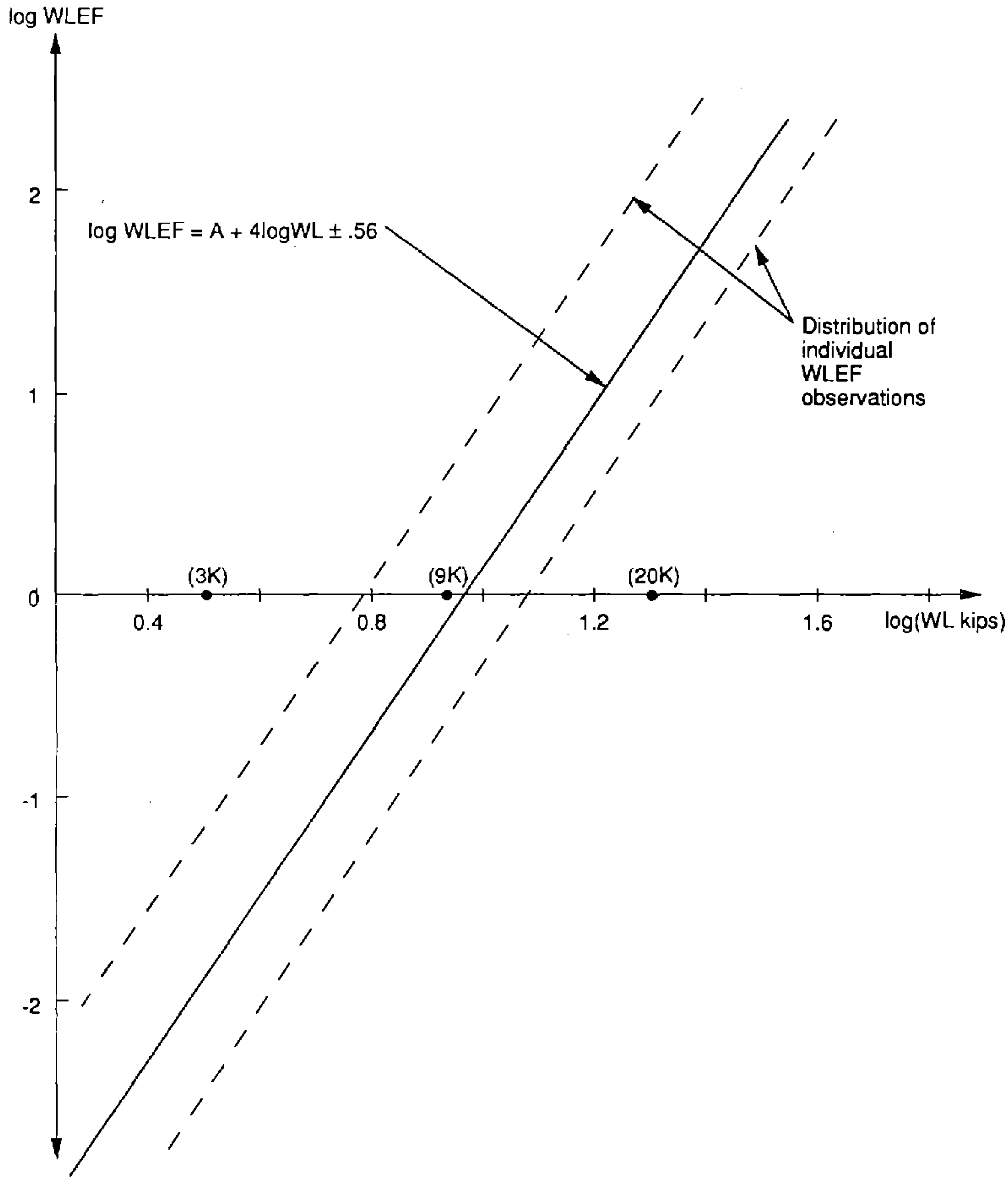

Figure 83. Log WLEF vs. Log WL at D = D*.

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