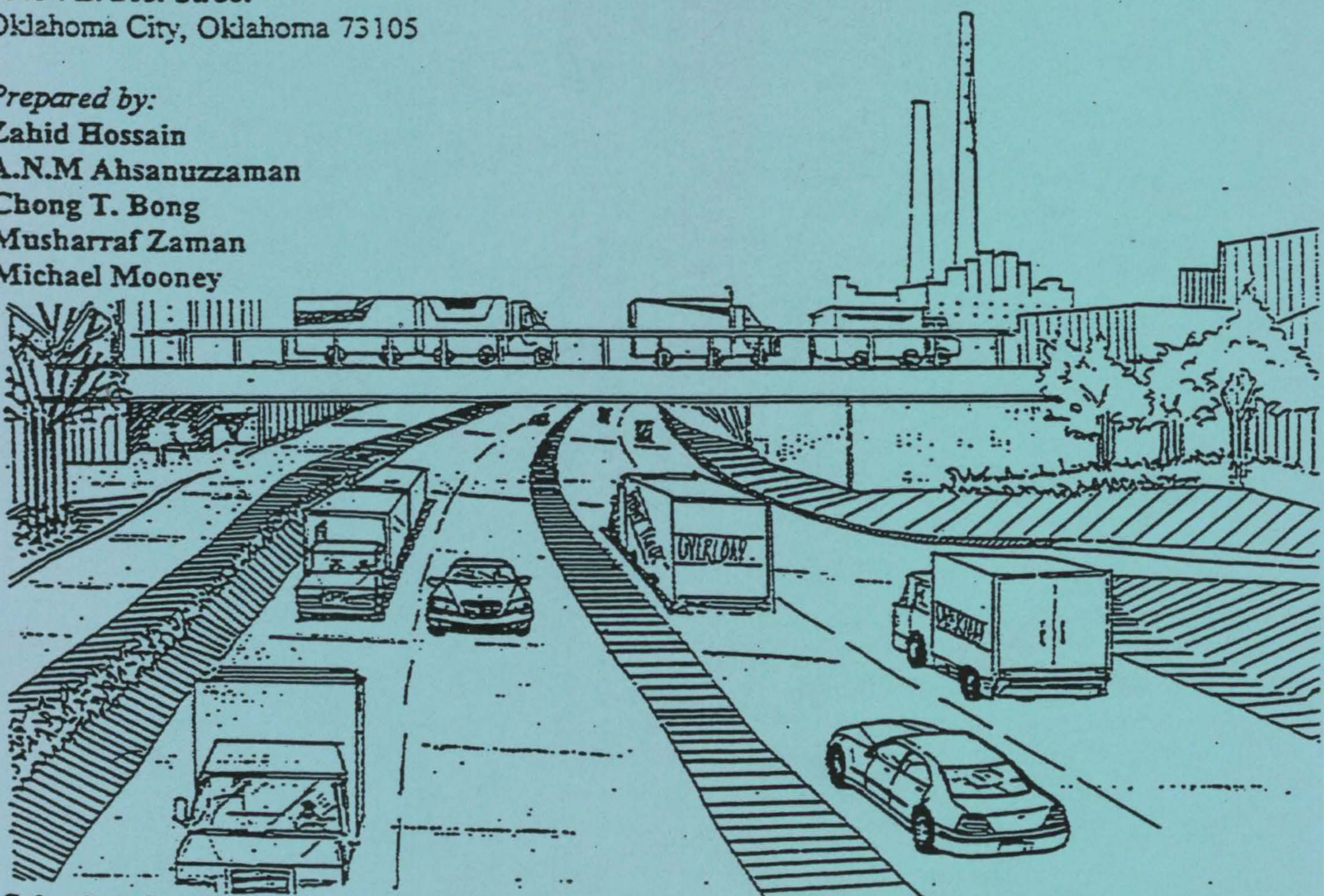
IMPACT OF LOADED TRUCKS ON HIGHWAY INFRASTRUCTURE DETERIORATION

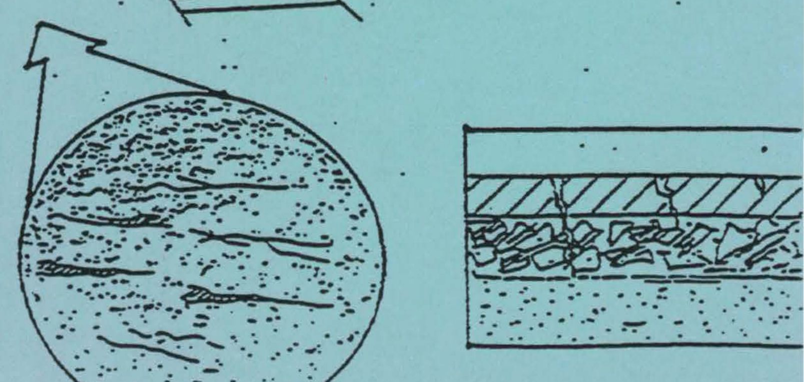
FINAL REPORT
(Item 2112; ORA 125-5111)

Submitted to:
Lawrence J. Senkowski
Assistant Division Engineer
Research and Development
Oklahoma Department of Transportation
200N. E. 21st Street
Oklahoma City, Oklahoma 73105

Prepared by:
Zahid Hossain
A.N.M Ahsanuzzaman
Chong T. Bong
Musharraf Zaman
Michael Mooney



School of Civil Engineering
and Enironmental Science
The University of Oklahoma
Norman, Oklahoma 73019.



From:
The Office of Research Administration
The University of Oklahoma
Norman, Oklahoma 73019

TECHNICAL REPORT STANDARD TITLE PAGE

REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO. HE336 .I46 1997
TITLE AND SUBTITLE "Impact of Loaded Trucks on Highway Infrastructure Deterioration"		5. REPORT DATE January, 1997
		6. PERFORMING ORGANIZATION CODE
AUTHOR(S) Zahid Hossain, A.N.M Ahsanuzzaman, Chong T. Bong, Musharraf Zaman, Michael Mooney		8. PERFORMING ORGANIZATION REPORT ORA 125-5111
PERFORMING ORGANIZATION AND ADDRESS The University of Oklahoma, Norman, Oklahoma 73019		10. WORK UNIT NO.
		11. CONTRACT OR GRANT NO. Item No. 2112
SPONSORING AGENCY NAME AND ADDRESS Oklahoma Department of Transportation, Research and Development, 200 NE 21 st Street, Oklahoma City, Oklahoma 73105		13. TYPE OF REPORT AND PERIOD COVERED Report (Sept. 1996 - Jan. 1997)
		14. SPONSORING AGENCY CODE
15. SUPPLEMENTARY NOTES Performed in cooperation with the US Department of Transportation and the Federal Highway Administration.		

16. ABSTRACT

Impact of loaded trucks on our transportation infrastructure system is becoming a growing concern for many state Departments of Transportation (DOTs) in the U.S. An increasing volume of loaded trucks due to the implementation of the NAFTA likely to make the situation worse for the NAFTA corridor states including Oklahoma, because the axle loads as well as gross vehicular weight limits for the Mexican and the Canadian trucks are much higher than the corresponding U.S. limits. This report attempts to document the effect of truck axle load, gross vehicular weight, and traffic volume on major damage to pavements and bridges. The information was mostly assembled through a comprehensive literature search and contacts with several state agencies. Due to limited resources, scope, and time, no laboratory and field study and material testing were performed.

The truck user fees and taxes in Oklahoma are not equitable with the damage these heavy vehicles cause to the transportation infrastructure. Over 92% of the total equivalent single axle loads (ESALs) on rural Interstate highways are contributed by truck traffic. The corresponding average ESALs for rural and urban highways are over 80%. Fatigue damage is one of the most common distresses in both flexible and rigid pavements, although it is more predominant in rigid pavements. Fatigue damage in pavement is highly sensitive to the axle load (proportional to the fourth power). Due to a 10% increase in axle load, from the current limit of 20 kips to 22 kips, the fatigue damage is increased by 46%, thus significantly reducing the remaining life of pavements. High axle loads in asphalt pavements drastically increase the rutting potential. The axle load magnitudes and frequency of truck traffic are largely responsible for faulting and pumping-induced deterioration in concrete pavements. Heavy vehicles (over 7,700 lb) are believed to be responsible for about 99% of the total traffic-related damages in pavements. An 80 kips truck has the same damaging potential as 9,600 automobiles. Also, the serviceability of pavements is shortened significantly by the action of heavy trucks.

Overstress in bridge members due to the passage of heavy Canadian trucks was analyzed in this study. About 70 percent of the Oklahoma interstate bridges were found safe with respect to overstressing. Almost all concrete culvert and concrete girder bridges are not likely to be susceptible to overstress, while the majority of steel bridges may undergo significant overstressing due to the passage of the Canadian trucks. Increase of truck load and volume can also result significant fatigue damage to steel bridges and thus reduce the life.

17. KEY WORDS Deterioration, Fatigue, Overstress, Axle load, ESAL, Serviceability, Gross vehicular weight, Heavy truck, Pavement, Bridge, Span Length.		18. DISTRIBUTION STATEMENT No restrictions. This publication is available from the Office of Research, Oklahoma DOT.	
19. SECURITY CLASSIF. (OF THIS REPORT) Unclassified	20. SECURITY CLASSIF. (OF THIS PAGE) Unclassified	21. NO. OF PAGES [illegible]	22. PRICE

IMPACT OF LOADED TRUCK ON HIGHWAY INFRASTRUCTURE DETERIORATION

FINAL REPORT
(Item 2112; ORA 125-5111)
"Technology Transfer Support Program"

Submitted to:
Lawrence J. Senkowski, P. E.
Assistant Divisional Engineer
Research, Development and Technology Transfer
Oklahoma Department of Transportation
200 N.E. 21st Street
Oklahoma City, Oklahoma 73105-3204

Prepared by:
Zahid Hossain
A.N.M. Ashanuzzaman
Chong Tat Bong
Musharraf Zaman
Michael Mooney

School of Civil Engineering and Environmental Science
The University of Oklahoma
Norman, Oklahoma 73019

From:
The Office of Research Administration
The University of Oklahoma
Norman, Oklahoma 73019

January 1997

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the information gathered herein. The contents do not necessarily reflect the official views of the Oklahoma Department of Transportation. This report does not constitute a standard, specification, or regulation.

ACKNOWLEDGMENTS

The authors would like to express their sincere appreciation to Dr. Joakim G. Laguros, David Ross Boyd Professor Emeritus, the University of Oklahoma (OU) School of Civil Engineering and Environmental Science for his technical assistance in the project and review of the project report. The authors gratefully acknowledge a number of people from the Oklahoma Department of Transportation (ODOT) who assisted this project in various capacities: Mr. Lawrence Senkowski, Mr. David Girdner, Mr. John S. Northup, Dr. Alan Soltani, Mr. Tim Borg, Mr. Charlie Younger. Mr. Senkowski provided the overall direction and helped to define the project goals and objectives. Mr. Girdner assisted in conducting the literature search. Dr. Soltani provided some valuable information about the current pavement design practice in Oklahoma. Mr. Borg and Mr. Younger assembled some of the statistics presented in Chapter II. Mr. Northup provided some relevant data on Oklahoma bridges.

The authors are thankful to Ms. Julie Zeleznik of the OU English Department for her help in proof-reading the report.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Figure	Title	Page

ABSTRACT

Impact of loaded trucks on our transportation infrastructure system is becoming a growing concern for many state Departments of Transportation (DOTs) in the U.S. An increasing volume of loaded trucks due to the implementation of the NAFTA is likely to make the situation worse for the NAFTA corridor states including Oklahoma, because the axle loads as well as gross vehicular weight limits for the Mexican and the Canadian trucks are much higher than the corresponding U.S. limits. To document the impact of loaded trucks on transportation infrastructure deterioration, the Oklahoma Department of Transportation (ODOT) funded a study under item 2112, *Technology Transfer Support Program*. The objectives of this study are: (i) to assess the role of loaded trucks on the deterioration of transportation infrastructure, especially state roads and bridges; (ii) to review the consideration of projected truck traffic in the design practice for pavements and bridges; and (iii) to assess the role of loaded trucks in the design of highway bridges in Oklahoma. This report attempts to document the effect of truck axle load, gross vehicular weight, and traffic volume on major damage to pavements and bridges. The information was mostly assembled through a comprehensive literature search and contacts with several state agencies. Due to limited resources, scope, and time, no laboratory and field study and material testing were performed.

The truck user fees and taxes in Oklahoma are not equitable with the damage these heavy vehicles cause to the transportation infrastructure. Illegally overloaded trucks often escape fines because of failure of the administrative procedures. Over 92% of the total

equivalent single axial loads (ESALs) on rural Interstate highways are contributed by truck traffic. The corresponding average ESALs for rural and urban highways are over 80%. Fatigue damage is one of the most common distresses in both flexible and rigid pavements, although it is more predominant in rigid pavements. Fatigue damage in pavement is highly sensitive to the axle load (proportional to the fourth power). Due to a 10% increase in axle load, from the current limit of 20 kips to 22 kips, the fatigue damage is increased by 46%, thus significantly reducing the remaining life of pavements. High axle loads in asphalt pavements drastically increase the rutting potential. The axle load magnitudes and frequency of truck traffic are largely responsible for faulting and pumping-induced deterioration in concrete pavements. Heavy vehicles (over 7,700 lb) are believed to be responsible for about 99% of the total traffic-related damages in pavements. An 80 kips truck has the same damaging potential as 9,600 automobiles. Also, the serviceability of pavements is shortened significantly by the action of heavy trucks.

Heavy truck damage to bridges may result due to fatigue and overstressing. It is evident from the extensive literature survey that heavy loaded trucks may cause significant fatigue cracking in concrete deck and steel details of composite bridges. To this end, reduction of bridge fatigue life may result from increased truck load and truck volume. Overstress in bridge members due to the passage of heavy trucks was analyzed in this study. Bridges with span length greater than 50 ft are found susceptible to overstressing. About 70% of the Oklahoma interstate bridges have span length above this level and thus safe with respect to overstressing. Almost all concrete culvert and concrete girder bridges are not likely to be susceptible to overstress, while the majority of steel (composite) bridges may undergo significant overstressing due to the passage of the heavy trucks.

Because the design, construction, maintenance, and management practices can very significantly among various States in the U.S., specific conclusions about the impact of loaded trucks on transportation infrastructure deterioration cannot be made from general literature survey. A more comprehensive study can be undertaken in which the existing data (e.g., traffic data, sufficiency rating, structural condition, maintenance cost, etc.) be analyzed and new data collected to evaluate the specific impacts of loaded trucks on our transportation infrastructure system.

CHAPTER I

INTRODUCTION AND RESEARCH APPROACH

1.1 INTRODUCTION

Truck traffic on the roads and highways of the United States has grown rapidly over the past three decades. This growth rate is expected to increase further due to various socio-economic and political reasons (Fekpe et al. 1995, pp. 39; Backlund et al. 1990, pp. 114-115) and in turn, the anticipated increase in truck traffic will affect the highway safety and traffic operations in many ways: congestion in highways and consequent increase of accidents, increase in price of commodities, and damage of transportation infrastructure. Trucking has become an important conflicting issue. On one hand, trucking is vital in promoting commerce, trade, and economic activity while it attempts to improve operating efficiency. On the other hand, it is simultaneously faced with regulatory limits and the deterioration effects it causes on the transportation infrastructure (Fekpe et al. 1995, pp. 39).

The performance of existing pavements and bridges is affected greatly by traffic load, age, environment, construction quality, etc. There is an immense need to assess the traffic-associated deterioration of transportation infrastructures. Among all traffic types, heavy trucks deliver the highest vehicular and axle loads to the pavements and bridges. By definition, a truck having gross vehicle weight greater than 55,000 lb is known as heavy truck (Reno and Stowers 1995, pp. 41). According to a recent study, heavy trucks contribute 80% of equivalent single axle loads (ESALs) to pavements for all highways and

92% of the ESALs on rural Interstate (Backlund et al. 1990, pp. 116). All loaded trucks, however, do not cause equal damage. Individual axle load, truck configuration, and gross vehicle weight (GVW) play a significant role.

Truck traffic may lead to excessive fatigue, permanent surface and subsurface deformation, and surface disintegration of pavements. Because of the nonlinear (fourth-order function) effect of axle loads, heavy trucks are far more damaging to pavement surfaces than are light vehicles (TRB 1989, pp. 173-175). Heavy trucks also cause over stress and fatigue-related damage to bridges. Overstress concerns include the possibility of severe damage and possible collapse caused by a single extreme loading event. Realizing the aforementioned damages, in recent years, engineers are continuously searching for improved and cost-effective design, construction, maintenance and management schemes for the nation's transportation infrastructure system that will carry larger and heavier vehicles without experiencing excessive deterioration and maintenance costs.

Information gathered during this study documents the effects of loaded trucks on the deterioration of transportation infrastructure, specifically state roads and bridges. In particular, this report summarizes how engineers design pavements and bridges based on projected truck traffic and elaborates on the impact of loaded trucks. Additionally, pavement types (e.g., asphalt concrete and Portland cement concrete) in relation to their use, life-cycle costs, and required maintenance are discussed in light of the objectives of this study (impact of loaded truck). Finally, an overview of the role that trucks play in the design of interstate bridges of Oklahoma is included.

1.2 OBJECTIVES

The overall objective of this report is to present the deteriorating effects of loaded trucks on the transportation infrastructure. This is a broad subject, which involves many components, such as recommendations and regulations pertinent to truck traffic, design and construction procedures for pavements and highway structures, and infrastructure maintenance and rehabilitation. With frequent use by extra heavy trucks, pavements and bridges in Oklahoma are likely to experience increased deterioration. To document the impact of loaded trucks on transportation infrastructure deterioration, the Oklahoma Department of Transportation (ODOT) contracted with the School of Civil Engineering and Environmental Science (CEES) at the University of Oklahoma (OU), under item 2112 "Technology Transfer Support Program." Based on that agreement, this study was pursued to achieve the following objectives:

(i) to assess the role of loaded trucks on the deterioration of transportation infrastructure, specifically state roads and bridges;

(ii) to review the consideration of projected truck traffic in the current design practice for pavements and bridges; and

(iii) to assess the role of loaded trucks in the design of highway bridges in Oklahoma.

1.3 METHODOLOGY

In this study, information was assembled mostly through literature search. No attempt was made to collect or interpret field data. Therefore, the discussion presented herein is based on a careful analysis of technical papers and reports published by the Transportation Research Board (TRB), the National Cooperative Highway Research

Program (NCHRP), the state Departments of Transportation (DOTs), as well as international journals and periodicals.

1.4 SCOPE AND LIMITATIONS

Based on thorough review of existing literature this report attempts to document the effect of truck axle load, GVW, and traffic volume on inducing major damage to pavements and bridges. Due to limited resources, scope, and time, or field study, material testing nor analysis were performed. To determine more specifically the effect of overloaded trucks on the transportation infrastructure, a detailed study should be undertaken incorporating the important factors. For example, an evaluation of fatigue-related damage to the bridges requires a comprehensive study, including assessment of expected traffic composition and bridge details.

1.5 ORGANIZATION OF THE REPORT

This report encompasses six chapters. The regulatory summary on truck size and weight limits and design aspects of pavements considering truck loading is provided in Chapter II. Deterioration of Asphalt Concrete and Portland Cement Concrete pavement due to loaded truck is explained in Chapters III and IV, respectively. Bridge loading and deterioration influenced by truck traffic are analyzed in Chapter V. Finally, the conclusions of the study and some recommendations for further research are outlined in Chapter VI.

There are two appendices in this report. Load equivalency factors for different axle loading are presented in Appendix A. Various truck types, used to find out their relative damage potential relative to pavements are discussed in Appendix B.

CHAPTER II

REGULATORY OVERVIEW AND DESIGN ASPECTS OF ROADWAY

2.1 FUNDING OF HIGHWAYS: Who Pays What?

Federal and state governments fund highway expenditures through user and non-user taxes in the form of gasoline and diesel taxes, as well as state and federal registration fees. In other words, those who benefit from the highways pay for their construction, maintenance and rehabilitation through specific taxes and fees. Distribution of national highway funds by source and by level of government is given in Fig. 2.1. Motor fuel taxes contribute over 75% of the revenue obtained by the Federal Highway Trust Fund. The federal heavy-vehicle tax shown in Fig. 2.1(a) is an annual tax on GVW above 55,000 lb, it is levied as $100 plus $22 per 1,000 lb over 55,000 lb. Figure 2.1(b) shows that federal funds contribute a significant amount toward building state funds. Approximately 28% and 26% of state funds come from motor fuel taxes and motor vehicle fees, respectively.

The proportion of sources of state funds varies from state to state. In Oklahoma, approximately 55% (44% motor fuel taxes plus 11% motor vehicle taxes) and 25% of highway funding came from highway user revenue and federal funds, respectively in 1992 (Reno and Stowers 1995, pp. 41). The annual vehicle license fee is based on its GVW as assessed by the Oklahoma Tax Commission (OS 1991, pp. 4760). The registration fees for trucks having GVW from 8,001 lb to 90,000 lb are shown in Table 2.1. User taxes are shared by automobiles, other light vehicles, and truck traffic. More than 60% of federal revenue used for highway expenditures comes from automobiles and other light vehicles (Fig. 2.2). The AASHO road test in Ottawa, Illinois 1958-1960 established that a

80,000 lb truck (considering maximum axle loading) weighing the equivalent of 20 automobiles causes as much damage as 9600 single automobiles (GAO 1991, pp. 24). This aspect of transportation infrastructure deterioration is discussed further in Chapters III and IV of this report.

To distribute the cost of federal highways, the 1982 U.S. Department of Transportation's Cost Allocation Study introduced the term "fair share" (USDOT, 1982*). According to Fig. 2.3, heavy trucks were required to pay 55% less than their "fair share" before the Surface Transportation Assistance Act (STAA)-82 was enforced (USDOT, 1982*); i.e., the heaviest trucks had been paying only 45 cents for each $1.00 of highway costs for which they were responsible. Although the STAA-82 levied additional taxes, heavy trucks have to pay about 29% less than their "fair share" (USDOT, 1983*). The sub committee on Truck Size and Weight of the AASHTO joint committee on Domestic Freight Policy (1995) already recommended that "Truck user fees should be equitable and charge appropriately for the use of transportation infrastructure. User fees should be equitable among vehicles of same class as well as among vehicles of different classes, including passenger vehicles." It also commented that "past methods of calculating user costs have not included environmental, congestion, and other less easily determined costs. Research is needed in these areas to further define costs associated with truck operation."

2.2 REGULATORY OVERVIEW ON TRUCK SIZE AND WEIGHT

Truck size and weight limits vary substantially among states. Each state assigns vehicle dimensions and weight limits considering aspects such as road maintenance, construction, congestion, and safety. The current regulations related to truck size, configurations and weight limits were established after a number of modifications and revisions.

2.2.1 Historical Background and Current Limits

During the First World War, most materials were shipped from inland factories to ports by truck due to the limitations of railroad. Consequently, state officials discovered the motor truck as a major cause of deterioration of state roads and bridges. The New York Highways Commissioner suggested the adoption of a truck weight limit in view of the massive deterioration of highways as well as the potential growth of the trucking industry after the war (Duffey 1918, pp. 4). Accordingly, the motor vehicle manufacturers committed to limit gross vehicle weight (GVW) of trucks to 28,000 lb, or 800 lb of weight per inch of solid tire width (Bennett 1921, pp. 12). In 1932, the American State Highway Officials (AASHO), which latter became the American Association of State Highway and Transportation Officials (AASHTO), recommended a single-axle limit of 16,000 lb and a tandem-axle limit as a function of the distance between the two axles. AASHO revised the 1932 policy in 1946 and recommended a single-axle limit of 18,000 lb, the tandem-axle limit of 32,000 lb, and a GVW limit of 73,280 lb (TRB 1990a, pp. 36). At the same time AASHO also suggested that GVW be based on axle spacing. After the Second World War, federal effort to fund highway construction was vastly expanded. Federal-Aid Highway Legislation was first enacted in 1956 and approved the AASHO 1946 policy to the Interstate highways. It also allowed operation of trucks with higher limits on the Interstate highways that were legal in some states before July 1, 1956; this was known as "grandfather clause" (TRB 1990a, pp. 38). In 1974, the U.S. Congress increased the GVW from 73,280 lb to 80,000 lb. That legislation also adopted a maximum speed limit of 55 mph on Interstate (CBO 1978 pp. 58). The 1974 Highway Act also contained "grandfather clauses" that allowed heavier trucks in excess of 80,000 lb to operate with a special permit. On the other hand, some states in the Mississippi Valley and Montana retained lower axle limits; these states were called the "barrier states." In 1982, the Surface Transportation Assistant Act (STAA-82) further increased the federal limits: a single-axle limit of 20,000 lb, a tandem axle limit of 34,000 lb, GVW of 80,000 lb. The STAA-82 also introduced the federal bridge table (Table 2.2) in relation to GVW and truck configuration. This act formed a uniformity of load limits for Interstate highways in all the states of the U.S.A. and resolved the problems faced by the truckers in the "barrier states" (TRB, 1990a, pp. 44). This authorized the U.S. Secretary of Transportation to designate a National Network of Interstate and other major highways (with 12 ft lane width) on which longer combination vehicles (LCV) could travel without any restriction. These LCV's included the wider (up to 8.5 ft) and longer tractor-semi-trailer (minimum trailer length 48 ft) and twin trailer (minimum trailer length 28 ft) trucks (TRB 1989, pp. 1). The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 continues to allow certain exemptions (grandfather clause) as did previous legislation. However, the ISTEA limits the operation of LCV - double and triple trailer combinations greater than 80,000 lb. GVW - to configuration types that were authorized by the state officials on or before June 1991. Due to the grandfather clause, state and regional regulations concerning truck size and weight continue to vary in large scale from state to state. Table

2.3 summarizes the current vehicle size and weight limits for all the states in the U.S. From this table it is evident that a single unit truck length varies from 40 ft to 60 ft while a semi-trailer length varies from 45 ft to 60 ft or it is not restricted. A twin trailer truck up to 88 ft long is allowed on designated state roads and the GVW ranges from 73,280 lb. to 143,000 lb. The differences among various states create an unfavorable situation for the trucking industry, which is seeking to promote uniform regulations throughout the country.

In Oklahoma, the legislature has set the maximum single axle load at 20,000 lb, the maximum tandem axle load at 34,000 lb and the maximum tridem axle load at 42,000 lb. The GVW is limited to 80,000 lb for Interstate and federally designed highways and 90,000 lb for all other state highways and supplemental roads. LCV are permitted on some routes. However, the legal GVW of any type of vehicle must satisfy the federal bridge formula. The maximum height and width for all vehicles is 13.5 ft and 8.5 ft, respectively, for both the Interstate and state highway systems. Currently, Oklahoma does not impose restrictions on the overall length of LCV, but the length of a trailer is limited to 29 ft in a tractor-twin-trailer. Also, the maximum length of a semi-trailer in a tractor-semi-trailer cannot exceed 59 ft. Furthermore, Oklahoma permits triples of unlimited overall length on its roadways, but restrictions on the length of individual trailers result in a practical maximum length of 105 ft. In Oklahoma, over 900 miles of Interstate highways are open to triple GVW of up to 80,000 lb, and about 900 miles of other primaries are open to triple GVW of up to 90,000 lb (AASHTO 1995, pp. 16).

2.2.2 Overload Permits and Regulations

All states have introduced special rules and regulations that apply to the movement of oversize and/or overweight trucks. These regulations are intended to address the safety requirements and guidelines for spreading the load over extra axles. Particularly, overweight and/or oversize vehicles are supposed to have their routes approved in advance. Certain groups of vehicles are immune to usual legal limits depending upon loading type (divisible or indivisible) or commodity type. It appears that there is no uniformity in this system among various states in the country. Oversize and overweight permits are generally issued for periods of time up to one year. Several states fix costs for these permits by an administrative fee plus a fee based on the excess weight. Some states fix the latter fees by forming a weight-distance or an axle-distance rule, while others fix fees similar to a registration fee that does not reflect any cost responsibility per weight (Terrel and Bell 1987, pp. 38-39). The special single trip permit system for overweight vehicles in Oklahoma is based on the gross vehicle weight: a flat fee of $20.00 plus $5.00 for every 1,000 lb overweight. The oversize vehicles have to pay $20.00 for single trip permit fees in Oklahoma. Again, that fee for vehicles having both overweight and oversize is a flat fee of $40.00 plus $5.00 for every 1,000 lb overweight. The annual permit for overweight special machinery on Interstate is $60.00 It is evident from Table 2.4 that the number of total overweight and oversight permits are 124,681 in the fiscal year 1996 which are about 6% more than that in the fiscal year 1995. The total fee collected from sales of permits in 1996 was $6,067,605 which is somehow less than that in 1995.

Vehicles that operate in excess of the prescribed limits without a special permit are cited by the highway patrol. Actual fines and possible jail terms vary according to the number of offenses. The fine structure for overweight violations in Oklahoma is as follows: \$50-\$200 or a jail term of not more than 30 days or both for the first offense; a second offense within 1 year results in a \$100-\$200 fine with a similar jail term, while a third offense within 1 year of the second offense leads to \$250-\$500 fine and/or a jail term of not more than 6 months. Fines for illegal overloading or oversizing are not related to the actual cost of pavement damage. Moreover, operators of illegally loaded vehicles often escape fines due to a failure in the judicial or enforcement branch of the government. The combination of low fines and a low probability of being captured provides a strong incentive for illegal overloading (Terrell and Bell 1987). About 20% of the vehicles operating on federal-aid highways have axle or gross loads in excess of statuary limits (Terrell and Bell 1987, pp. 1). Furthermore, it is estimated that the cost of overloaded vehicles to the federal-aid highway system is of the order of \$1 billion annually. In a State of Washington case study Barron et al. (1994*), it was found that the actual capture rate of overloaded trucks is estimated to be 10%. An NCHRP study by Terrel and Bell (1987) recommended that the states introduce appropriate legislation for both permit fees and fine schedule, and enforce the associated laws strictly, considering the damaging responsibility of overweight vehicles. They also suggested that each state evaluate appropriate methodologies to enforce the law against people who control the loading and operation of overweight and/or oversize vehicles.

2.2.3 Canadian and Mexican Truck Size and Weight Limits

Owing to the North Atlantic Free Trade Agreement (NAFTA), both Canadian and Mexican trucks are expected to travel on the U.S. highway system with an increased frequency. Both the axle load limits and the gross weight limits of Mexico and Canada are quite different from those of the United States. The single axle load limits of Canada and Mexico are 20,056 lb and 22,040 lb, respectively. The allowable tandem-axle loads in Canada and Mexico are 7,468 lb and 9,672 lb, respectively, over the U.S. limits. From Table 2.5, the allowable GVW of Mexican 7-axle Tractor-Twin-trailers is about 1.75 times that of the U.S. limit. The GVW limits of Canadian Tractor-Twin-Trailers (7 axles) are also about 1.5 times higher than the U.S. limits. If these trucks are allowed to travel on the existing roads and bridges in the U.S., they are likely to accelerate the deterioration of our transportation infrastructure system. To avoid such anticipated problems, the size and weight of the NAFTA trucks should be consistent with the AASHTO Domestic Freight Policy. However, political decisions may prevail engineering judgments.

2.3 OKLAHOMA INTERSTATE HIGHWAY PAVEMENTS

The total Interstate highway mileage in Oklahoma is about 950 miles. More than 60% of total Interstate pavements (576 miles) in Oklahoma are Asphalt Concrete (AC), the rest are Portland Cement Concrete (PCC) pavement (ODOT 1994 pp. 78). The sufficiency rating of these pavements is performed by assigning relative point values to several elements of design and condition. Sufficiency ratings from 80 to 100 are considered adequate, 70 to 79 considered tolerable, and 69 or less considered critical. The current condition of Oklahoma Interstate pavements and average daily traffic (ADT) are summarized in Table 2. 6 . Approximately 83% of the Interstate highway pavements are currently considered adequate, over 1% are tolerable, and about 16% are considered critical.

2.4 CURRENT DESIGN CONSIDERATION FOR TRUCK LOADING

For the structural design of each type of pavement, traffic parameters (especially truck traffic) are the most important. The other important factors include the roadbed material characteristics, the pavement layer characteristics, the serviceability characteristics, the climate conditions, and the reliability factors for all of these parameters. Highway deterioration will be minimized if the pavement is designed with due consideration to the type and amount of traffic that will use the pavement and accounts for the prevalent environmental conditions.

Like most state DOTs, roadway pavements in Oklahoma following the AASHTO (1986) guidelines are based on a 20-year design period (Forsyth 1993, pp. 5-15). The AASHTO guidelines (1993) currently recommend a 20 to 50-year design life for Interstate highways and high volume urban roads. An appropriate traffic growth factor (Table 2.7) dependent upon the analysis period and the traffic growth rate is considered in estimating the future traffic. The compound growth rate is usually taken as 2.5% by designers for all types of traffic. The growth rate for trucks is higher than that of cars or buses. For example, during the 1970 to 1983 period, the total percent contribution (volume) of passenger cars and buses decreased from 77 to 63, while the percent of truck traffic having

5 axles or more increased from 9 to 17 percent on the rural Interstate highways (AASHTO, 1993, pp. D-1). Furthermore, all light vehicles are relatively trivial load factors in comparison to heavy truck traffic.

The projected traffic load is determined by converting the axle-load throughout the analysis period of pavements into 18-kip equivalent single-axle loads (ESALs). According to a recent study, only heavy trucks (most are conventional 3-S2 and larger trucks) contribute up to 92% of the ESALs assigned to pavements on rural Interstate highways and 80% of ESALs for all highways (Backlund and Gurver 1990, pp. 116). Also, the Five-Axle Tractor Semi-trailer (3-S2) is the most common type of truck in the United States and Canada, accounting for approximately 70% of all trucks (Fekpe et al. 1995, pp. 40; Backlund and Gurver 1990, pp. 115). The ESAL factors vary sharply with axle load, following roughly a fourth power relationship; the ESAL of a single axle load is given by $(x/18)^4$ where x represents the axle weight in kips. For example, the ESAL factor for a 22,000-lb single axle is 2.23. Figures 2.4 and 2.5 show the high increasing rate of equivalency factors when axle loads are higher than the current federal limits. Truck load factors (TLF) or average ESALs per truck, are obtained as the weighted sum of the ESAL factors. Figure 2.6 shows the total ESALs for some common trucks along with their axle load distribution on AC pavement (structural number = 5; terminal serviceability = 2.5) as well as on PCC pavement (slab thickness = 10 inch; terminal serviceability = 2.5). It is evident that the number of axles is also an important factor; other things being equal, a vehicle with more axles has less ESALs. Thus, a Nine-Axle Double (3-S2-4) truck weighing 129,000 lb has much less ESAL on AC pavement than a Five-Axle Double (2-

S1-2) truck carrying only 80,000 lb (Fig. 2.6). However, a comparison of vehicles in terms of the ESALs does not account for the fact that vehicles with higher weights require fewer trips to transport the same amount of freight, thereby offsetting part of the additional pavement wear caused by increased weight. To circumvent this problem, vehicles can be compared in terms of ESALs per unit weight of freight. Figure 2.7 shows that a conventional Five-Axle Tractor Semi-trailer (3-S2), having 130 kips GVW, has about 7 times as much ESALs per million pounds of freight as a Nine-Axle Double (3-S2-4) having the same GVW on flexible pavement. This factor is approximately 8 for rigid pavements (Fig. 2.8).

The directional distribution factor (D_d) is generally taken as 0.5 for most cases, although it may vary from 0.3 to 0.7 depending upon the truck condition, "loaded" or "unloaded." The lane distribution factor (L_d) is the distribution of truck loads by travel lane during an average day. The design 18-kip ESALs is estimated as follows:

$$\log W_{t18} = \log\left(ADT \cdot P \cdot D_d \cdot L_d \cdot TF \cdot 365 \cdot GF_n\right) \tag{2.1}$$

where

W_{t18} = predicted traffic in terms of accumulated number of ESALs (lb) during the analysis period;

ADT = average daily traffic;

P = percent trucks;

D_d = directional distribution;

L_d = lane distribution;

TF = truck load factor; and

GF_n = growth factor for a prescribed design period.

The probability that a pavement system will perform its intended functions over its design life, under the conditions encountered during its operation, is considered a reliability factor. The levels of reliability for various functional classifications are given in Table 2.8, as recommended by AASHTO(1986). The overall standard deviation (traffic prediction plus performance prediction) for flexible and rigid pavement corresponds to 0.45 and 0.35, respectively.

The serviceability is defined as the ability to serve the type of traffic (trucks and automobiles) that use a roadway and it is measured by the Present Serviceability Index (PSI) which ranges from 0 to 5. Designers and analysts consider initial serviceability (p_o) as 5.0 and terminal serviceability (p_t) as 2.5 or greater for major highways and 2.0 for low volume roads. The Design Serviceability Loss (ΔPSI) is the difference of p_0 and p_t. Figure 2.9 shows that there is an exponential relationship between ΔPSI and cumulative ESALs. Temperature and moisture changes also affect the strength, durability, and load carrying capacity and therefore reduce the serviceability.

2.4.1 Flexible Pavement

Figure 2.10 presents the nomograph followed by designers for determining the design SN for specific conditions including W_{t18}, R, S_o, M_R and ΔPSI. This nomograph solves Equation (2.2) which was developed on the basis of a 2-year study, AASHTO Road Test. The long term effects of moisture and temperature were ignored; that is, the ΔPSI due to these effects is not included in Equation (2.2).

$$\log_{10}(W_{t18}) = Z_R \times S_o + 9..36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.4}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07 \qquad (2.2)$$

where the symbols bear their usual meanings.

After determining the structural number (SN) required for the performance period, Equation (2.3) is used to determine the layer thickness.

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \qquad (2.3)$$

where

a_1, a_2, a_3 = layer coefficients of surface, base and subbase courses, respectively;

D_1, D_2, D_3 = actual thickness (in inches) of surface, base and subbase courses, respectively;

m_2, m_3 = drainage coefficients for base and subbase layers, respectively.

It is evident that the SN equation does not have a unique solution. The actual layer thickness of surface, base, and subbase courses are determined by solving a number of equations. The thicknesses of the AC pavement layers are rounded to the nearest half inch. Placing surface, base, or subbase courses of less than some minimum thickness is generally impractical and uneconomical. AASHTO (1986) recommends a minimum thickness for AC and aggregate base as shown in Table 2.9.

Oklahoma Subgrade Index Method

The Oklahoma Subgrade Index (OSI) method is an empirical method for flexible pavement design developed by the Oklahoma Department of Transportation (ODOT) in the early 1960's. The OSI method is followed by some agencies in times for designing Oklahoma county roads. The equivalent base thickness (EBT) is determined on the basis of an OSI number and wheel load.

The wheel load which is considered for design depends on the functional classification of highways and the average daily traffic (ADT). The minimum design wheel load that should be considered for various classifications of highways is presented in Table 2.10. The non-adjusted EBT (determined on the basis of wheel load and OSI number) is adjusted by an ETB adjustment factor to obtain the total EBT ($EBT_{total} = EBT_{non\text{-}adjusted} + EBT_{adjustment\ factor}$). The EBT adjustment factor is related to the STC and shoulder factors. The STC factor is the product of shoulder factor, traffic factor, and climate factor. The traffic factor is directly proportional to ADT, percentage of heavy common trucks (T_3), percentage of overloaded axles (T_0) and lane distribution factor (L_d). The lane distribution factor (L_d) is 1.0 for a 2-lane road, 0.8 for a 4-lane road, and 0.6 for a 6 or more lanes road. It is calculated by using Equation 2.3.

$$\text{Traffic factor} = ADT \times T_3 \times T_o \times L_f \tag{2.3}$$

where, Lx = load in kips in a single axle.

2.4.2 Rigid Pavement

A majority of states in the U.S. design Portland Cement Concrete (PCC) pavement using the AASHTO guidelines, while only a limited number utilize the Portland Cement Association (PCA) method. There are roughly 1000 miles of PCC pavement in the Oklahoma state highway system. Of these, 322 miles of PCC pavement are jointed plain concrete pavement (JPCP), 486 miles are partially reinforced concrete (dowel) pavement and 187 miles are continuously reinforced concrete pavement (CRCP). All PCC pavement are designed based on the 1986 AASHTO procedures (Forsyth 1993, pp. 15).

The design traffic prediction, reliability factor, and standard deviation for PCC pavement are essentially the same as for AC pavement. The effective subgrade reaction (K-value) is directly proportional to the M_R value of roadbed soil. The K-value also depends upon subbase thickness (D_{SB}), loss of support (LS), and depth of rigid foundation. The other inputs are concrete modulus of rupture (S_c), load transfer coefficient (J), and drainage coefficient (C_d). The design slab thickness (D) is determined from nomograph (Fig. 2.11) solving Equation 2.7. In accordance with this Figure, the slab thickness varies exponentially with the design ESALs. The optimum combination of slab and subbase thickness is selected on the basis of economics and other agency policy requirements. The thickness is generally rounded to the nearest inch, but the use of controlled grade slip form pavers may permit half inch increments.

$$\log_{10}(W_{t18}) = Z_R \times S_0 + 7..35 \times \log_{10}(D+1) - 0.60 + \frac{\log\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.624\times10^7}{(D+1)^{8.46}}} +$$

$$(4.22 - 0.32 \times p_t) \times \log 10\left[\frac{S_c' \times C_d \times (D^{0.75} - 1.132)}{215.63 \times J\left[D^{0.75} - \frac{18.42}{(E_c/K)^{0.25}}\right]}\right] \qquad (2.7)$$

2.5 MANIFESTATION OF PAVEMENT DETERIORATION

Typically highway pavement deterioration is manifested in three ways: cracking, permanent deformation, and disintegration. These are discussed briefly below:

- Cracking is characterized by two distinct phases, an initiation phase after construction before the defects first appear on the surface, and a propagation phase during which the defects progressively develop in extent of the surface area and in severity. Cracking results in fatigue damage. Different types of cracking are listed below:
  - (a) Longitudinal cracking: Line cracks which are formed along the longitudinal direction of the pavement.
  - (b) Transverse cracking: Line cracks which are formed along the transverse direction of the pavement.
  - (c) Alligator or crocodile cracking: Interconnected polygons of a diameter less than 12 in.

(d) Map cracking: Interconnected polygons of a diameter more than 12 in.

(e) Irregular cracking: Unconnected cracks without distinct pattern.

(f) Block cracking: Intersecting line cracks in a rectangular pattern with spacing greater than 3.25 ft.

- Permanent Deformation: It is the deviation of the plane surface from its original due to traffic loads and/or other damaging factors. This type of distress can be classified as follows:

  (a) Rutting: The longitudinal deformation in wheel paths.

  (b) Depression: Bowl-shaped depression in surfacing.

  (c) Mount: Localized rise in surfacing.

  (d) Ridge: Longitudinal rise in surfacing.

  (e) Corrugation: Transverse depressions at close spacing

  (f) Undulation: Transverse depression in long spacing (greater than 16 ft).

  (g) Roughness: Irregularity of pavement surface in wheel paths.

  (h) Potholes: An open cavity in surfacing with a diameter greater than 6.0 inch and a depth greater than 2.0 inch.

- Disintegration (surface defect): The construction materials of pavement are disintegrated by various unfavorable factors. Disintegration is classified into the following categories:

  (a) Raveling: Loss of stone particles from surfacing.

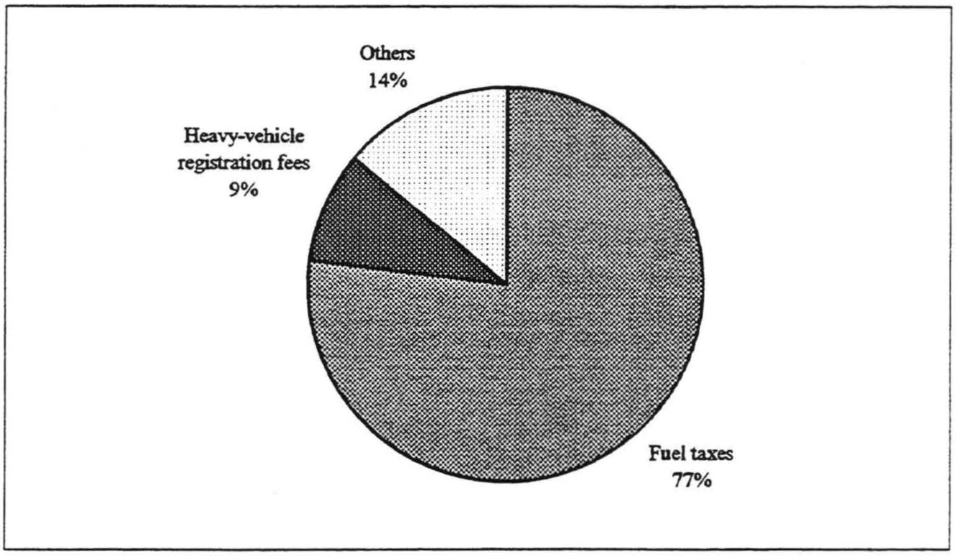
(b) D-cracking: It is associated primarily with the use of coarse aggregates in the concrete that disintegrate of when they become saturated and are subjected to repeated cycles of freezing and thawing.

(c) Edge break: Loss of fragments at the edge of surfacing.

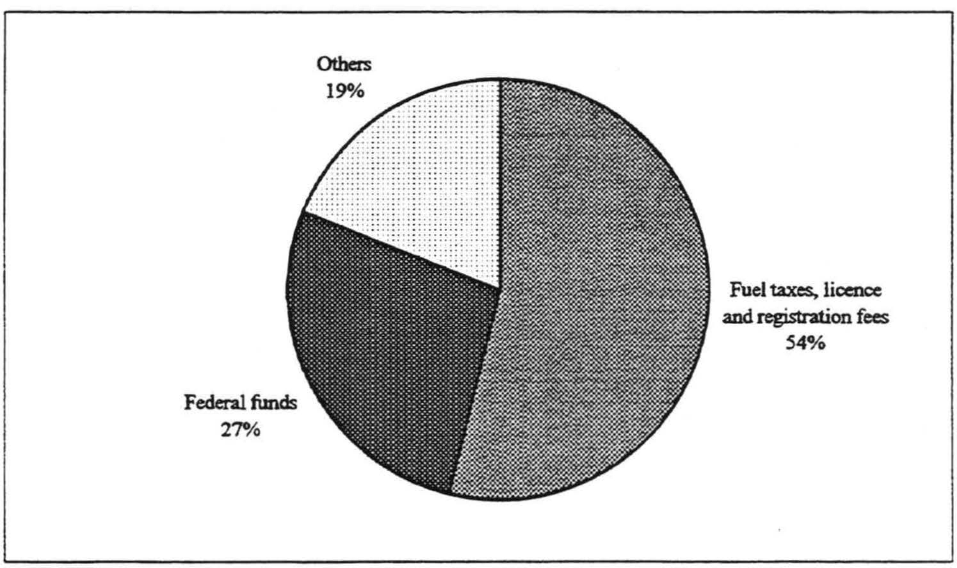
AC pavement can develop cracks, surface deformation and disintegration. PCC pavement also develop various forms of cracking and surface roughness and can exhibit joint-related problems as well. These forms of pavement deterioration or distress can be caused by a variety of factors acting both independently and in combination. These factors can be listed into two broad categories:

- Traffic related factors:
  - (a) Gross vehicle weight (GVW);
  - (b) Load, type and axle spacing;
  - (c) Tire pressure and contact area;
  - (d) Traffic characteristics.
- Non-traffic related factors:
  - (a) Environmental effects;
  - (b) Subgrade earth and rock conditions;
  - (c) Pavement design and construction;
  - (d) Pavement maintenance practices
  - (f) Pavement age etc.

Tables 2.11 and 2.12 indicate whether the mode of some major distresses is associated with load, an environmental problem, or a material problem. Although there is not enough documentation relating increased pavement damage with heavy loaded vehicles, every state has experienced rapidly deteriorating pavements caused by both an increase in the volume of trucks and an overall increase in their axle and GVW. For major highways, it is a common belief that most pavement damage is caused by truck traffic. The AASHTO (1984) publication "Our Highways-Why Do They Wear Out? Who Pays for Them?" clearly states that this may be observed on freeways where a majority of the truck traffic is in the right lane and the majority of damage occur in this lane. The truck-load oriented distresses of pavements are further discussed in Chapters III and IV.



(a) Federal fund



(b) State fund

Fig. 2.1 Source of major highway funding (Reno and Stowers 1995, pp. 40).

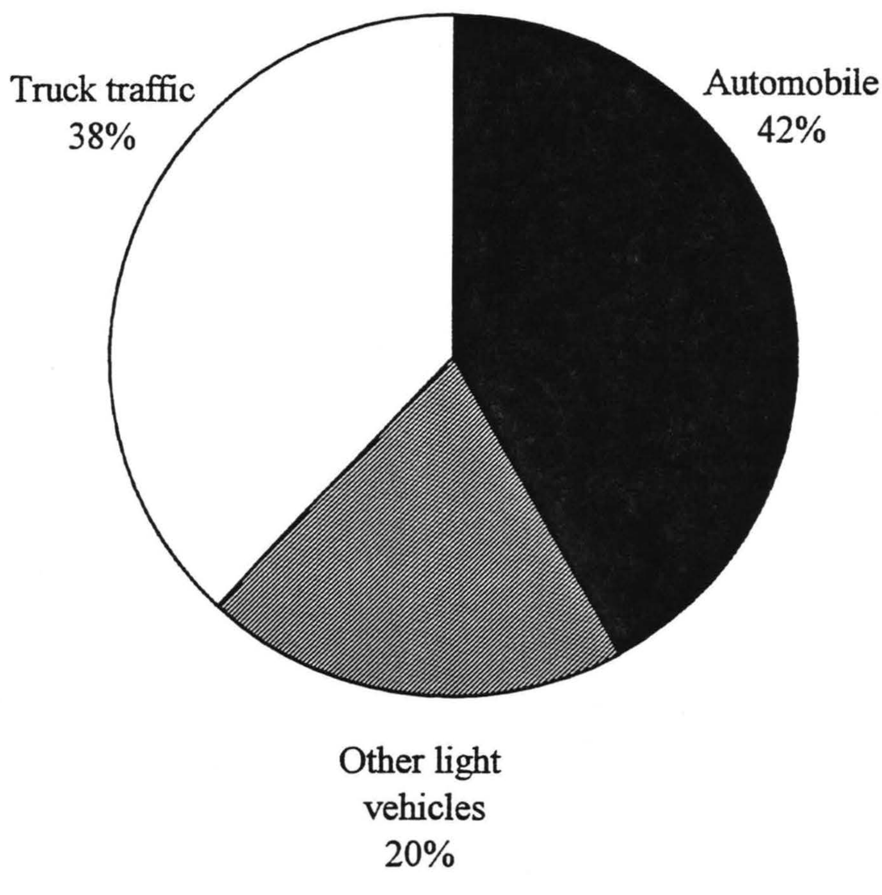


Fig. 2.2 Shares of federal revenue by major class of vehicle.
(TRB 1989, pp. 189).

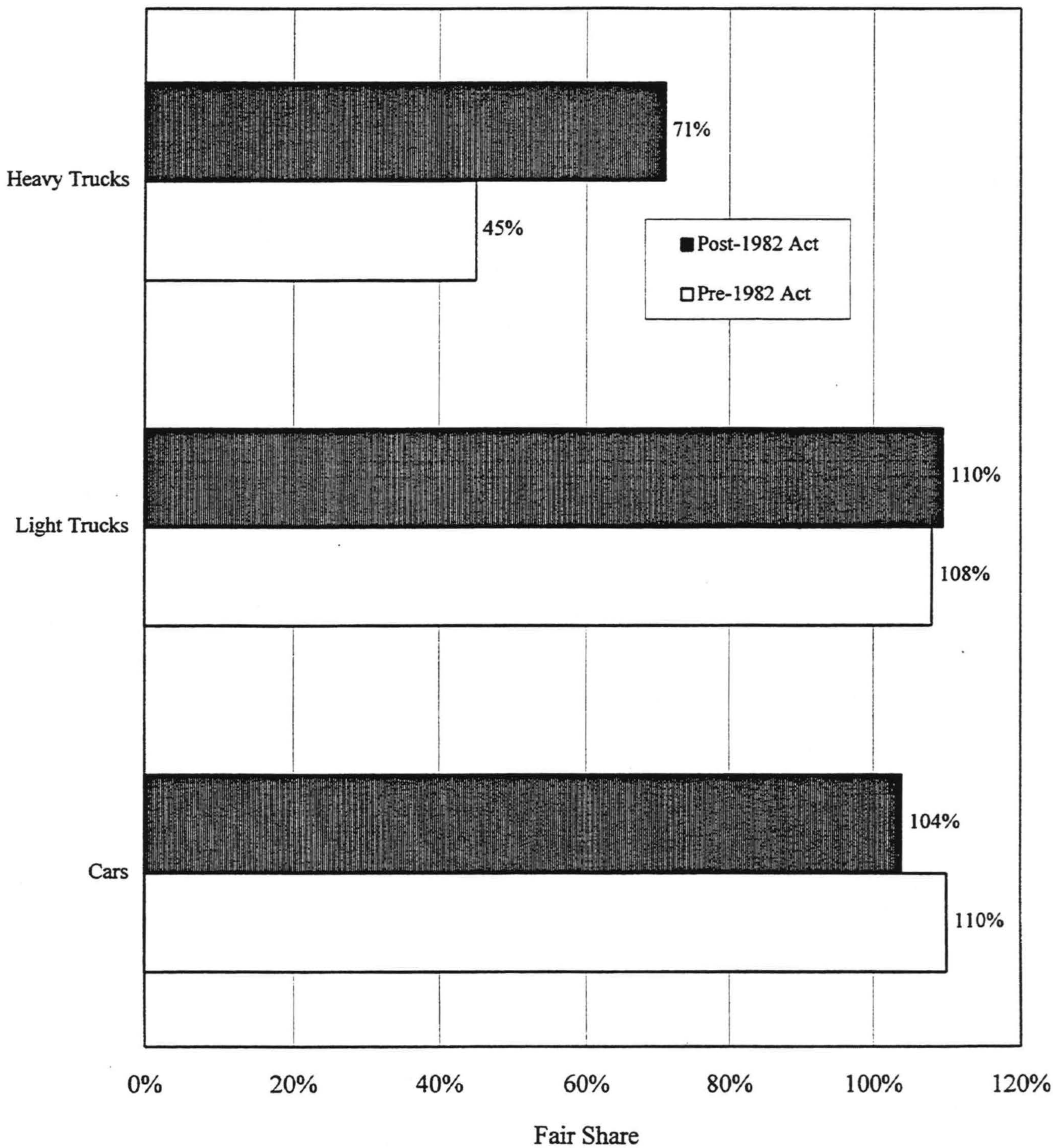


Fig. 2.3 Fair Share of Different Vehicles (USDOT, 1982* and USDOT, 1983*)

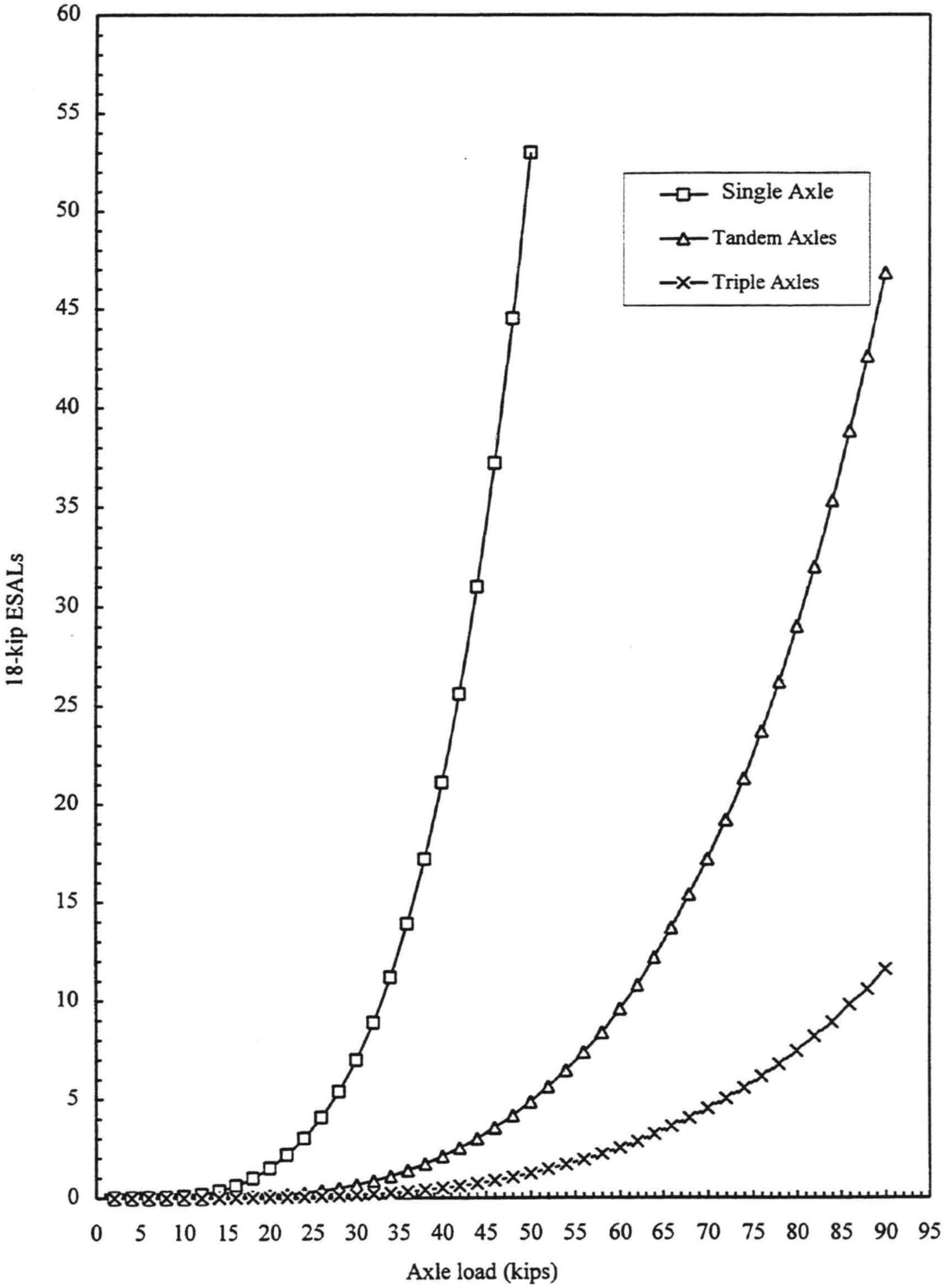


Fig. 2.4 Axle load equivalency factors for flexible pavements(ASSHTO 1986, pp. D-6-D-8).

Note: Structural Number=5; Terminal Serviceability=2.5

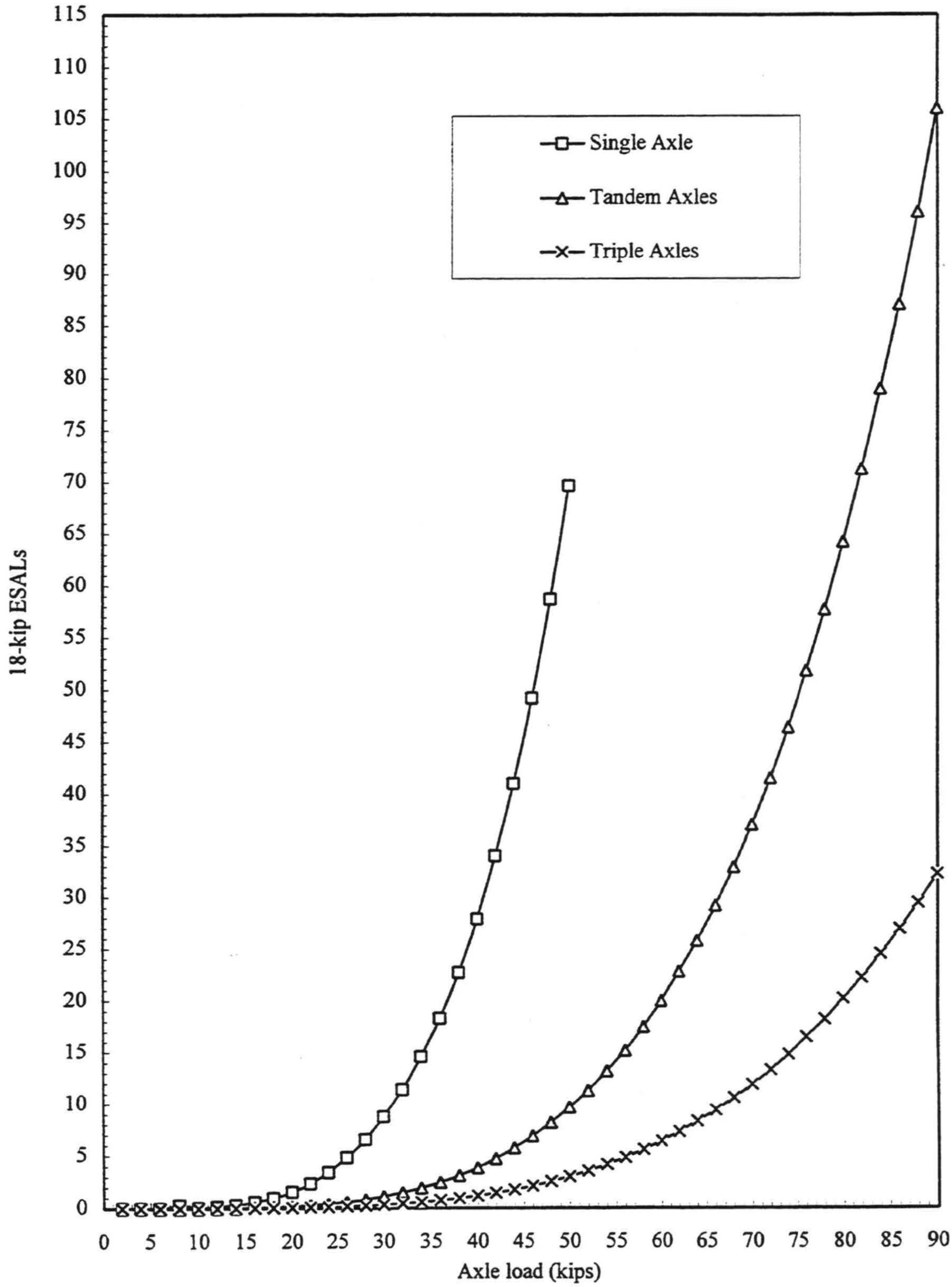


Fig. 2.5 Axle load equivalency fcators for rigid pavements (AASHTO 1986, pp. D-15-D17)

Note: Slab Thickness=10 inch; Terminal Serviceability=2.5

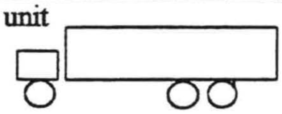
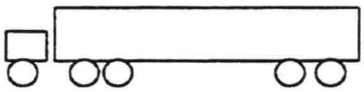
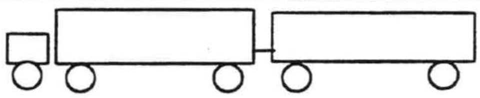
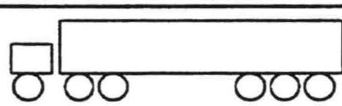
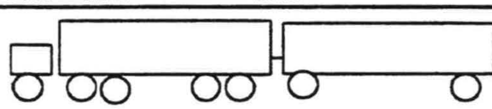
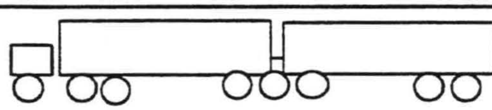
Configuration and Load Distribution of Common Trucks						Total
USA Trucks						
3-axle single unit 						
Weight (kips)	16	32				48
ESALs						
Flexible	0.62	0.86				1.48
Rigid	0.60	1.50				2.10
3-S2 						
Weight (kips)	12	34	34			80
ESALs						
Flexible	0.19	1.09	1.09			2.37
Rigid	0.17	1.95	1.95			4.07
2-S1-2 						
Weight (kips)	9	20	19	16	16	80
ESALs						
Flexible	0.06	1.51	1.24	0.62	0.62	4.05
Rigid	0.05	1.58	1.26	0.60	0.60	4.09
3-S3 						
Weight (kips)	12	34	42			88
ESALs						
Flexible	0.19	1.09	0.06			1.88
Rigid	0.17	1.95	1.45			3.57
3-S2-2 						
Weight (kips)	9	31	30	16	15	101
ESALs						
Flexible	0.06	0.75	0.66	0.62	0.48	2.57
Rigid	0.05	1.31	1.14	0.60	0.46	3.56
3-S3-2 						
Weight (kips)	12	34	42	34		122
ESALs						
Flexible	0.19	1.09	0.60	1.09		2.97
Rigid	0.17	1.95	1.45	1.95		5.52

Fig. 2.6 Equivalent single-axle load for some common trucks. *Flexible pavements*: structural number = 5; terminal serviceability = 2.5; *Rigid pavements*: slab thick=10 in.; terminal serviceability=2.5 (TRB 1990a, pp.78-79).

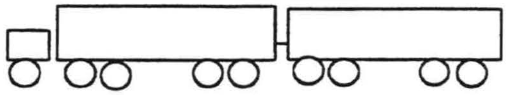
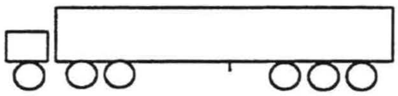
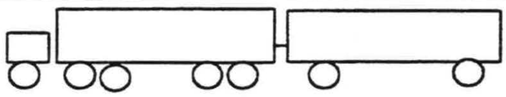
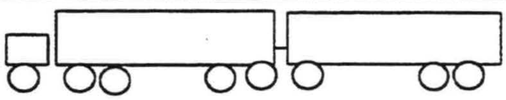
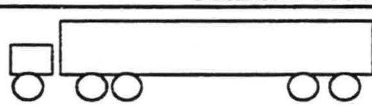
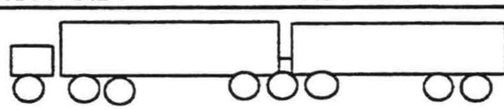
Configuration and Load Distribution of Common Trucks						Total
3-S2-4						
Weight (kips)	12	33	28	28	28	129
ESALs						
Flexible	0.19	0.97	0.50	0.50	0.50	2.66
Rigid	0.17	1.71	0.85	0.85	0.85	4.43
Canadian Interprovincial Trucks						
TST						
Weight (kips)	13.2	40.8		57.7		111.7
ESALs						
Flexible	0.289	2.25		2.15		**4.69**
Rigid	0.270	3.66		5.04		**8.97**
A and C-train doubles						
Weight (kips)	13.2	40.8	40.8	16.85	16.85	128.5
ESALs						
Flexible	0.289	2.25	2.25	0.78	0.78	**6.35**
Rigid	0.27	3.66	3.66	0.77	0.77	**9.13**
B-train doubles						
Weight (kips)	13.2	40.8	55.2		40.8	150
ESALs						
Flexible	0.289	2.25	1.80		2.25	**6.60**
Rigid	0.270	3.66	4.23		3.66	**11.84**
Mexican Trucks						
3-S2						
Weight (kips)	12.1	39.6		39.6		91.3
ESALs						
Flexible	0.204	1.99		1.99		**4.18**
Rigid	0.180	3.24		3.24		**6.66**
3-S3-2						
Weight (kips)	12.1	39.6	49.5		39.6	140.8
ESALs						
Flexible	0.204	1.99	1.17		1.99	**5.35**
Rigid	0.180	3.24	2.73		3.24	**9.39**

Fig. 2.6 Continued.

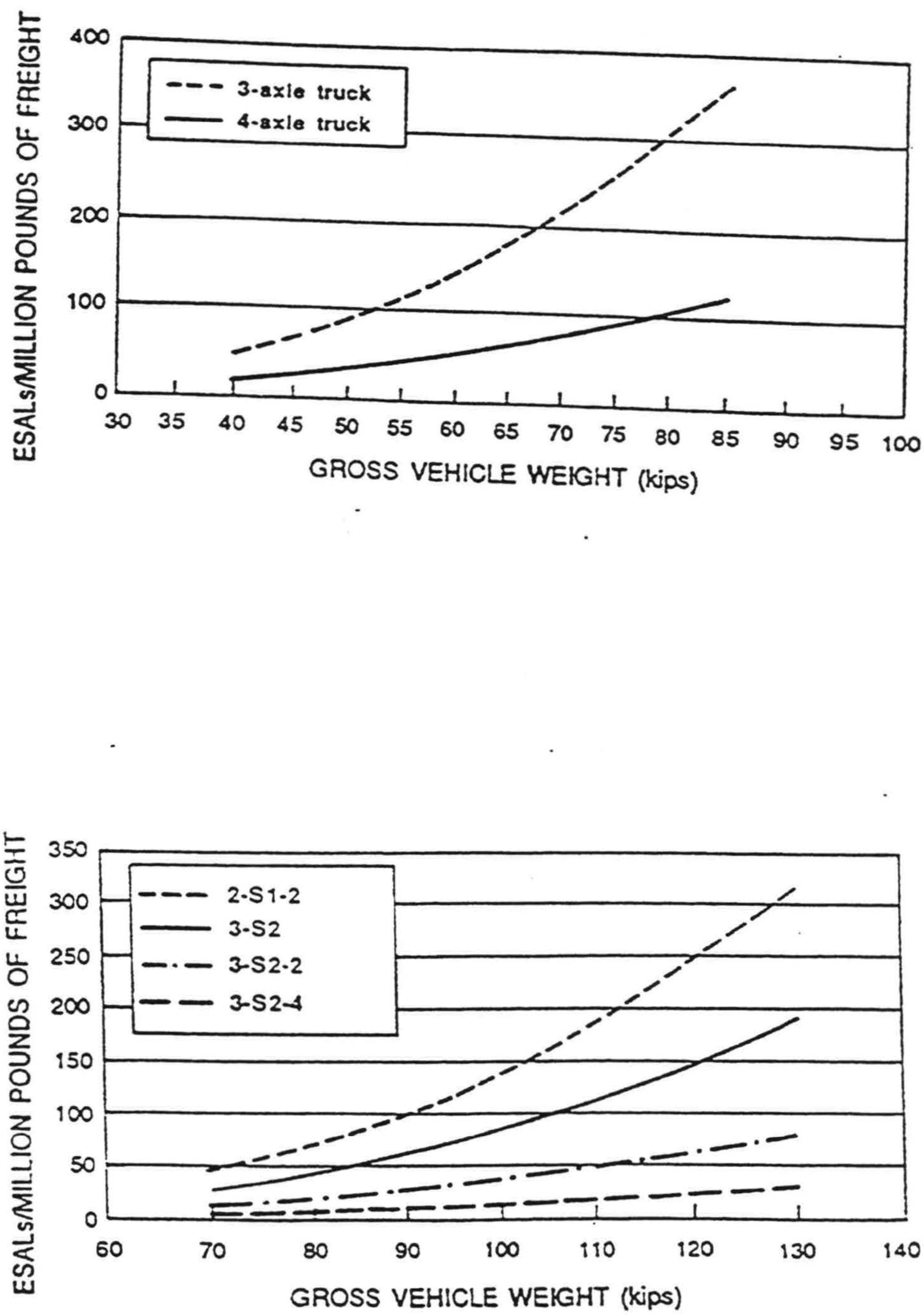
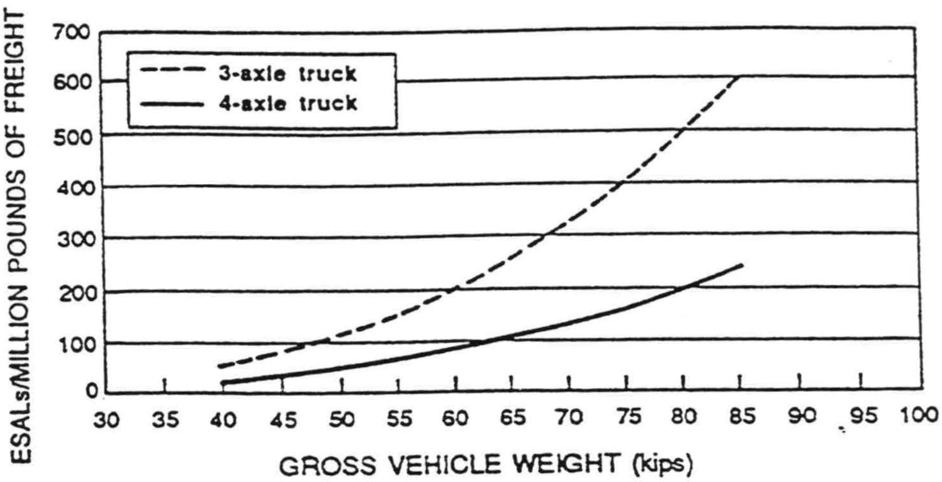


Fig. 2.7 ESALs for various trucks per million pounds of freight on flexible pavements (TRB 1990a, pp. 80-81)



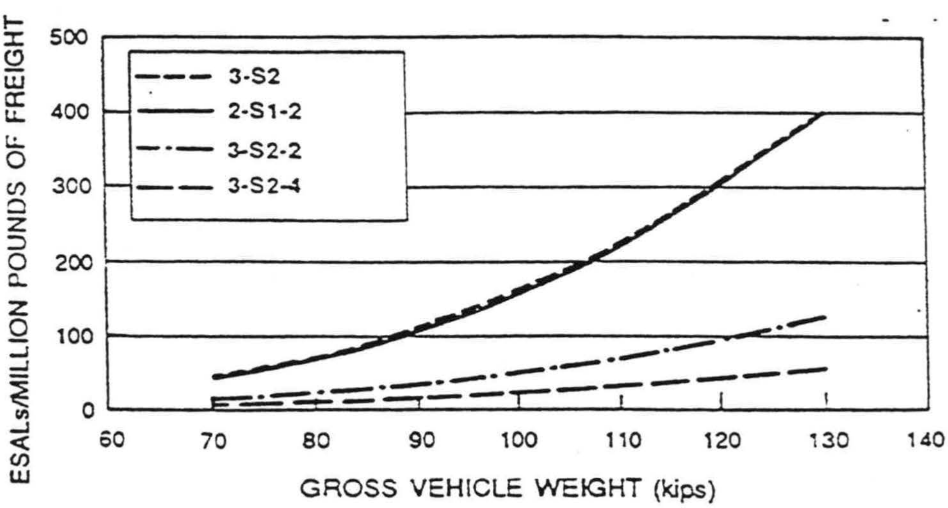


Fig. 2.8 ESALs for various trucks per million pounds of freight on rigid pavements (TRB 1990a, pp. 80-81)

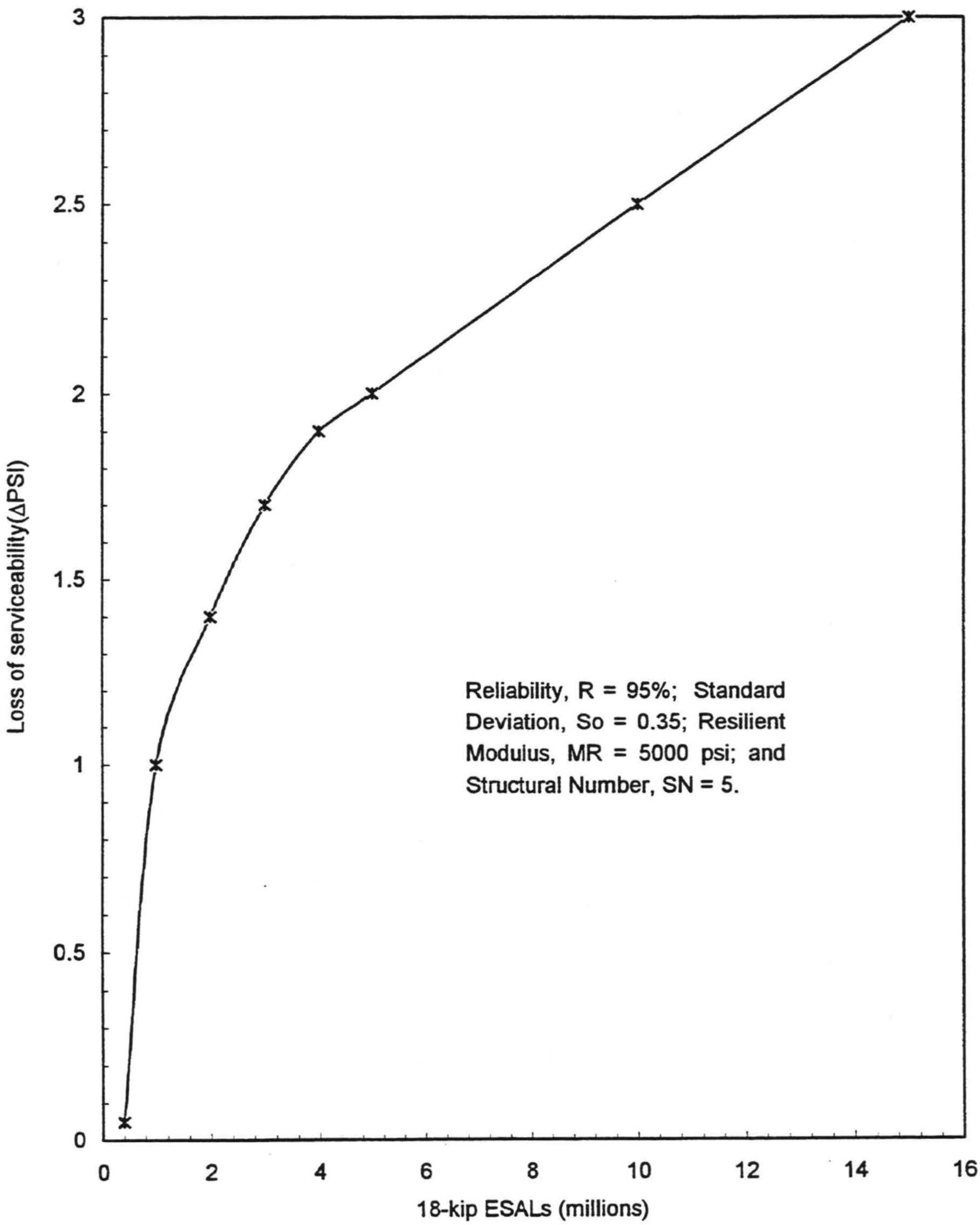


Fig. 2.9 Sensitivity of loss of serviceability to cumulative load repetitions (AASHTO 1993, pp. II-35)

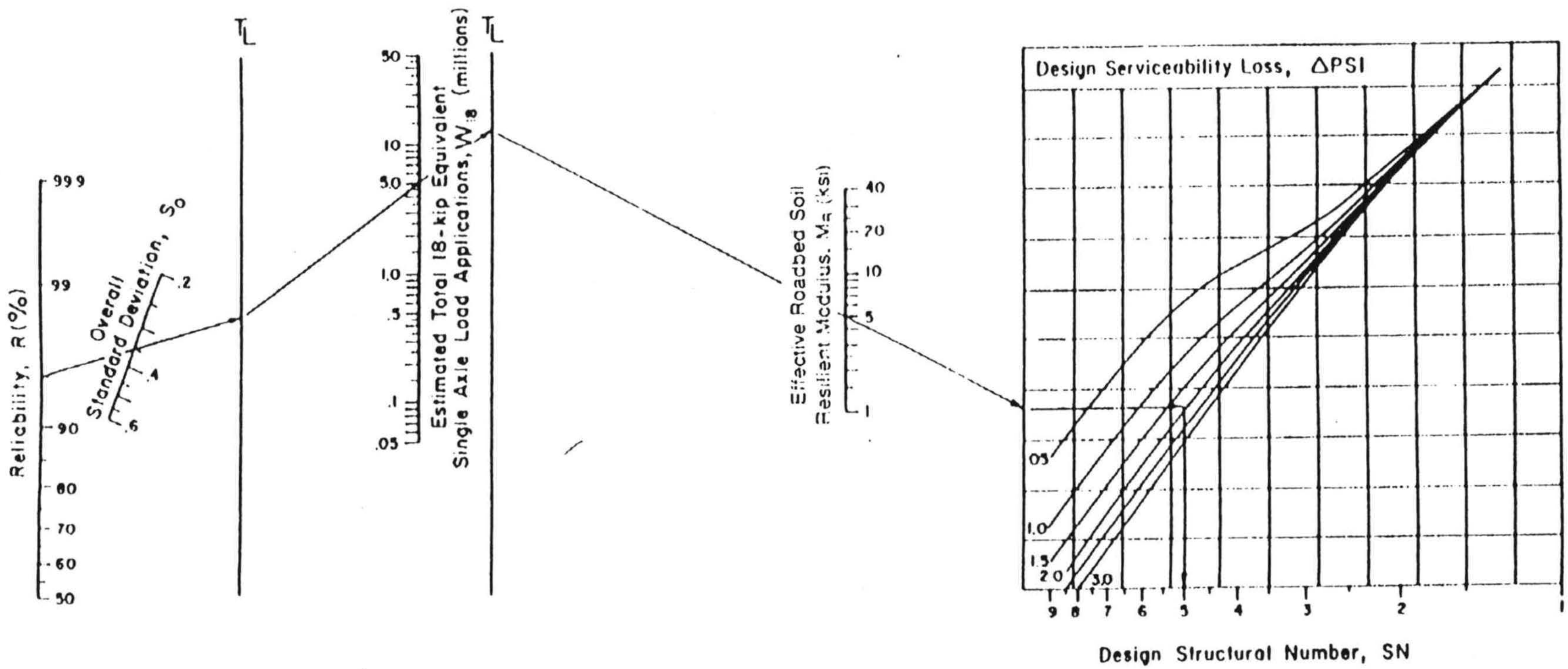


Fig. 2.10 Design chart for flexible pavements (AASHTO 1986, pp. II-35).

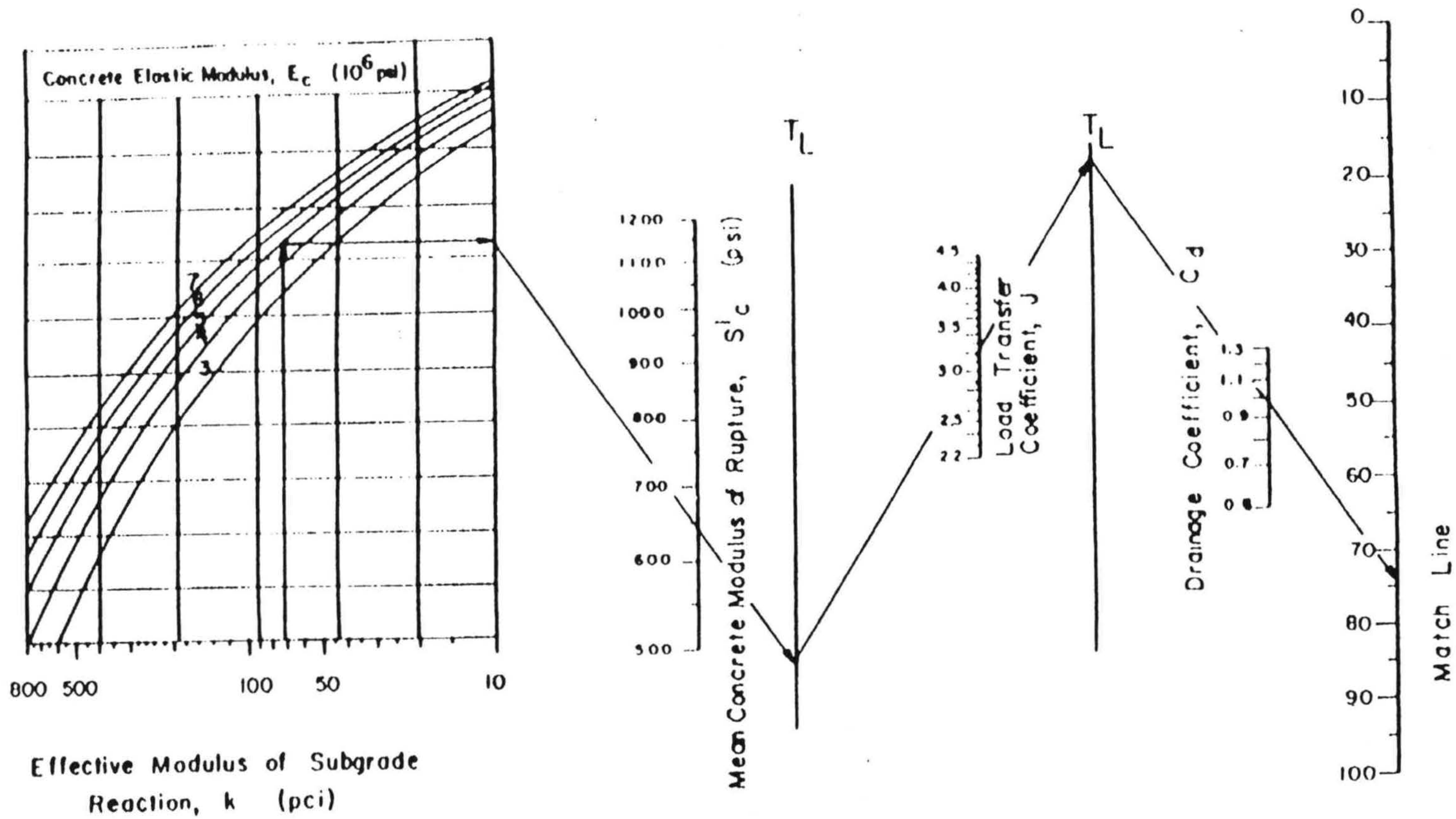


Fig. 2.11 Design chart for rigid pavements (AASHTO 1986, pp. II-46, II-47).

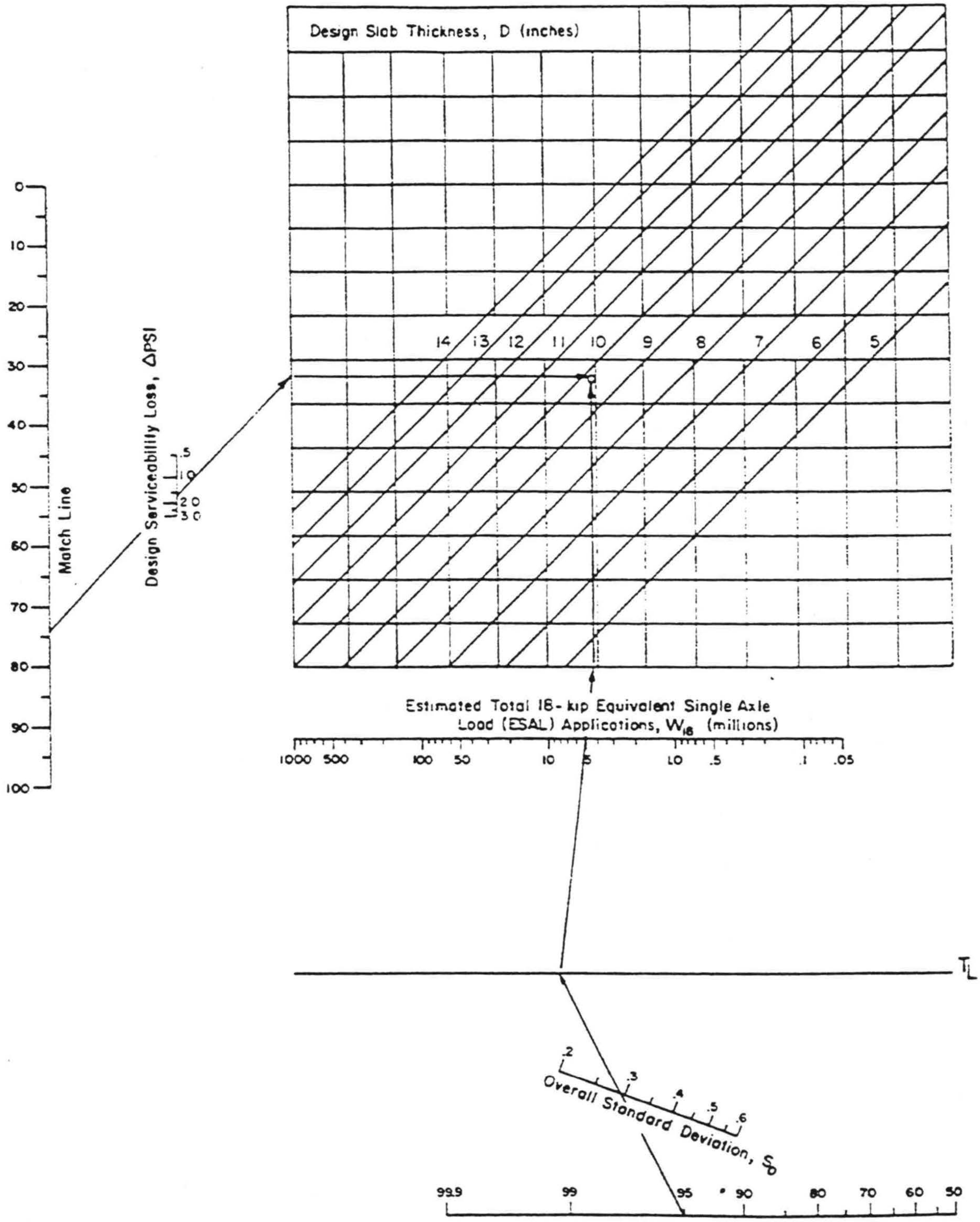


Fig. 2.11 Continued.

Table 2.1 Registration Fees Schedule for Commercial Trucks, Truck-Tractors, Trailers, and Semi-trailers of GVW over 8,000 lb (OS 1991, pp. 4760-4761).

Gross vehicle weight (lb)		License fee (US Dollars)
From	To	
8,001	15,000	95.00
15,001	18,000	120.00
18,001	21,000	155.00
21,001	24,000	190.00
24,001	27,000	225.00
27,001	30,000	260.00
30,001	33,000	295.00
33,001	36,000	325.00
36,001	39,000	350.00
39,001	42,000	375.00
42,001	45,000	400.00
45,001	48,000	425.00
48,001	51,000	450.00
51,001	54,000	475.00
54,001	57,000	648.00
57,001	60,000	681.00
60,001	63,000	713.00
63,001	66,000	746.00
66,001	69,000	778.00
69,001	72,000	817.00
72,001	73,280	857.00
73,281	74,000	870.00
74,001	75,000	883.00
75,001	76,000	896.00
76,001	77,000	909.00
77,001	78,000	922.00
78,001	79,000	935.00
79,001	80,000	948.00
80,001	81,000	961.00
81,001	82,000	974.00
82,001	83,000	987.00
83,001	84,000	1000.00
84,001	85,000	1013.00
85,001	86,000	1026.00
86,001	87,000	1039.00
87,001	88,000	1052.00
88,001	89,000	1065.00
89,001	90,000	1078.00

Table 2.2 Permissible Gross Loads for Vehicles in Regular Operation (TRB 1990a, pp. 40-41).

Distance between the extremes of any group of two or more consecutive axles (ft)	Maximum load (lb) by number of axle groups							
	2 axles	3 axles	4 axles	5 axles	6 axles	7 axles	8 axles	9 axles
4	34,000							
5	34,000							
6	34,000							
7	34,000							
8	34,000	34,000						
9	39,000	42,500						
10	40,000	43,500						
11		44,000						
12		45,000	50,000					
13		45,500	50,500					
14		46,500	51,500					
15		47,000	52,000					
16		48,000	52,500	58,000				
17		48,500	53,500	58,500				
18		49,500	54,000	59,000				
19		50,000	54,500	60,000				
20		51,000	55,500	60,500	66,000			
21		51,500	56,000	61,000	66,500			
22		52,500	56,500	61,500	67,000			
23		53,000	57,500	62,500	68,000			
24		54,000	58,000	63,000	68,500	74,000		
25		54,500	58,500	63,500	69,000	74,500		
26		55,500	59,500	64,000	69,500	75,000		
27		56,000	60,000	65,000	70,000	75,500		
28		57,000	60,500	65,500	71,000	76,500	82,000	
29		57,500	61,500	66,000	71,500	77,000	82,500	
30		58,500	62,000	66,500	72,000	77,500	83,000	
31		59,000	62,500	67,500	72,500	78,000	83,500	
32		60,000	63,500	68,000	73,000	78,500	84,500	90,000
33			64,000	68,500	74,000	79,000	85,000	90,500
34			64,500	69,000	74,500	**80,000**	85,500	91,000
35			65,500	70,000	75,000	80,500	86,000	91,500
36			66,000	70,500	75,500	81,000	86,500	92,000
37			66,500	71,000	76,000	81,500	87,000	93,000
38			67,500	71,500	77,000	82,000	87,500	93,500
39			68,000	72,500	77,500	82,500	88,500	94,000
40			68,500	73,000	78,000	83,500	89,000	94,500
41			69,500	73,500	78,500	84,000	89,500	95,000
42			70,000	74,000	79,000	84,500	90,000	95,500
43			70,500	75,000	**80,000**	85,000	90,500	96,000
44			71,500	75,500	80,500	85,500	91,000	96,500
45			72,000	76,000	81,000	86,000	91,500	97,500

Table 2.2 Continued

Distance between the extremes of any group of two or more consecutive axles (ft)	Maximum load (lb) by no of axle groups							
	2 axles	3 axles	4 axles	5 axles	6 axles	7 axles	8 axles	9 axles
46			72,500	76,500	81,500	87,000	92,500	98,000
47			73,500	77,500	82,000	87,500	93,000	98,500
48			74,000	78,000	83,000	88,000	93,500	99,000
49			74,500	78,500	83,500	88,500	94,000	99,500
50			75,500	79,000	84,000	89,000	94,500	100,000
51			76,000	**80,000**	84,500	89,500	95,000	100,500
52			76,500	80,500	85,000	90,500	95,500	101,000
53			77,500	81,000	86,000	91,000	96,500	102,000
54			78,000	81,500	86,500	91,500	97,000	102,500
55			78,500	82,500	87,000	92,000	97,500	103,000
56			79,500	83,000	87,500	92,500	98,000	103,500
57			**80,000**	83,500	88,000	93,000	98,500	104,000
58				84,000	89,000	94,000	99,000	104,500
59				85,000	89,500	94,500	99,500	105,000
60				85,500	90,000	95,000	100,500	105,500

Note: The weights in this table are based on the formula W=500[LN/(N-1)+12N+36], modified. The permissible loads are computed to the nearest 500 lb. The modification consists in limiting the maximum load on any single axle to 20,000 lb.

[a] The following loaded vehicles must not operate over H15-44 bridges: 3-S2 (five axles) with wheelbase less than 38 ft; 2-S1-2 (five axles) with wheelbase less than 45 ft; 3-3 (six axles) with wheelbase less than 45 ft; and seven-, eight-, and nine- axle vehicles regardless of wheelbase.

Table 2.3 Vehicle Sizes and Weights: Maximum Limits, January 1, 1991 (AASHTO 1995, pp. A4-A7)

DES. = Interstate and federally designed state highways.
OTHER = All other state highways and supplemental routes

State	Height (feet)	Width (inches)		Weight (1,000 pounds)					
				Single axle weight		Double axle weight		Gross vehicle weight	
		DES	OTHER	INT.	OTHER	INT.	OTHER	INT.	OTHER
Alabama	13.5	102	L	20	20	34	40	80	84
Alaska	14	102	102	20	20	38	38	K	NS
Arizona	13.5	102	96	20	20	34	34	80	80
Arkansas	13.5	102	102	20	20	34	34	80	80
California	14	102	102	20	20	34	34	80	80
Colorado	14.5	102	102	20	20	36	40	80	85
Connecticut	13.5	102.36	102.36	22.4	22.4	36[W]	36[W]	80	80
Delaware	13.5	102	102	20	20	34	40	80	80
District of Columbia	13.5	102	102	22	22	38	38	80	80
Florida	13.5	102	102	22	22	44	44	80	80
Georgia	13.5	102	102	P	P	Q	37.34	80	80
Hawaii	13.5	108	108	22.5	22.5	34	34	80.8	88
Idaho	14	102	102	20	20	34	34	80	105.5
Illinois	13.5	H	H	20[E]	18	34[E]	32	80[E]	73.28
Indiana	13.5	102	102	20	20	34	34	80	80
Iowa	13.5	102	96	20	20	34	34	80	80
Kansas	14	102	102	20	20	34	34	80	85.5
Kentucky	13.5	102	96	20	20	34	34	80	J
Louisiana	13.5	102	96	20	22	34	37	80	80
Maine	13.5	102	102	R	22.4	34	38	80	80
Maryland	13.5	102	96	Z	Z	Z	Z	80	80
Massachusetts	13.5	102	102	22.4	22.4	36	36	80	80
Michigan	13.5	102	96	JJ	JJ	JJ	JJ	JJ	JJ
Minnesota	13.5	102	102	20	18	34	34	80	80
Mississippi	13.5	102	102	20	20	34	34	80[V]	80[V]
Missouri	14[KK]	102	96	20	18	34	32	80	73.28
Montana	14	102	102	20	20	34	34	80	80
Nebraska	14.5	102	102	20	20	34	34	80	95
Nevada	14	102	102	20	20	34	34	80	M
New Hampshire	13.5	102	102	Z	Z	22.4	36	80	80
New Jersey	13.5	102	96	22.4	22.4	34	34	80	80
New Mexico	14	102	102	21.6	21.6	34.32	34.32	86.4	86.4
New York	13.5	102	L	20[II]	22.4	34[II]	36	80	80
North Carolina	13.5	102	102	20	20	38	38	80	80
North Dakota	13.5	102	102	20	20	34	34	80	105.5
Ohio	13.5	102	102	20	20	X	X	80	80
Oklahoma	13.5	102	102	20	20	34	34	80	90
Oregon	14	102	102	20	20	34	34	80	80
Pennsylvania	13.5	102	96	22.4[Z]	22.4[Z]	36[Z]	36[Z]	80	80
Rhode Island	13.5	102	102	22.4	22.4	44[RR]	44[RR]	80	80
South Carolina	13.5	102	96	20	22	35.2[FF]	39.6	80	80.6
South Dakota	14	102	102	20	20	34	34	80	K
Tennessee	13.5	102	102	20	20	34	34	80	80
Texas	13.5	102	102	20	20	34	34	80	80
Utah	14	102	102	20	20	34	34	80	80
Vermont	13.5	102	102	22.5	22.5	36	36	80	80
Virginia	13.5	102	96	20	20	34	34	80	80
Washington	14	102	102	20	20	34	34	80	80
West Virginia	13.5	102	96	20	20	34	34	80	65[DD]
Wisconsin	13.5	102	102	20	20	34	34	80	80
Wyoming	14	102	102	20	20	36	36	80	80

Table 2.3 Continued

DES. = Interstate and federally designed state highways.
OTHER = All other state highways and supplemental routes.

	State					OTHER				
	Straight trucks	Combination s +		Trailing units ¦		Straight trucks	Combinations +		Trailing units !	
	Single unit	Tractor-semi-trailer	Tractor-twin-trailer	Semi-trailer	Trailer	Single unit	Tractor-semi-trailer	Tractor-twin-trailer	Semi-trailer	Trailer
Alabama	40	Ø	Ø	53	28.5	40	Ø	Ø	53	28.5
Alaska	40	Ø	Ø	48	48	40	70	75	45	45
Arizona	40	Ø	Ø	57.5	28.5	40	65	Ø	51	28.5
Arkansas	40	Ø	65	53.5	28.5°	40	Ø	65	53.5	28.5°
California	40	B	B	B	B	40	B	B	B	B
Colorado	40	Ø	Ø	57.33[D]	28.5[D]	40	Ø	Ø	57.33[D]	28.5[D]
Connecticut	60	Ø	Ø	48	28	60	Ø	Ø	48	28
Delaware	40	Ø	Ø	53	29	40	60	60	NS	NS
District of Columbia	40	Ø	Ø	48	28	40	55	A	NS	A
Florida	F	Ø	Ø	53[OO]	28	F	Ø	A	53[OO]	A
Georgia	60	Ø	Ø	53[NN]	28	60[NN]	60[NN]	A[NN]	53[NN]	A[NN]
Hawaii	40	NS	NS	NS	NS	40	60	65	NS	NS
Idaho	40	Ø	Ø	48	61[GG]	40	Ø	Ø	48	61[GG]
Illinois	42	G	Ø	53[G]	28.5	42	G	G	53[G]	28.5
Indiana	36	Ø	Ø	53[HH]	28.5	36	Ø	Ø	53[HH]	28.5
Iowa	40	Ø	Ø	53	28.5	40	60	60	NS	NS
Kansas	42.5	Ø	Ø	53	28.5	42.5	Ø	Ø	53	28.5
Kentucky	45	Ø	Ø	53	28	45	55	A	NS	A
Louisiana	40	Ø	Ø	59.5	30	40	65	A	50	A
Maine	45	Ø	Ø	48	28.5	45	65	A	48	A
Maryland	40	Ø	Ø	48	28	40	Ø	A	48	A
Massachusetts	40	Ø	Ø	48	28	40	60	A	48[BB]	A
Michigan	40	Ø	59	53[QQ]	28.5	40	Ø	59	50	NS
Minnesota	40	Ø	Ø	53[EE]	28.5	40	65	E	48[EE]	28.5[EE]
Mississippi	40	Ø	Ø	53	30	40	Ø	Ø	53	30
Missouri	40	Ø	Ø	53	28	40	60	65	NS	NS
Montana	40	Ø	Ø	53	28.5	40	Ø	Ø	53	28.5
Nebraska	40	Ø	Ø	53	65[I]	40	Ø	Ø	53	65[I]
Nevada	40	Ø[I]	Ø[I]	53[I]	28.5[I]	40	Ø[I]	Ø[I]	48[I]	28.5[I]
New Hampshire	40	NS	NS	48	28	40	Ø	Ø	48	28
New Jersey	35	Ø	Ø	48	28	35	Ø	A	48	28
New Mexico	40	Ø	Ø	57.5	28.5	40	65	65	NS	NS
New York	35	Ø	Ø	48	28.5	35	60[AA]	60	45[AA]	NS
North Carolina	F	Ø	Ø	53[OO]	28	F	60	A	NS	A
North Dakota	50	Ø	Ø	53	53	50	75[U]	75[U]	53	53
Ohio	40	Ø	Ø	53	28.5	40	Ø	Ø	53	28.5
Oklahoma	45	Ø	Ø	C	C	45	Ø	Ø	59	29
Oregon	40	Ø	Ø	53	N	40	N	N	N	N
Pennsylvania	40	Ø	Ø	48[J]	28.5	40	60	A	NS	A
Rhode Island	40	Ø	Ø	48.5	28.5	40	Ø	Ø	48.5	28.5
South Carolina	F	Ø	Ø	53[OO]	28.5	F	60	A	45	A
South Dakota	45	Ø	Ø	53	S	45	Ø	Ø	53	S
Tennessee	40	Ø	Ø	50[LL]	28.5	40	Ø	A	50[LL]	A
Texas	45	Ø	Ø	59	28.5	45	Ø	Ø	59	28.5
Utah	45	92	92	48	61[Y]	45	Ø	Ø	48	61[Y]
Vermont	60	Ø	Ø	48	28	60	65[MM]	A	45[MM]	A
Virginia	40	Ø	Ø	53	28.5	40	60	A	NS	A
Washington	40	Ø	Ø	48	60[Y]	40	Ø	Ø	48	60[Y]
West Virginia	40	Ø	Ø	48	28.5	40	60	A	NS	A
Wisconsin	40	Ø	Ø	53[FF]	28.5	40	60	A	48	A
Wyoming	60	Ø	Ø	60	CC	60	Ø	Ø	60	CC

Note: No state shall prohibit the use of trailers of such dimensions as those that were in actual or lawful use in such state on December 1, 1982. Neither shall any state prohibit the use of existing trailers or semitrailers of up to 28.5 feet in length in a truck tractor-semitrailer-trailer combination if those trailers and semitrailers were actually and lawfully operating on December 1, 1982, within a 65-foot length limit in any state.

TOLERANCES:
Alabama-10% weight tolerance on other roads
California-200 lbs on Platform Scales, or 2% of scale wt. on Platform Scales.
Connecticut-2% tolerance if below 73,000 lbs.
District of Columbia-1,000 lbs tolerance on GVW.
Hawaii-5% weight tolerance on state and supplemental routes only.
Kentucky-5% weight tolerance on length.
Maryland-1,000 lb. tolerance on GVW.

Table 2.3 Continued

Mississippi-None on federal highways, 5% on tandem and 2% on gross on selected other highways.
Missouri-If on highways other than Interstate, can exceed axle and gross weight limitations up to 2,000 lbs.
Montana-Up to 5% (7% for livestock). $10 trip permit fee charged.
New Hampshire-5% tolerance below 80,000 lbs on supplemental highways only.
Pennsylvania-3% on axle weight except when weighted on stationary scales on Interstate highways.
Vermont-On other highways only- 10% on axles, 5% on gross.

+	Only tractor-semitrailer and tractor-twin-trailer combinations are considered here. For other combination, contact state agency.
!	Semi-trailer in tractor-semitrailer combination, and trailer in tractor-twin-trailer combination
Ø	Not overall length restrictions imposed.
NS	Not specified.
A	Not allowed (allowed in some states by permit).
B	On any highways tractor semi-trailer combination 65 ft. (distance between kingpin and rearmost semitrailer axle must be 40 ft. or less: single axle semitrailer kingpin dimension is limited to 38 ft.). On federally designated highways, no overall combination length limitation or kingpin restriction if semitrailer is 48 ft. or less. Or, semitrailer may be 53 ft. if kingpin to centerline of rearmost axle of tandems is no longer than 40 ft. Single rear axle is limited to 38 ft. twin-trailer combinations 65 ft. on all highways if either trailer exceeds 28.5 ft.; 75 ft. on non-designated highways if neither trailer exceeds 28.5 ft; and unlimited length on federally designated system if neither trailer exceeds 28.5 ft.
C	No limit on Interstate or 4-lane highways, otherwise 59-foot semitrailer and 29-foot twin trailers.
D	Combinations with semitrailers or twin-trailers in excess of limits may not exceed 70 ft.
E	On class I, II, and III highways.
F	2 axles, 35 ft. : 3 axles, 40 ft.
G	Any semitrailer operated on any highways whose length exceeds 48 ft., is limited to a maximum distance of 42 ft and 6 in. from kingpin to center of rear most axle. On class II, III and non-designated highways, maximum tractor-semitrailer wheelbase, 55 ft. On class II highways., maximum tractor-twin-trailer wheel base, 65 ft. On class III and non-designated highways, maximum combination vehicle length, 60 ft.
H	102 in. on class I and II highways: 96 in. on class III and non-designated highways.
I	53 ft. long, 8 ft. wide trailer also legal if total length does not exceed 60 ft.
J	80,000 lbs on class AAA highways: 62,000 lbs on class AA highways: and 44,000 lbs on class A highways.
K	GVW is governed by Bridge formula.
L	53 ft. trailers permitted if distance between last axle of tractor and first axle of semitrailer does not exceed 37 feet.
M	Uncapped Federal Bridge Formula
N	Tractor-semitrailer combination 60 ft. or group 1 highways: 50 ft for groups 2 and 3 highways. Semitrailers not specified for group 1: 40 ft for group 2: and 35 ft. for group 3. Tractor- twin-trailers 75 ft. for group 1: 65 ft. for group 2: and 50 ft. for group 3. Trailers 40 ft. for group 1: 35 ft. for groups 2 and 3. On Interstate and designated highways, no semitrailer or trailer in a twin-trailer combination may exceed 40 ft.: both trailing units together measured from the front of the first to the rear of the second may not exceed 68 ft.
O	28.5 ft. if trailer was manufactured prior to December 2, 1982: 28 ft if trailer was manufactured after December 1, 1982.
P	18,000 lb. +13%.
Q	34,000 lb. Exception: If vehicle is less than 55 ft. long and gross weight is less than 73,280 lb. will allow 40,680 lb.
R	Single axle 22,000 lbs. if GVW is less than 73,280 lbs.: and 20,000 lbs. if GVW is more than 73,280 lbs. but less than 80,000 lbs.
S	28.5 ft. on each trailer unit operating in a road tractor-trailer-trailer combination if the towbars do not exceed 19 ft. and the overall length of the trailer-trailer unit including towbars do not exceed 80 ft. The maximum length of semitrailer-semitrailer or semitrailer-trailer combination, excluding the length of the truck-tractor, is 81.5 ft. provided the maximum length of either unit does not exceed 45 ft. If the towbar length exceeds 19 ft., the towbar shall be flagged during day light hours and lighted at night. The weight of the second unit may not exceed the weight of the first unit by more than 3000 lb.
T	70 ft. overall limit if semitrailer is over 53 ft. on network (48 ft. on other roads) or twin trailers are over 28.5 ft.
U.	2,3 and 4-unit combination, 110 ft. on 4-lane divided highways.
V	80,000 lbs. or 57,650 lbs., depending on highway classification.
W	If axles of tandem are less than 6 ft. apart.
X	Two successive axles spaced 4 ft. or less, 24,000 lb.: axles spaced more than 4 ft. up to 10 ft., 34,000 lb. and 1,000 lb. for each foot or fraction thereof over 4 ft.
Y	As measured from front of the first trailing unit to rear of the second.
Z	When GVW is 73,280 lbs. or less, single axle may not exceed 22,400 lbs., and tandem: 36,000 lbs.: if GVW exceeds 73,280 lbs., single axle may not exceed 20,000 lbs., and tandem 34,000 lbs.
AA	Tractor-semitrailer combination 60 ft if semitrailer is 45 ft or less. Tractor-semitrailer combination 55 ft if semitrailer is greater than 45 ft. and less than 48 ft.
BB	If have 54 ft between first tractor axle and last trailer axle , plus overall length not over 60 ft.
CC	48 ft 1st semitrailer, 40 ft 2nd semitrailer, but combined length of the two may not exceed 80 ft. including connecting devices. Other combinations not shown ,85 ft.
DD	73,500 on some roads.
EE	If over 48 ft, kingpin to rear axle cannot exceed 41 ft. Tractor-twin-trailer combinations allowed on state designated routes only.
FF	Provided distance between kingpin and center of the rearmost axle group is 41 ft. or less.
GG	Combinations of trailers can be 61 ft including tongue, or 75 ft. overall.
HH	Kingpin to rearmost axle control exceed 40.5 ft.: if the semitrailer was manufactured before January 1, 1985, the kingpin to rearmost axle distant shall not exceed 42 ft. and 6 in. A semitrailer, regardless of when it was manufactured, that is longer than 48 ft. and 6 in. and that has a distance between the kingpin and rearmost axle of 43 ft. or less may be operated on the Interstate system and have 10 miles of access.
II	If GVW is below 71,000 lb., single axle weight may be 22,400 lb., tandem axle weight may be 36,000 lb.
JJ	Variable. Contact Michigan Department of Transportation.
KK	14 ft. on Interstate and designated system only. otherwise 13.5 ft.
LL	Measured from the point of attachment (kingpin) to end of trailer or load. If the semitrailer (or trailer) length limit exceeds 48 ft. the distance between the kingpin and the rearmost axle or a point midway the two rear axles, if the two rear axles are tandem axle, shall not exceed 41 ft.
MM	A 48 ft. trailer and 60 ft. overall length is also legal.
NN	53 ft. semitrailer must have maximum of 41 ft. from center of kingpin to center of rear tandem on trailer or center of rearmost axle in the case of a single axle or "stretch tandem" trailer: 67.5 ft. semitrailer combinations and twin trailer combinations, allowed on state designated system.
OO	41 ft. maximum from kingpin to center of rear axle assembly. If the semitrailer is longer than 48 ft., it must be equipped with a rear underside guard.
PP	If gross weight is more than 75,185 lb., legal tandem weight is 34,000 lb.
QQ	Semitrailer can only have 2 axles. Kingpin to center of tandem axle cannot exceed 40.5 ft. ± 0.5 ft.
RR	Eff. 4-1-91 decreased to 34,000 lbs.

Table 2.4 Total Fees Collected from Sales of Permits Since July, 1985 Through June, 1996 (Oklahoma Tax Commission 1996).

Fiscal Year[a]	Overweight (Dollars)	Oversize (Dollars)	Total (Dollars)	Total Permits (Oversight and Overweight)
1985-1986	2,487,166.00	866,650.00	3,353,816.00	150,379
1986-1987	2,067,174.00	1,162,942.00	3,230,116.00	120,357
1987-1988	2,307,115.00	1,159,350.00	3,466,465.00	120,185
1988-1989	2,223,390.00	1,103,095.00	3,326,485.00	114,297
1989-1990	2,528,345.00	1,114,225.00	3,642,570.00	115,377
1990-1991	2,906,615.00	1,162,925.00	4,069,540.00	120,825
1991-1992	2,496,995.00	1,043,700.00	3,540,695.00	108,273
1992-1993	2,965,180.00	1,087,425.00	4,052,605.00	112,144
1993-1994	5,099,720.00	2,004,070.00	7,103,790.00	113,180
1994-1995	4,301,670.00	2,302,560.00	6,604,230.00	117,694
1995-1996	3,631,545.00	2,436,060.00	6,067,605.00	124,681

Note:
[a] Fiscal year starts from July, 1 and ends to June, 30.

Table 2.5 International Truck Size and Weight Limit (AASHTO 1995)

Size/Weight	U.S	**CANADA**	**MEXICO**	EUROPE
Width	102 in	102.4 in	98.4 in	98.4 in
Height	13.5 ft	13.6 ft	13.6 ft	13.1 ft
Length				
Semi-Trailer	48.0 ft	48.0 ft	48.0 ft	39.4 ft
Full-Trailer	28.5 ft	province limits	27.0 ft	39.4 ft
Straight Truck	60 ft	province limits	40 ft	39.4 ft
Tractor-semi-trailer	varies by state from 60 ft	75.4 ft	55.8 ft	54.1 ft
Road Train (Truck-full trailer)	varies by state from 50 ft	------	62.3 ft	60.2 ft
Tractor-semi-trailer-semi-trailer	varies by state from 65 ft	75.4 ft	72.2 ft	-----
WEIGHT				
Single axle	20,000 lb	**20,056 lb**	**22,040 lb**	22,040 lb
Tandem axle	34,000 lb	**37,468 lb**	**39,672 lb**	39,672 lb
Tridem axle	42,000 lb	**48,576 lb**	**49,590 lb**	52,892 lb
GVW				
Tractor -semi-trailer(5 axle)	80,000 lb	**87,058 lb**	**91,508 lb**	88,160 lb
Tractor -Twin trailers(7 axle)	80,000 lb	**117,914 lb**	**135,608 lb**	-----

Table 2.6 Summary of Interstate Highway Pavements Needs for Oklahoma State (ODOT 1995a, pp. 25).

Dist.	Rural Interstate				Municipal Interstate			
	Adequate (miles)	Tolerable (miles)	Critical (miles)	AADT	Adequate (miles)	Tolerable (miles)	Critical (miles)	AADT
1	35.45	0.00	54.85	12,065	0.00	0.00	6.97	12,303
2	0.00	0.00	0.00	0	0.00	0.00	0.00	0
3	122.67	0.00	2.84	17,058	32.31	1.00	8.84	36,711
4	108.28	0.00	1.80	16,280	104.90	9.58	12.98	49,695
5	42.30	0.00	39.01	15,405	0.00	0.00	4.96	15,697
6	0.00	0.00	0.00	0	0.00	0.00	0.00	0
7	154.49	0.00	7.35	11,510	12.01	0.00	1.75	16,043
8	131.56	0.00	0.00	15,626	24.26	0.00	9.30	56,526
Total	594.75	0.00	105.85	14,550	173.48	10.58	44.80	44,407

Note: AADT: Annual Average Daily Traffic.

Table 2.7 Traffic Growth Factors[1] (AASHTO 1986, pp. D-23).

	Annual growth factor, percent (g)							
Analysis period (years)	No growth	2	4	5	6	7	8	10
1	1.0	1.0	1.00	1.00	1.00	1.00	1.00	1.00
2	2.0	2.02	2.04	2.05	2.06	2.07	2.08	2.10
3	3.0	3.06	3.12	3.15	3.18	3.21	3.25	3.31
4	4.0	4.12	4.25	4.31	4.37	4.44	4.51	4.64
5	5.0	5.20	5.42	5.53	5.64	5.75	5.87	6.11
6	6.0	6.31	6.63	6.80	6.98	7.15	7.34	7.72
7	7.0	7.43	7.90	8.14	8.39	8.65	8.92	9.49
8	8.0	8.58	9.21	9.55	9.90	10.26	10.64	11.44
9	9.0	9.75	10.58	11.03	11.49	11.98	12.49	13.58
10	10.0	10.95	12.01	12.58	13.18	13.82	14.49	15.94
11	11.0	12.17	13.49	14.21	14.97	15.78	16.65	18.53
12	12.0	13.41	15.03	15.92	16.87	17.89	18.98	21.38
13	13.0	14.68	16.63	17.71	18.88	20.14	21.50	24.52
14	14.0	15.97	18.29	19.61	21.01	22.55	24.21	27.97
15	15.0	17.29	20.02	21.58	23.28	25.13	27.15	31.77
16	16.0	18.64	21.82	23.66	25.67	27.89	30.32	35.95
17	17.0	20.01	23.70	25.84	28.21	30.84	33.75	40.55
18	18.0	21.41	25.65	28.13	30.91	34.00	37.45	45.60
19	19.0	22.84	27.67	30.54	33.76	37.38	41.45	51.16
20	20.0	24.30	29.78	33.06	36.79	41.00	45.76	57.28
25	25.0	32.03	41.65	47.73	54.86	63.25	73.11	98.35
30	30.0	40.57	56.08	66.44	79.06	94.46	113.28	164.49
35	35.0	49.99	73.65	90.32	111.43	138.24	172.32	271.02

Note:
[1]Factor = $((1+g)^n - 1)/g$, where g = rate/100 and is not zero. If annual growth rate is zero, the growth factor is equal to the analysis period.

Table 2.8 Recommended Levels of Reliability for Various Roadway (AASHTO 1993, pp. II-9).

Functional classification	Level of reliability (R)	
	Rural	Urban
Interstate and other freeways	88 - 99.9	85 - 99.9
Principle arterial roads	75 - 95	80 - 99
Collector roads	75 - 95	80 - 95
Local roads	50 - 80	50 - 80

Table 2.9 Minimum Thickness for AC Pavement (AASHTO 1986, pp. II-37)

Design traffic (ESAL)	Asphalt concrete thickness (inch)	Aggregate base thickness (inch)
<50,000	1.0	4
50,001 - 150,000	2.0	4
150,000 - 500,000	2.5	4
500,001 - 2,000,000	3.0	6
2,000,001-7,000,000	3.5	6
>7,000,000	4.0	6

Table 2.10 Minimum Design Wheel Load Considered in OSI Method for Different Roads (ODOT 1991)

Area	Functional Classification	Design ADT Range	Minimum Design Wheel Load (lb)[1]
Rural	Freeway	All	15,000
	Principle arterial	0-5000	12,000
		Over 5000	15,000
	Other arterial	0-2500	9,000
		Over 2500	12,000
	Collector	0 to 1200	7,000
		Over 1200	9,000
	Local roads	All	7,000
Suburban or Urban	Freeway	All	15,000
	Principle arterial	All	12,000
	Other arterial	All	9,000
	Collectors	All	9,000
	Local streets	All	7,000

Note:
[1] For facilities with heavy truck traffic (T3>25%), use a design wheel load of 15,000 pounds.

Table 2.11 Distress Manifestation for Flexible Pavements (FHWA, 1981*)

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associatied
Surface Defect	Abrasion	No	No	Aggregate	No
	Bleeding	No	Accentuated By High Temp	Bitumen	No
	Stripping	Yes	Yes	Both	Yes
	Ravelling	No	No	Aggregate	Slightly
	Weathering	No	Huminity And Light-Dried Bitumen	Bitumen	No
Surface Deformation	Corrugation Or Rippling	Slight	Climatic & Suction Relations	Unstable Mix	Yes
	Shoving	No		Unstable Mix Loss Of Bond	Yes
	Rutting	Excess in Granular Layer	Suction & Material	Compaction Properties	Yes
	Depression	Excess	Suction & Materials	Settlement, Fill Material	Yes
	Potholes	Excess	Frost Heave	Strength-Moisture	Yes
Cracking	Longitudinal	Yes	Spring-Thaw Strength Loss		Yes
	Alligator	Yes	Drainage	Possible Mix Problems	Yes
	Transverse	Yes	Low-Temp., F-T Cycles	Thermal Properties	No
	Slippage	Yes	No	Loss Of Bond	Yes

Table 2.12 Distress Manifestations for Rigid Pavements (FHWA, 1981*)

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associatied
Surface Defects	Spalling	Possible	No	Chemical Influence	No
	Scaling	Yes	F-T Cycling		No
	D-Cracking	Yes	F-T Cycling	Aggregate	No
	Crazing	No	No	Rich Mortar	No
Surface Deformation	Blow-Up	No	Temperature	Thermal Properties	No
	Pumping	Yes	Moisture	Fines In Base Moisture Sensitive	Yes
	Faulting	Yes	Moisture-Suction	Settlement Deformation	Yes
	Curling	Possible	Moisture And Temp.		No
Cracking	Corner	Yes	Yes	Follows Pumping	Yes
	Diagonal Transverse Longitudinal	Yes	Possible	Cracking Follows Moisture Builup	Yes
	Punch Out	Yes	Yes	Deformation Following Cracking	Yes
	Joint	Produces Damage Later	Possible	Proper Filler And Clean Joints	No

CHAPTER III

DETERIORATION OF FLEXIBLE PAVEMENT DUE TO HEAVY TRUCKS

3.1 INTRODUCTION

The deterioration of AC pavement can be classified under three modes of distress: cracking, permanent deformation, and disintegration. Truck traffic plays a major role in cracking resulting in fatigue damage and permanent deformation, e.g., rutting (Gillespie et al. 1993, pp. 15-16). Although truck wheel loads do not appear to have any initial influence on some types of disintegration, they certainly increase the rate of distress whose initial phase has occurred due to truck traffic characteristics, environmental effects, poor construction, inadequate design and other factors (Paterson 1987, pp. 219-221). Gradual or sudden failure of pavements may occur due to the occurrence of distress. To offset the deterioration, highway maintenance programs are mandatory. A well-planned maintenance and rehabilitation program decreases the rate of pavement deterioration. Newly constructed pavements with inadequate maintenance can deteriorate faster than normally expected. For this reason, at present, pavement engineers are focusing more on pavement management systems rather than new construction aspects. Selection of the pavement type and the design method are important features of pavement management systems. Considering the construction, operation, and maintenance aspects during the design life of pavement, life-cycle cost design is adopted. The design engineer finds an appropriate design that will serve the needs in terms of traffic volume and loads for a given level of service with the lowest cost.

This chapter describes the effect of truck load on fatigue, permanent structural deformation, and disintegration of AC pavements. It also explains the relative damage on AC pavements due to traffic and non-traffic related factors. Finally, an overview of the relative damage caused by heavy trucks and light vehicles is presented.

3.2 FATIGUE DAMAGE

Fatigue damage of AC pavements results from repeated loading on the surface of pavement. In accordance with the study performed by Gillespie et al. (1993, pp. 13), the longitudinal strain at the bottom of the asphalt surface is considered when estimating the fatigue damage in AC pavements. Fatigue damage expressed in terms of ESALs depends upon the truck traffic characteristics, such as axle loads, GVW, axle spacing, speed, and maneuvering. The simplest way to measure the fatigue damage due to a truck is to calculate its respective ESALs.

The fatigue damage of AC pavements is highly dependent on the truck axle load. Based on the previous studies, the fatigue damage is found to be proportional to axle load raised to the fourth power (Gillespie et al. 1993, pp. 13). Thus, a 20-kip single axle load is 10,000 times as damaging as a 2-kip single axle load. A truck with only 10% more single axle load (22 kips) than the limiting load (20 kips) causes approximately 50% more fatigue damage than a truck with the legal single axle load. Since the load in multiple axles is distributed over a number of axles, the fatigue damage of AC pavement is significantly lower in cases of tandem-axle or tridem-axle when compared with single-axle loads. Figure 3.1 illustrates that an 18-kip single axle load has almost the same relative fatigue damage (1 ESAL) potential as a 31-kip tandem axle or a 42-kip tridem axle.

The fatigue damage of AC pavement for selected axle groups was investigated by Southgate et al. (1983). The damage factor as a function of the total group load is presented in Fig. 3.2. This analysis was based on the strain energy approach for pavements subjected to heavy trucks. The load-damage factor relationship in this figure is based on Equation (3.1). The regression coefficients a, b, and c, used in this equation, depend upon axle type and configuration (Table 3.1).

$$\log10(DF) = \left[a + b(\log Load) + c(\log Load)^2\right] \tag{3.1}$$

where

DF = damage factor, and

a,b,c = regression coefficients.

It is noted that the GVW has not as much influence to the fatigue damage as the axle load. The fatigue damage is very high for a single unit truck with high GVW relative to other types of trucks. The fatigue damage is much lower in case of high GVW with sufficient number of axles which distribute the total weight to a greater number of axles. For example, a 3-axle Refuse Hauler weighing 64,000 lb causes more than twice (Fig. 3.3) as much fatigue damage as a 9-axle Turner Doubles weighing 114,000 lb with a 2-in. thick wear course. The steer axle and tandem axle loads of the former are 20,000 lb and 44,000 lb, respectively while those of the latter are only 10,000 lb and 26,000 lb, respectively. It is noted that the ESALs of a tandem axle are much smaller than those of the steer axles. Truck sizes, weights and additional characteristics used in Fig. 3.3 are presented in

Appendix B. Another important item to note is that the relative fatigue damage on thin pavements is more severe than on thick pavements.

3.3 DEFORMATION

Deformation of AC pavement is usually exhibited in the form of rutting, shoving, heave, small depression, etc. Among these, rutting is the most common type of truck load-associated deformation. The excessive tire pressure of truck traffic causes vertical strains in the subgrade. If the vertical strains are too high, plastic deformation occurs which results in rutting. The accumulated vertical strains have a direct link with the axle load. The vertical strain due to a 34-kip single axle load is about twice that caused by a 18-kip single axle load for thin pavements (Fig. 3.4). The vertical strain is nearly the same for a specific axle-load and not predominantly influenced by the axle configuration. Figure 3.4 shows that the vertical strains of thin pavements are more than the limiting value for all types of axles weighing over 20,000 lb. Severe rutting may occur when the axle loads are too high. A three-dimensional dynamic finite element analysis of AC pavement by Zaghloul and White (1993) shows that the rut depth of a 58-kip single axle load with dual wheels is approximately 100 times higher than that of an 18-kip single axle load with the same number of wheels (Fig. 3.5). Figure 3.6 illustrates that the permanent deformation for an 18-kip load is limited within the asphalt layer while only 5% of the permanent deformation for a 58-kip axle load develops in the viscoelastic asphalt surface, 10% in the base course, and the remaining in the elastoplastic subgrade layer. Therefore, only a few passes with a loaded truck carrying such heavy axle loads are enough to cause considerable rutting in an AC pavement. In different types of trucks, a predominant factor causing rutting is the GVW. The total rut depth (as a function of ESALs) caused by a passing truck is evaluated by integrating the rut depth caused by all axles. Figure 3.7 shows that a 9-axle Double weighing 140,000 lb causes rutting damage which is roughly four times greater than that causes by a 2- axle straight truck weighing only 32,000 lb.

3.4 DISINTEGRATION

The loss of surface materials affecting the structural and functional integrity of pavements is known as "disintegration." Disintegration includes raveling, polishing, edge break, etc. These types of distress are highly influenced by age, weather conditions, construction quality, and secondarily by traffic volume (Paterson 1987, pp. 331; OECD 1988, pp. 27). Raveling can be defined as the loss of stone from the surfacing either by mechanical fracture of the binder film or by loss of adhesion between binder and stone. Raveling is generally independent of the truck axle load magnitude but traffic flow has a significant and reasonable effect on the age of surface treatments at its initiation (TY_{rav}). The relationship between traffic volume and TY_{rav} is presented in Equation (3.2), as given by Paterson (1987) and from which it is apparent that "slurry seal" has higher durability in respect to raveling than the other surface types (Fig. 3.8). There is a decreasing rate of "time to raveling initiation" with an increasing rate of "traffic flow" or traffic volume.

$$TV_{rav}(sp) = K_{sp} \times a_{s} \; \exp(-0.655CQ - 0.156YAX) \qquad (3.2)$$

where,

$TY_{rav}(sp)$ = predicted age of surface treatments at the initiation of raveling, with probability of survival sp, in years ;

CQ = construction quality (0 if no faults, 1 if faulty);

YAX = annual flow of all vehicle axles, million/lane/year;

a_s = construction related to surface type (for chip seal: a_s= 10.5; for slurry seal: a_s=14.1; for cold-mix: a_s= 8.0); and

K_{sp} = factor depending on probability of survival, sp.

Potholes are distinguished from raveling as follows: "a pothole is a cavity in the road surface which is 6 inch or more in average diameter and 1 inch in depth." The volume of traffic is likely to affect the timing of potholing as shown in Fig. 3.9. Like raveling and potholing, other types of disintegration are independent of truck axle load as well, however, cumulative traffic load has an influence on their occurrence.

3.5 RELATIVE DAMAGING EFFECTS

The relative damage of pavements due to vehicular traffic versus non-traffic associated effects and the comparative degradation of pavements due to heavy truck traffic compared to other vehicular traffic are important issues in pricing and taxation efforts. Environmental effects are most evident in the freezing climates where a considerable acceleration of rutting, raveling, and potholing occur during thawing periods. Consequently, attributing the damage of pavements to traffic load is a complex task. To perform this task, roughness can be considered as primary damage because the major rehabilitation costs and all user costs are related to roughness, the same notion was also considered by Paterson (1987). The cumulative roughness damage usually results from three components: surface distress (S), structural deformation mechanisms (D), and age and environmental effects (E). The fraction of cumulative damage due to these factors varies with respect to the load intensity as well as pavement age. Figure 3.10 illustrates that the fraction of cumulative damage attributable to deformation mechanisms of an AC pavement, at initial stage, subjected to light loading (0.02 million ESAL/year) is only 20%. The fraction of damage attributable to deformation mechanism increases with the increase in load and is more than 80% in case of pavements carrying overloaded truck traffic (0.50 million ESAL/year).

The relative damaging effects due to heavy vehicle and light vehicle were also established by the 1962 AASHTO Road Test which demonstrated that the highway damage increases exponentially (fourth power) as the axle weight increases. The test established a relationship between damage caused by single loads and an equivalent number of automobiles (Fig. 3.11). Only a single pass of a 20,000-lb single axle load causes the same damage as that of more than 4,000 automobiles (Terrel and Bell 1987, pp. 32). It is also found that even though an 80,000 lb truck weighs only as much as 20 automobiles, it has the same damaging potential as 9,600 automobiles (GAO 1991, pp. 24).

According to Paterson (1987) the load-damage power varies from 0 to 6. Specifically, for fatigue it varies from 2 to 6 but an average value of 4 is appropriate; for rutting and disintegration the corresponding values are 2 and 0, respectively. The ratio of heavy vehicle to light vehicle damage for various load-damage powers (0 to 6) and several axle load spectrum types is shown in Table 3.2. This study was based on data from the Brazil-UNDP study and the Tunisia study. Three types of axle load spectrum (i.e., three different percentages of heavy loaded traffic within mixed traffic) were considered in the Brazil-UNDP study. The percentage of damage due to heavy vehicles is presented in Column 7 and the ratio of damage caused by heavy vehicles and by light vehicles is presented in Column 8 in Table 3.2. From Column 7 it is apparent that heavy vehicles are responsible for over 99% of the total damage if the damage power is considered to be 4, as in the Brazil-UNDP study. The Tunisia study, although, suggests 97% of the damage is due to heavy vehicles. This difference occurs because the light vehicles included in the Tunisia study were those having a GVW up to 13,200 lb, while in the Brazil-UNDP study the GVWs were less than 7,700 lb.

3.6 INCREASING LOAD EFFECTS ON PAVEMENT LIFE

For AC pavements, the pavement wear due to a given load increment increases significantly as the total ESAL increases. According to the 1991 GAO report "FHWA estimated that the 1974 amendments' weight provisions could reduce a highway's life from 20 years to 16.4 years, even accounting for reduced number of trips a larger truck would need to make." The existing theoretical and empirical models show that most forms of pavement distress are related exponentially (fourth power) to axle load. As an example, a pavement of 16-year design life with a 10-kip single axle load is expected to serve for only 2 years with a 20-kip single axle load. Considering GVW, a 10% increase in the total weight of a 60,000-lb tractor semi-trailer increases the effect on pavement wear by over 40 percent. So, if the over loaded trucks are allowed to move on the Interstate and state highways, they are likely to decrease the pavement life quite significantly. Furthermore, this damaging effect on pavement life is expected to be highly exponential (Fig. 3.12).

The axle loads of heavy trucks reduce the design fatigue life and decrease the life span of pavement against rutting. AC pavement with heavy trucks has a shorter service life than those with an equivalent number of benchmark trucks (based on federal maximum weight limits for international highway systems) carrying the same freight (Lee and Peckham 1990, pp. 164). To assess the impact of heavy vehicles on AC pavements, representable segments of Interstate highways of two states (Maine and Rhode Island) were considered in this study. Table 3.2 indicates that there are 46,956 and 315,042 heavy trucks on the travel lane annually at the Maine and Rhode Island sites, respectively. To represent the legal fleet more trucks were added to the original heavy truck fleet number, making the number of equivalent legalized trucks as 53,073 and 418,788, respectively, at these sites. The design fatigue life was found to decrease by about 20% at the Maine site and by about 32% at the Rhode Island site for heavy loaded trucks. From the deformation consideration, the design lives decreased about 15% and 32% at the respective sites.

Another deteriorating factor, namely, environment also influences pavement life. In reality, the interaction of heavy trucks in combination with environmental conditions appear to have the greatest influence on actual pavement life.

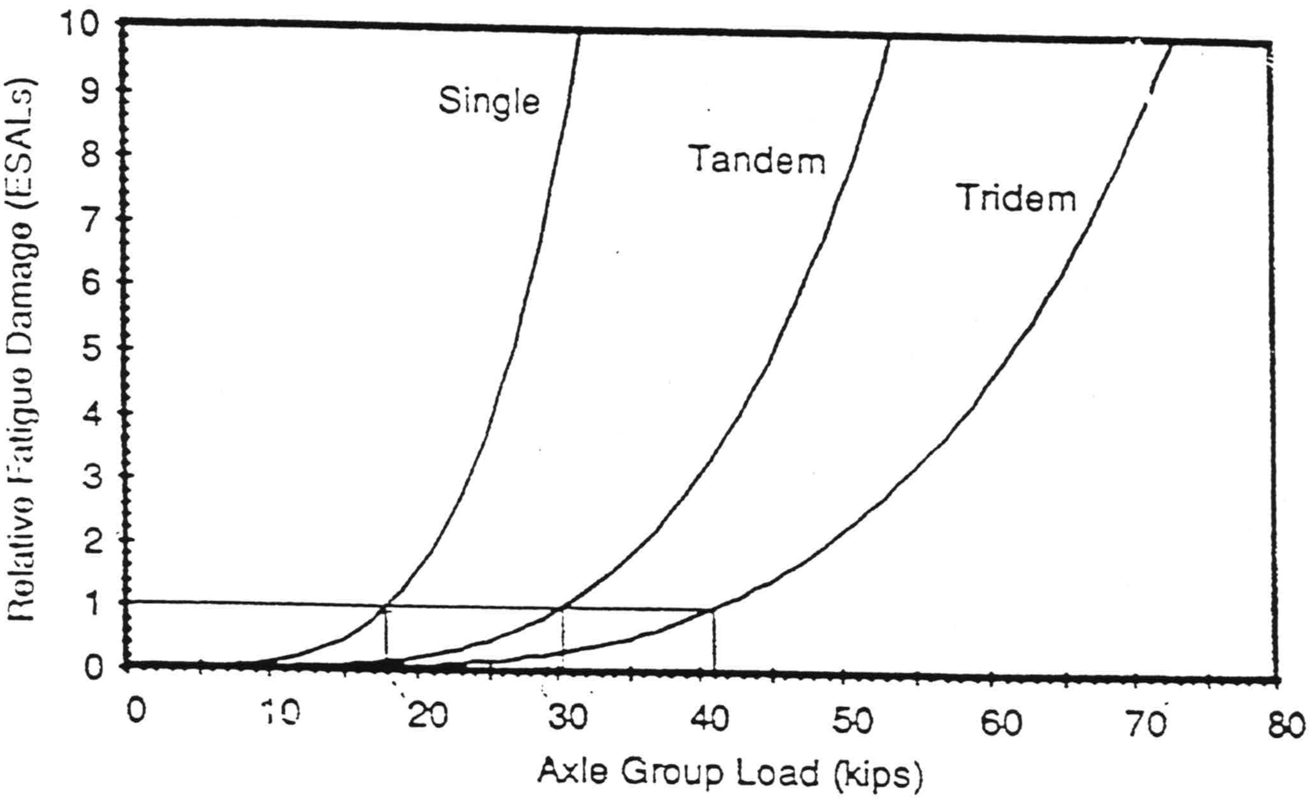


Fig. 3.1 Effect of axle load on fatigue damage of AC pavements (Gillespie et al. 1993, pp. 13).

Note: Surface layer thickness = 5 inch.

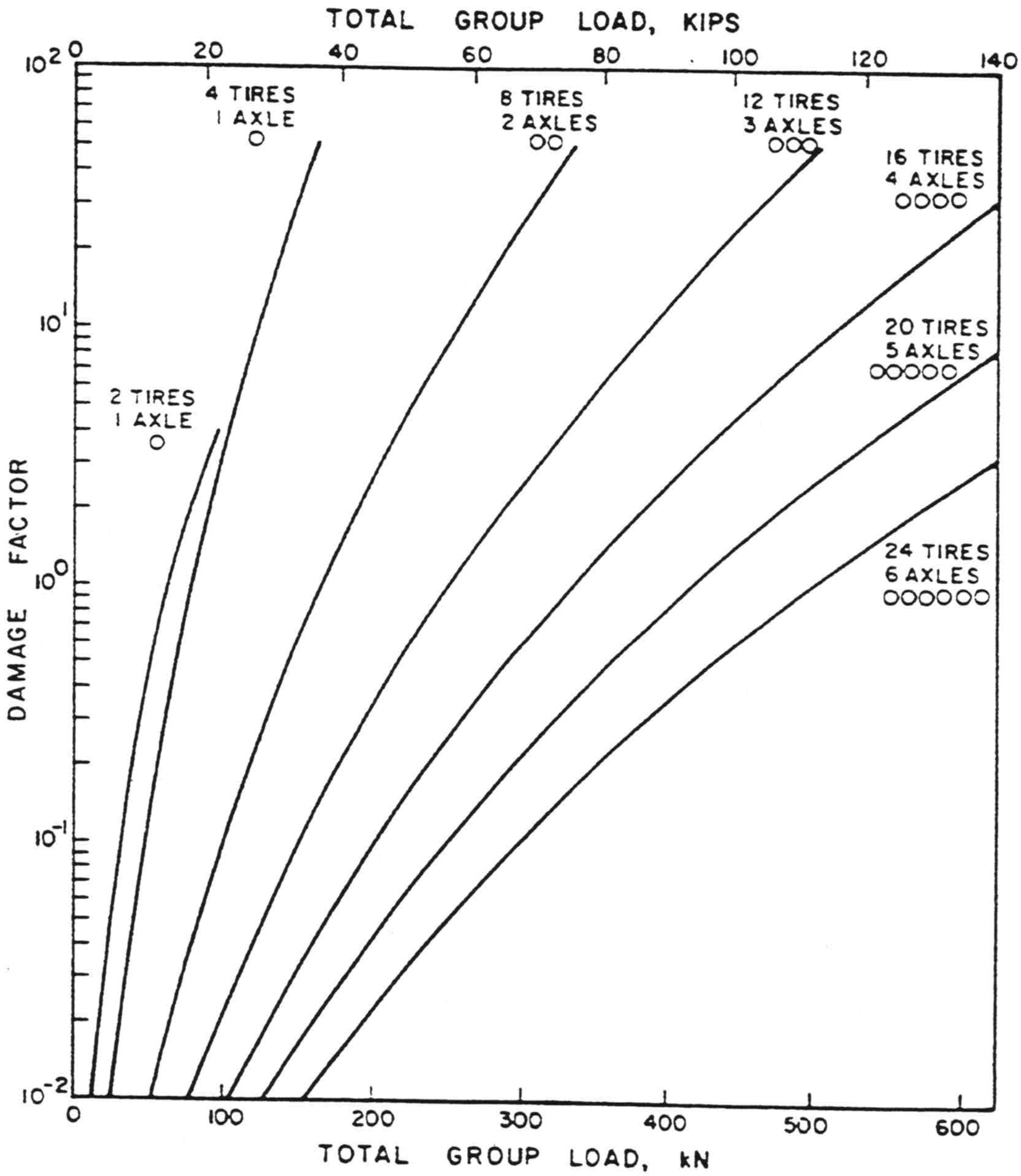


Fig. 3.2 Damage factor for selected axle groups on the basis of strain energy (southgate et al. 1983, pp. 13).

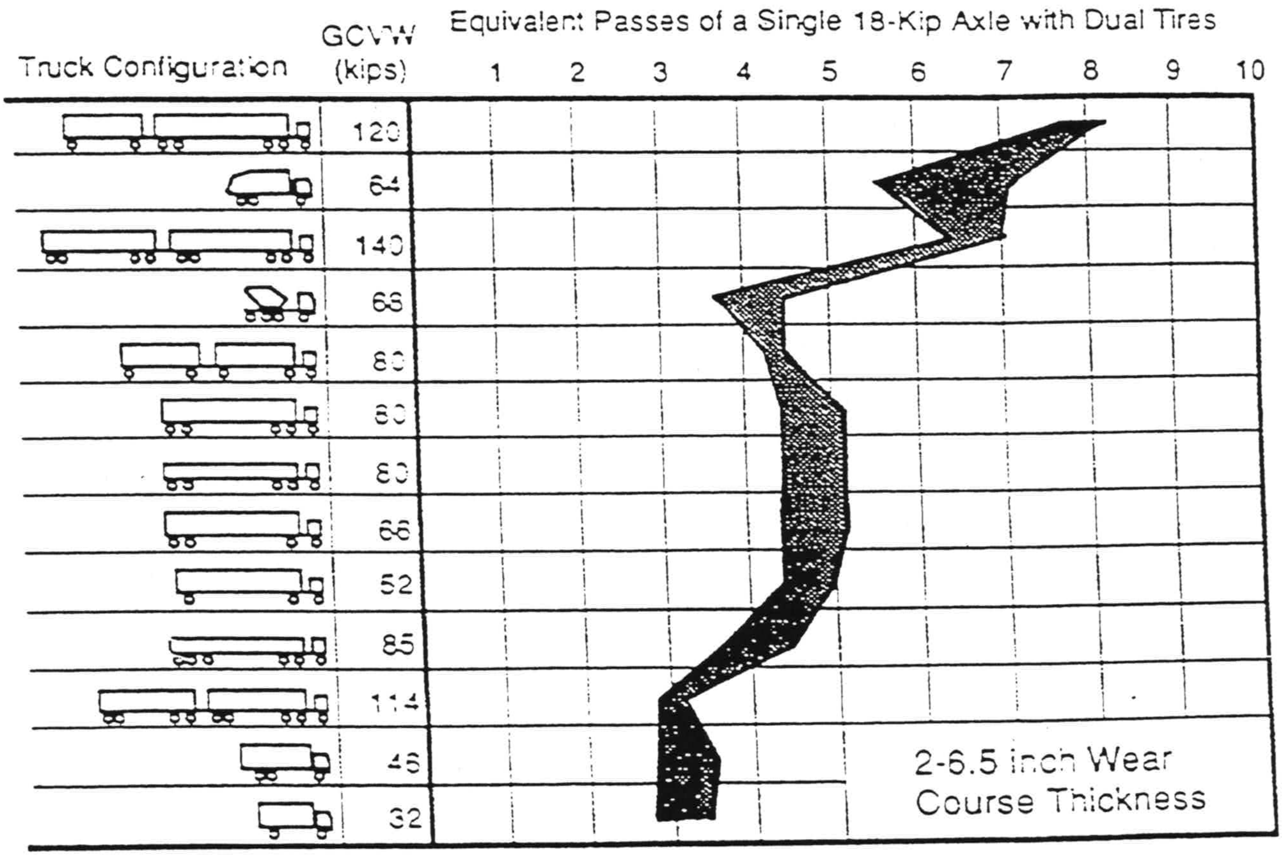


Fig. 3.3 Relative fatigue damage of typical trucks on AC pavements (Gillespie et al. 1993, pp. 15).

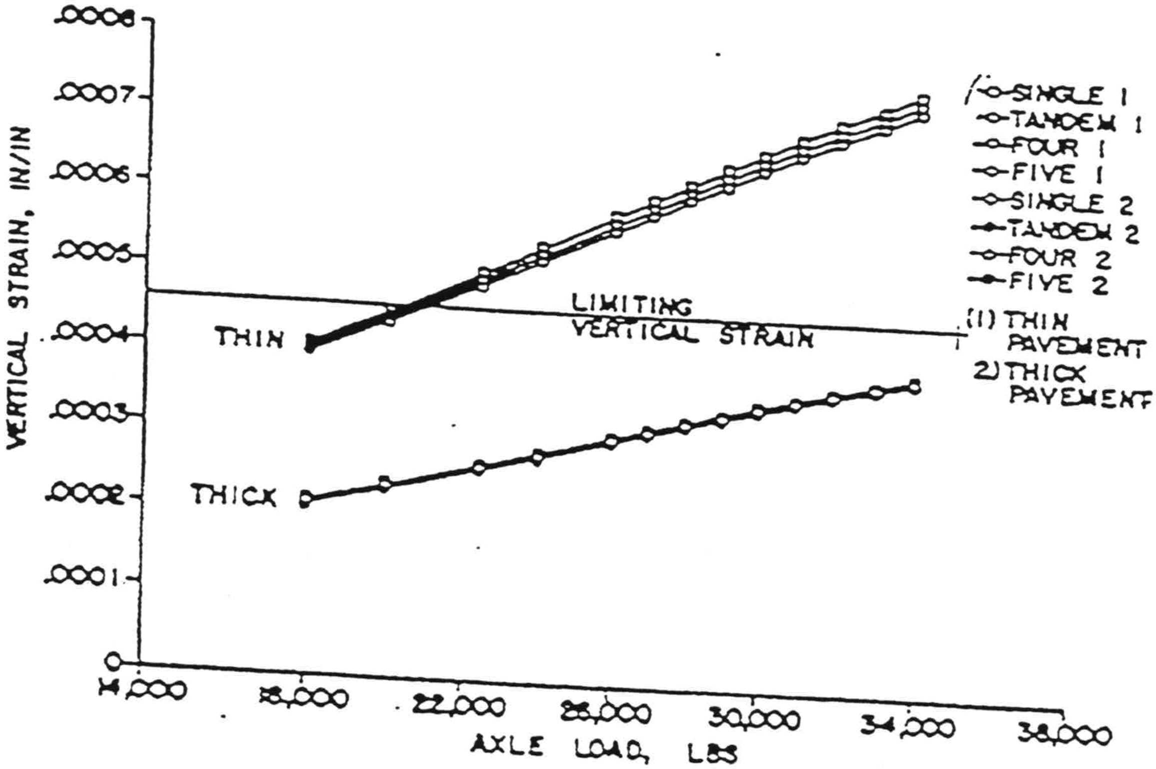


Fig. 3.4 Vertical strain at the top of the subgrade of thin (SN = 2.92) and thick (SN = 4.82) pavements (Kilareski 1989, pp.201).

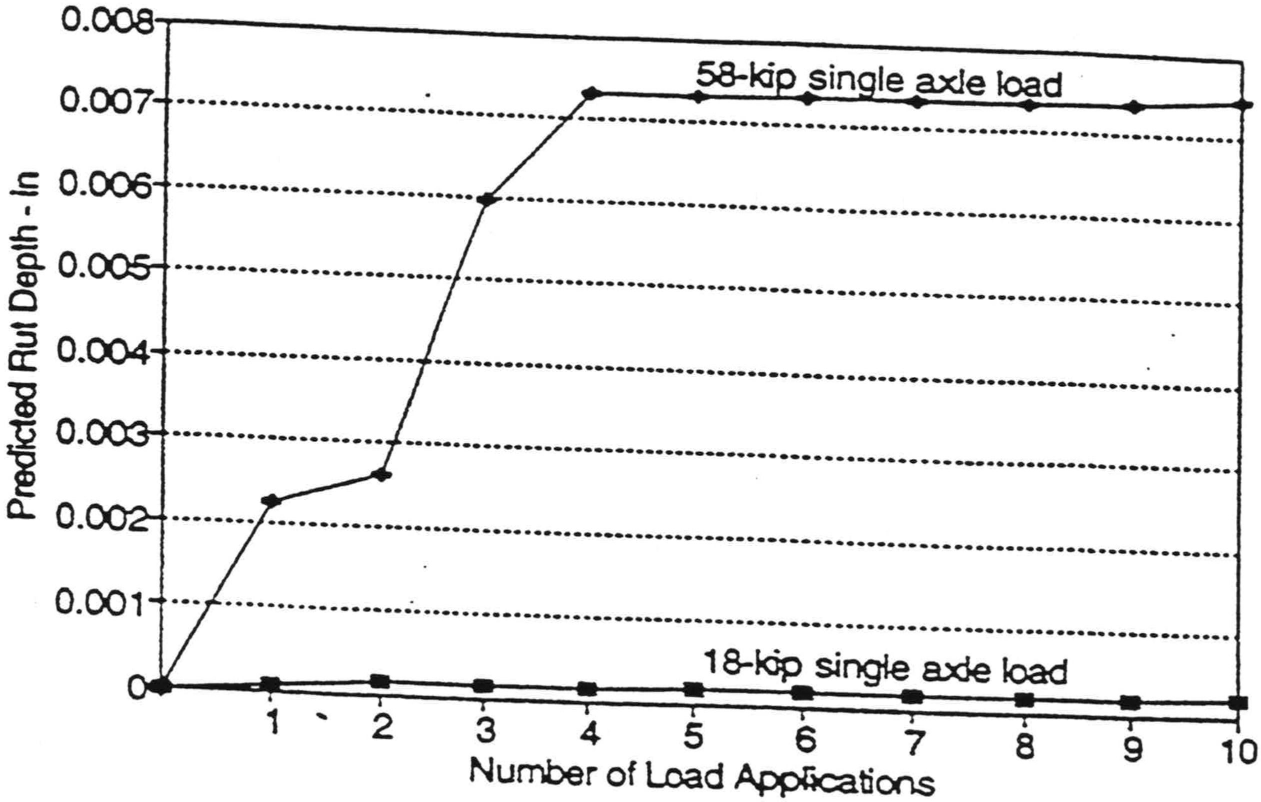


Fig. 3.5 Effect of axle loads on pavement rutting (Zaghloul et al. 1993, pp.60).

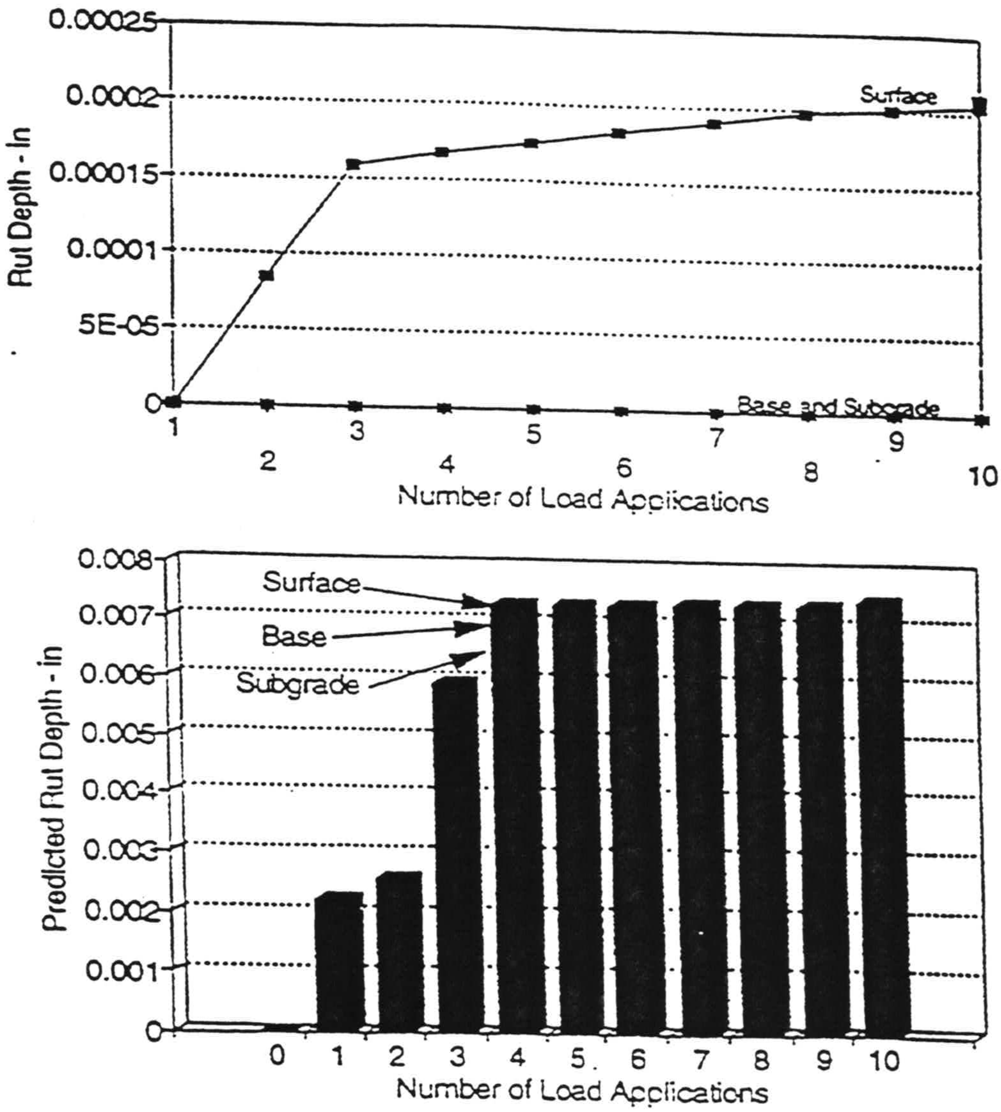


Fig. 3.6 Predicted pavement rutting: top, 18-kip single-axle load; bottom, 58-kip single-axle load (Zaghloul et al. 1993, pp.60).

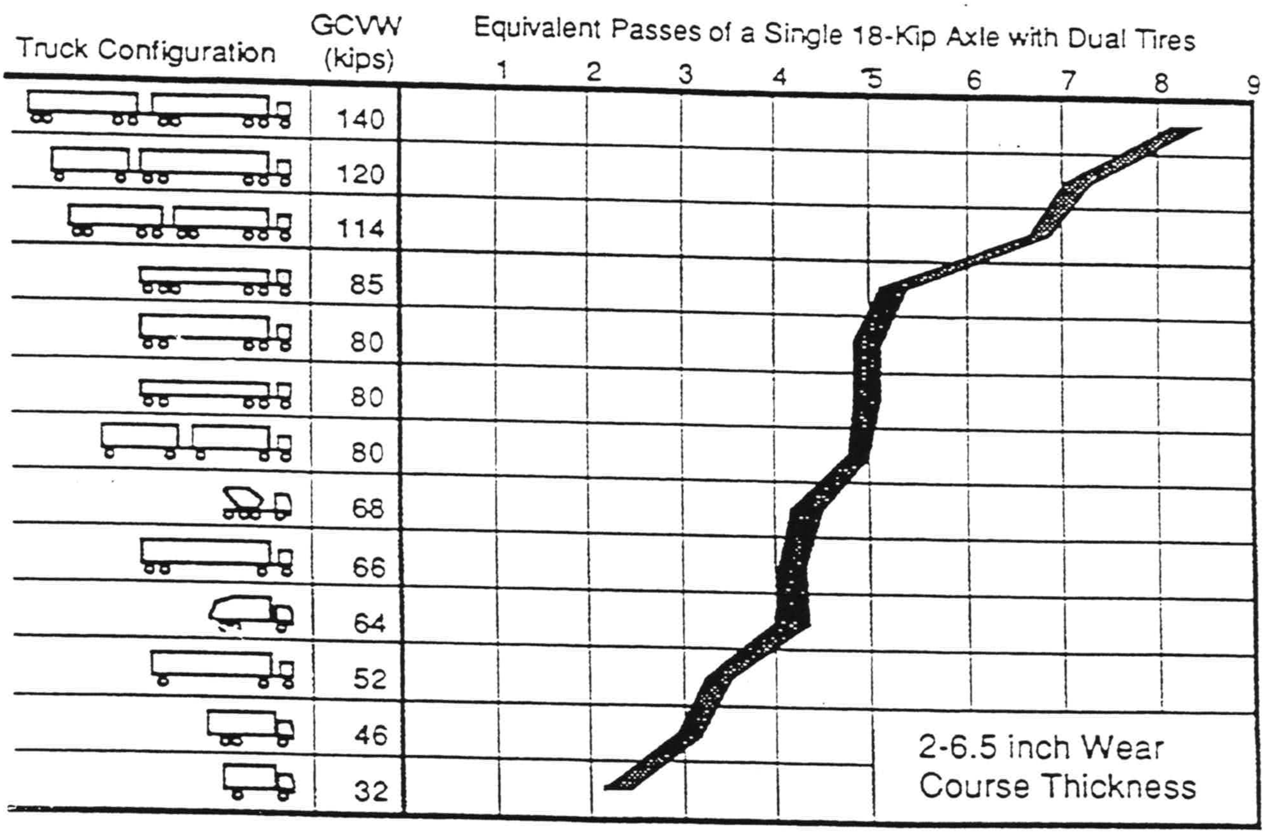


Fig. 3.7 Rut depth production expressed as ESAL exposure per pass deriving over a range of trucks and pavement wear course thickness (Gillespie et al. 1993, pp. 16)

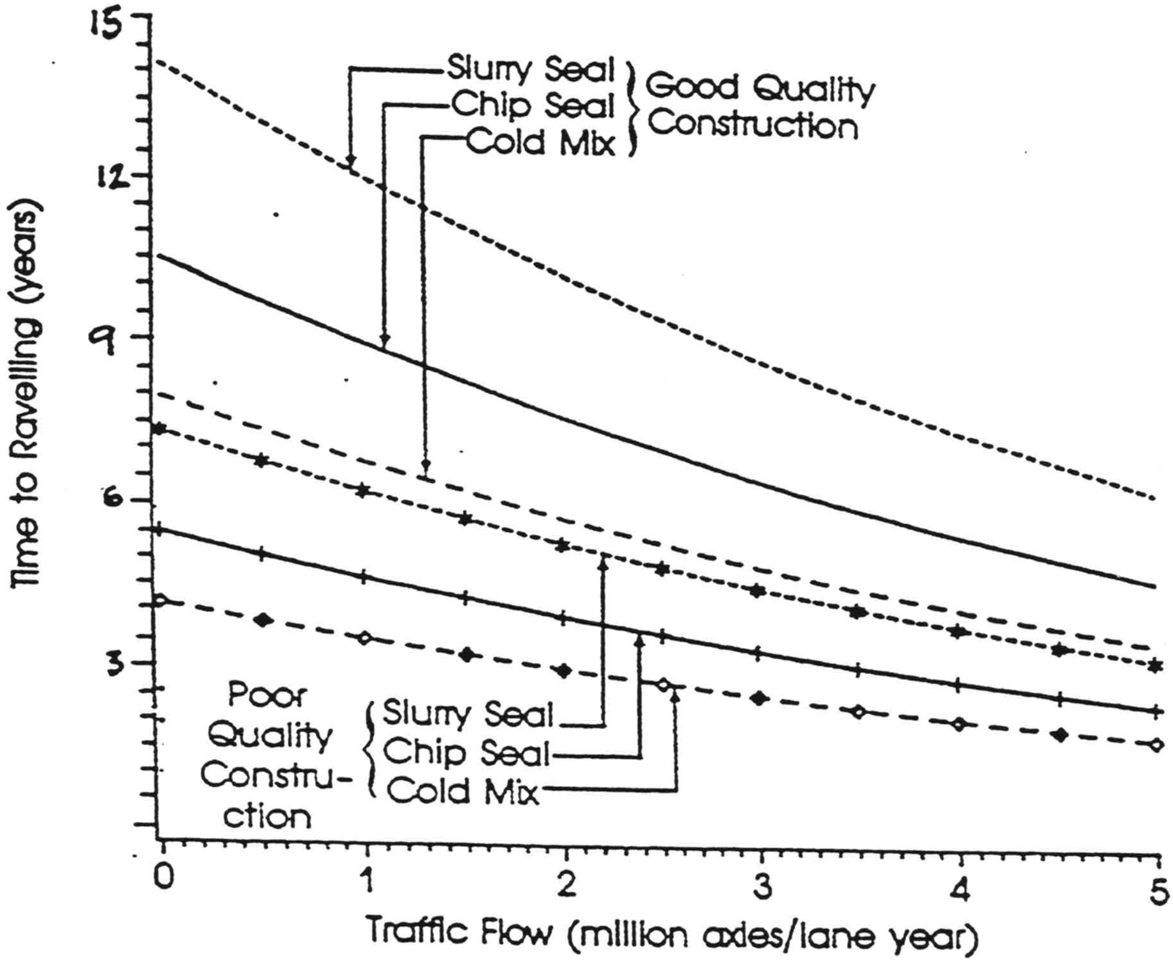


Fig. 3.8 Time of raveling initiation of various surface treatment types (Paterson 1987. pp.226).

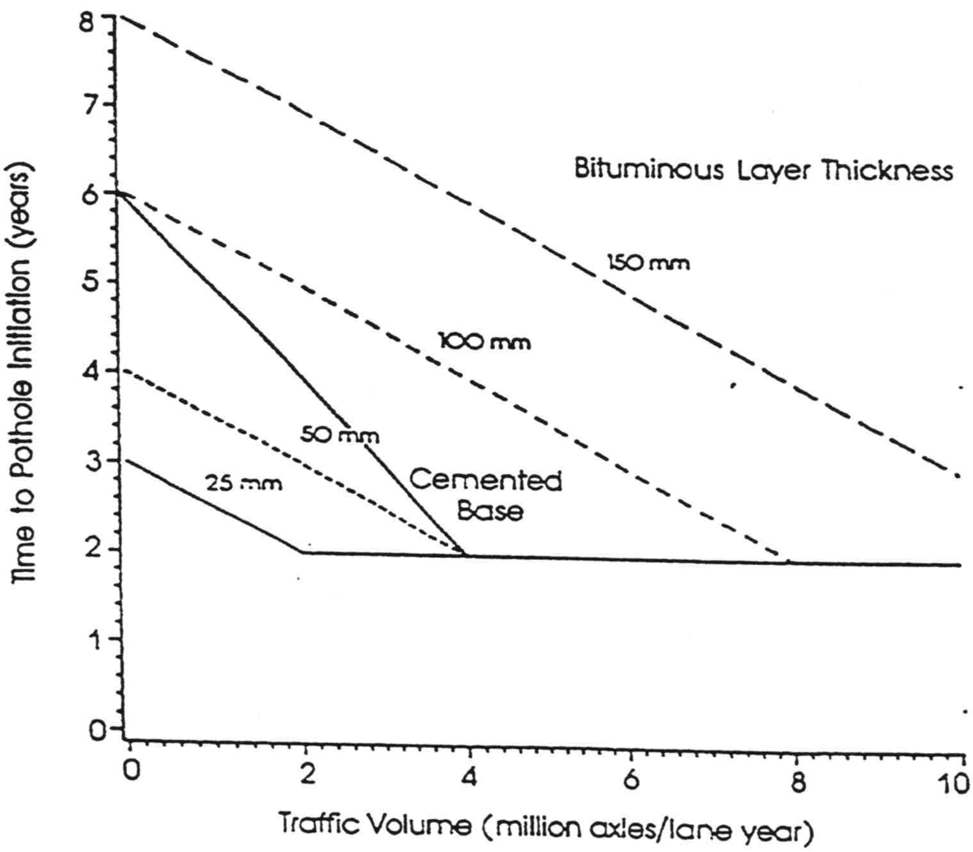
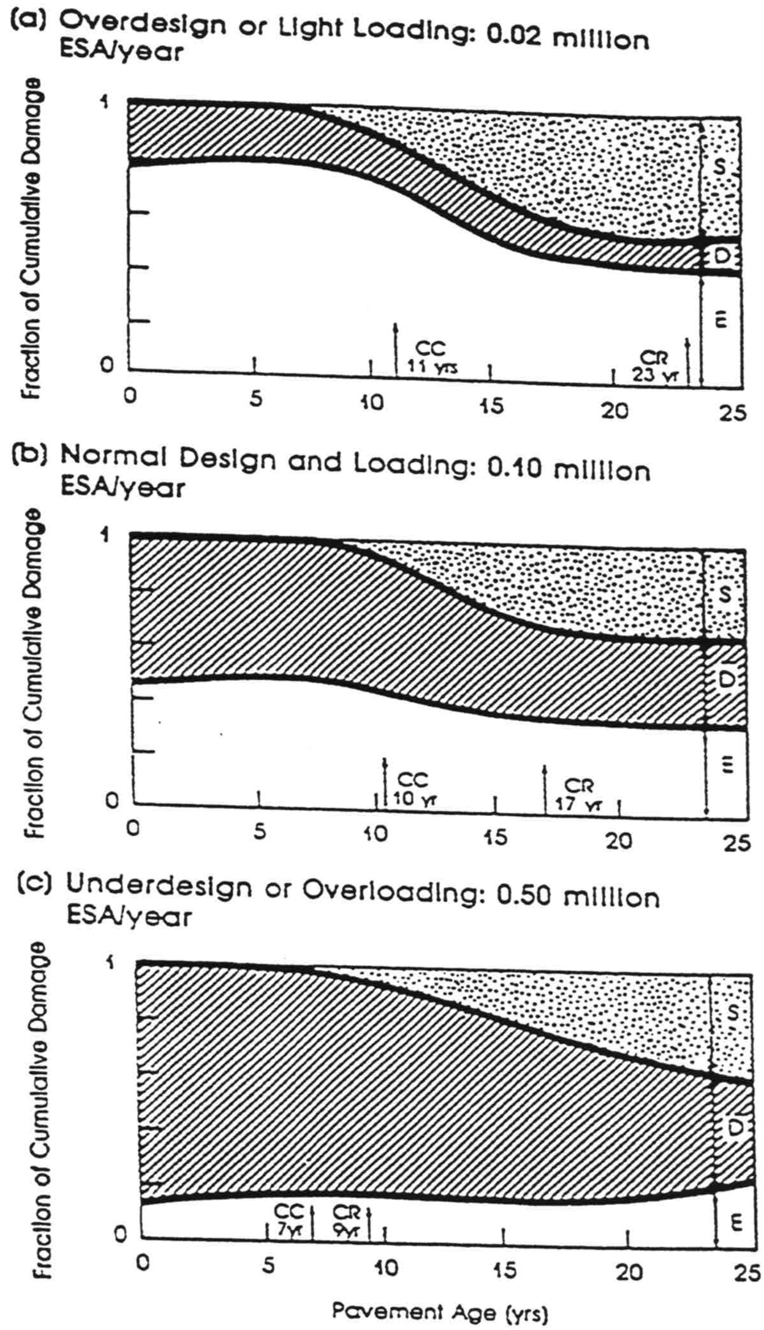


Fig. 3.9 Predicted minimum time between wide cracking and potholing initiations (Paterson 1987, pp.235).



Notes: Pavement characteristics - Modified Structural Number 3, Asphalt concrete surfacing

Code:
CC = critical cracking 30 percent
CR = critical roughness of 5 m/km IRI (after pothole patching)
E = roughness attributable to age and environmental effects
D = roughness attributable to deformation mechanisms
S = roughness attributable to surfacing distress

Source: Author's computation using Road Deterioration and Maintenance Model of HDM-III.

Fig. 3.10 Influence of traffic loading on attribution of roughness damage for constant pavement strength (Paterson 1987. pp.361).

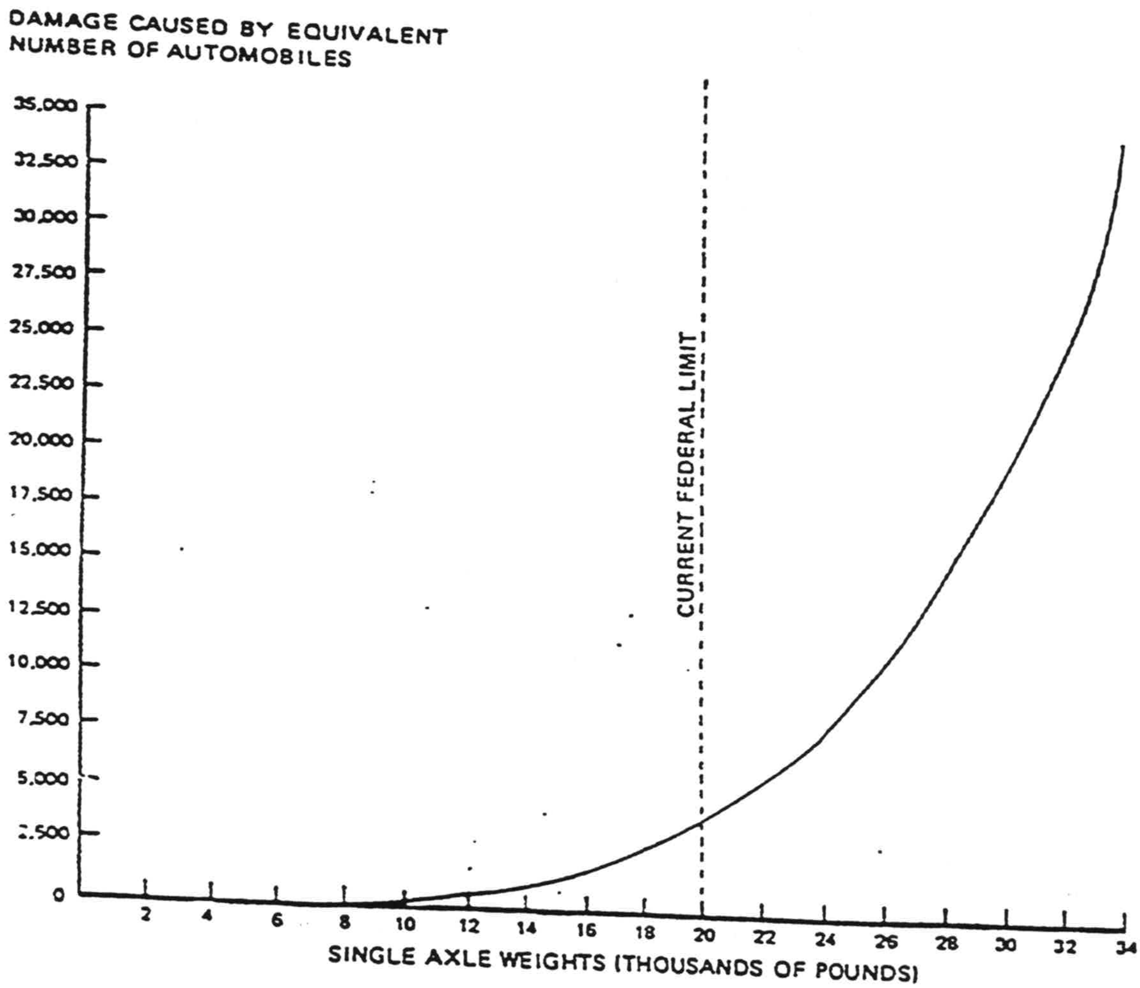


Fig. 3.11 Relative damage caused by increase in single axle weights above federal limits (Terrel and Bell 1987, pp. 32).

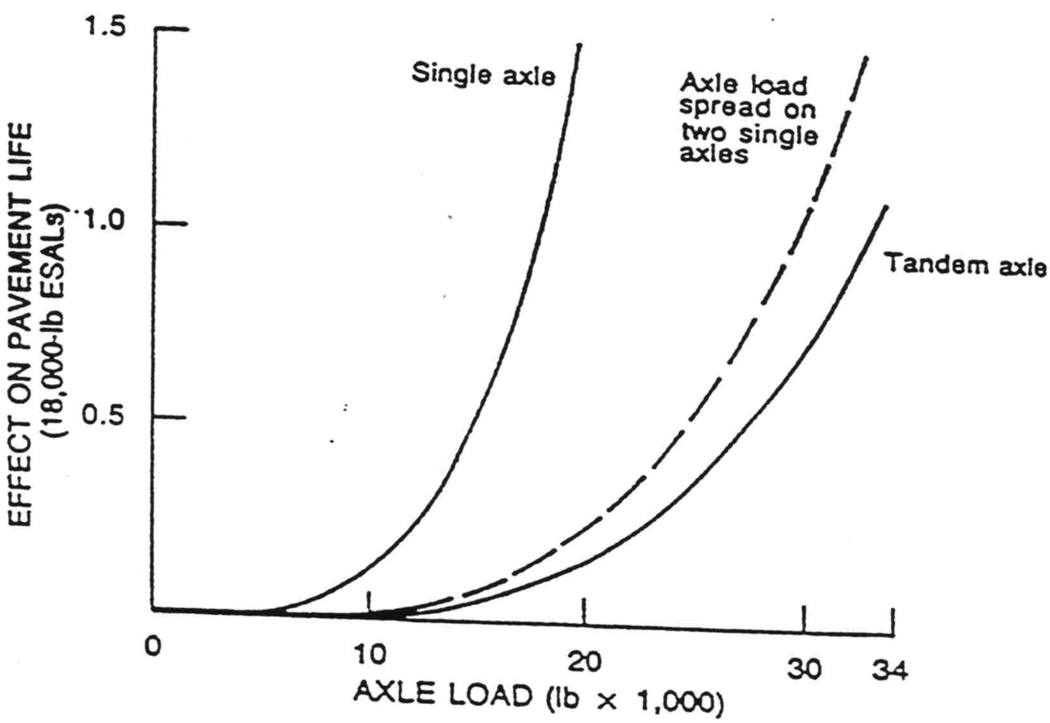


Fig. 3.12 Effect of axle load on pavement life (TRB 1986, pp. 162).

Note: Equivalency factors based on SN = 3 and $p_t = 2.5$.

Table 3.1 Regression Coefficients to Calculate Damage Factors for Various Axle Configurations (Southgate et al. 1983, pp. 13)

Log (Damage Factor) = a + b (Log Load) + c (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	-1.973428	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 3.2 Traffic Composition on Major Highway Systems of Oklahoma (ODOT 1995b)

Highway and Location	Traffic volume	% of trucks in WADT
I-35 (Blackwell)	12,886 (1995 AADT)	23.5 (June 1-7)
I-40 (Pharoah)	14,510 (1995 AADT)	23.6 (June 1-7)
I-35 (Edmond)	16,036 (June MADT)	13.2 (June 1-7)
I-44 (Lawton)	13,941 (1995 AADT)	9.3 (July 9-15)
US-69 (Mazie)	10,083 (June MADT)	30 (June 1-7)
US-270 (Watonga)	3,288 (1995 AADT)	22.5 (Feb 8-14)
US-70 (Admore)	4445 (1995 AADT)	11 (June 8-14)
I-35 (Davis)	20,271 (June MADT)	19.8 (June 1-7)
SH-112 (Potoeau)	6,681 (1995 AADT)	8.5 (June 1-7)
SH-266 (Tulsa)	44,201 (1995 AADT)	14.8 (June1-7)

Note: AADT: Annual Average Daily Traffic; MADT: Monthly Average Daily Traffic; WADT: Weekly Average Daily Traffic.

Table 3.2 Relative Damaging Effects of Heavy Vehicle to Light Vehicles for Major Load Distribution Types and Varying Load Damage Power (Paterson 1987, pp. 369).

Load damage power	Average ESA/veh		Inci-dence heavy % ADT	ESA/veh	Spectrum power ratio ESA_n / ESA_4	Alloca-tion to all heavy vehicles %	Ratio of damage heavy: light vehicles
	all heavy vehicles	all light [1]/ vehicles		All Vehicles			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Brazil-type A							
0	2.012	2.000	25.1	2.009	15.46	25.1	1
2	0.722	0.020	25.1	0.196	1.51	92.4	36
4	0.516	0.0002	25.1	0.130	1.00	**99.9**	**2580**
6	0.609	0.0000	25.1	0.153	1.18	100.	99000
Brazil-Type B							
0	2.191	2.000	24.0	2.046	8.09	25.7	1.1
2	0.979	0.020	24.0	0.250	0.99	93.9	49
4	1.053	0.0002	24.0	0.253	1.00	**99.9**	**5260**
6	1.560	0.0000	24.0	0.374	1.48	100.	99000
Brazil-Type C							
0	2.422	2.000	29.6	2.125	1.73	33.8	1.2
2	2.485	0.020	29.6	0.750	0.61	98.1	124
4	4.144	0.0002	29.6	1.227	1.00	**100.**	**20700**
6	8.215	0.0000	29.6	2.433	1.98	100.	99000
Tunisia							
2	1.57	0.076	16.6	0.324	0.86	80.4	21
3	1.73	0.030	16.6	0.312	0.83	92.0	58
4	2.19	0.014	16.6	0.375	1.00	**96.9**	**156**
5	3.09	0.007	16.6	0.519	1.38	98.8	441
6	4.63	0.003	16.6	0.771	2.06	99.7	1540

1/ Light vehicles in Tunisia included those of gross vehicle Weight (GVW) of up to 13,200 lb. In Brazil, light vehicles were defined as those with GVW of less than 7,700 lb.

Table: 3.3 Effect of heavy loaded truck on pavement life (Lee et al. 1990, pp.164))

Site	The Maine site		The Rhode Island site	
Truck type	Heavy trucks	Legalized truck	Heavy trucks	Legalized truck
No of trucks (annual)	46,956	53,073	315,042	418,788
Damage				
Fatigue	0.1363E+00	0.1156E+00	0.2711E00	0.1841E+00
Deformation	0.4649E-02	0.3942-02	0.5493E-01	0.3731E-01
Design life				
Fatigue	7.3 yrs	8.7 yrs	3.6 yrs	5.3 yrs
Deformation	215 yrs	254 yrs	18.2 yrs	26.8 yrs

Note: ESALs determined by AASHTO 1986 guide

CHAPTER IV

DETERIORATION OF RIGID PAVEMENT DUE TO HEAVY TRUCKS

4.1 INTRODUCTION

Many factors affect pavement performance including load, environment, construction quality, and maintenance. One of the most important factors affecting the deterioration of rigid (PCC) pavement is heavy truck loading (Backlund et al. 1990, pp. 114). Compared to truck traffic, loads imposed by automobiles and other light vehicles are negligible (Paterson 1987, pp. 369). According to Gillespie et al. (1987, pp. 21), the axle load of a loaded truck contributes significantly to forms of distress in PCC pavement: (a) fatigue damage and (b) permanent deformation. Fatigue damage is directly related to the axle load and the number of repetitions. Due to the repeated action of loaded trucks, tension cracks develop at the bottom of a concrete slab and propagate upward. Deformation of PCC pavement at the joints, edges, and corners of a supported slab is associated with traffic load. Repeated application of loaded trucks also cause joint faulting and pumping leading to the removal of fines which results in the loss of support. Surface distress is usually initiated by environmental factors and/or construction faults but the movement of heavy trucks increases the rate of such deterioration. As a result, the service of pavements can be significantly shortened and increased maintenance works are necessary.

In this chapter, the effect of increasing axle load of loaded trucks on fatigue and deformation distress of PCC pavement is discussed. Finally, the effect of increasing load on the service life of PCC pavement is explained.

4.2 FATIGUE DAMAGE

Heavy wheel loads imposed by loaded trucks are believed to be one of the most significant causes of fatigue damage, which results from repeated tensile stresses occurring at the bottom of a concrete slab. A comprehensive numerical study by Zaghloul et al. (1994a, pp. 50) showed that fatigue cracks develop beneath the PCC pavement slabs and propagate upward if the ratio of the stress induced by load to the modulus of rupture (concrete property) exceeds 0.5. Generally, the situation is manifested by longitudinal cracks and/or transverse cracks. In a recent NCHRP study by Gillespie et al. (1993, pp. 12), it was reported that this damage is proportional to the axle load and proportional to the load raised to the fourth power. The variation of fatigue damage (in terms of ESALs) with the axle load for different axle types (single, tandem, and tridem) is shown in Fig. 4.1. When the current axle load of 20,000 lb is increased by 10% to 22,000 lb. the resulting fatigue damage increases by 46%. The total damage due to a particular truck can be obtained by summing the fatigue damages incurred by all groups of axles present in a truck. The relative fatigue damage caused by major truck types is shown in Fig. 4.2. Note that the GVW is not directly related to fatigue damage. For example, a 3-axle Refuse Hauler weighing 64 kips causes about 3.5 times as much fatigue as a 9-axle Turner Doubles weighing 114 kips even though the GVW of the latter is 75% greater.

4.3 DEFORMATION AND PUMPING

Deformation of PCC pavements is usually manifested by joint faulting, pumping, blow-ups, curling, etc. Recent studies show that the differential vertical displacement of joints, identified as joint faulting, is highly influenced by axle loads of heavy trucks (Mcghee

1995, pp. 28; Ksaibati et al. 1994, pp. 1). As shown in Fig. 4.4, traffic load is the most critical factor affecting transverse joint faulting in PCC pavements (Darter et al. 1991, pp. 145). Transverse joint faulting is a major distress type responsible for the loss of serviceability in a jointed concrete pavement. The existence of free water beneath the slab increases the rate of joint faulting. As shown in Fig. 4.3, the approaching wheels of a loaded truck depress the approach side of the joint and force free water and suspended materials to move slowly towards the bottom of the leave slab. When truck wheels cross the joint, there is a sudden rebound of the approach slab followed instantaneously by a rapid depression of the leave slab. Consequently, the water and suspended materials are forced back under the approach slab at a high velocity and some of the solids are deposited under the approach slab causing it to gradually rise as repetitive wheel loads continue. After numerous cycles, the deposited materials cause a permanent rise of the approach slab. In addition, a permanent depression of leave slab occurs exacerbating the problem. Considering ESALs, slab thickness, and the drainage factor, the amount of faulting in an undoweled jointed PCC pavement can be estimated by the following as proposed by Ksaibati and Staigle (1995, pp. 4):

$$F=3.49+3.62E-07 \times ESAL-0.0107 \times D-0.324 \times D_f \qquad (4.1)$$

where

$F =$ predicted faulting (mm),

$ESAL =$ cumulative 18-kip equivalent single-axle load application,

$D =$ pavement thickness (mm), and

$D_f =$ numerical indicator of drainage provided (e.g., 0, if no edge drains exist, and 1, if edge drains exist).

Pumping is another load-associated PCC pavement distress where fine materials from subbase and subgrade often go into suspension and are expelled with water through faulted joints or corners under the action of repeated wheel loads. The upper surface of the slab expands with an increase in temperature, and corners as well as edges are forced into contact. When loaded trucks pass over such pavements, corners punch into the base materials. Due to repeated action, supporting materials are crushed and forced to move laterally. As a result, a large void is formed beneath the slab. When water invades the slab, fine particles are pumped out rapidly. Corner breaks are the result of excessive pumping in which the supporting materials are fully removed such that wheel loads can no longer be supported.

4.4 SURFACE DEFECTS

PCC pavement surface defects are classified as spalling, scaling, D-cracking, crazing, etc. Due to temperature changes or weather action these distresses appear on road surfaces. When loaded trucks are driven on a distressed roadway, the damaging rate increases rapidly, causing a structural failure of the pavement. As an example, D-cracking is not caused by truck loads, but it eventually results in severe deterioration of the PCC pavements. A study performed by Hall et al. (1991) on the Illinois Interstate highway system found that severe D-cracking of the 8-in. CRCP on the I-70, in combination with high volume of repeated heavy loads (10 million ESALs since 1980), caused the concrete to completely disintegrate in some locations. The ESAL survival curves for both thin (maximum thickness 3.25 in.) and thick (maximum thickness 6.0 in.) AC overlays of D-cracked and non-D-cracked JRCP as shown in Fig. 4.5. The mean life (in terms of ESALs) for thin AC overlays of non-D-cracked JRCP is 2.9 times more than that for thin AC overlays of D-cracked JRCP (18.4 versus 6.3 million ESALs). On the other hand, the mean life for thick AC overlays of non-D-cracked JRCP is 3.1 times more than that for thick AC overlays of D-cracked JRCP (45.4 versus 14.7 million ESALs).

4.5 EFFECT OF INCREASING LOAD ON PAVEMENT LIFE

PCC pavements are designed to accommodate a projected number of repetitions of a specified load for the projected service life. As mentioned earlier, according to the 1986 AASHTO design procedures, to calculate the design load the projected traffic load for the performance period is converted into ESALs. These ESALs are the basis for determining the thickness of the concrete slab required to provide the desired design life. Figure 4.6 shows the relationship between axle load and pavement life is a power function. The effect of a single axle on PCC pavement life increases as approximately a fourth-power function of axle load (TRB 1986, pp. 161-163). For example, a 36,000-lb single axle load is only two times larger than a 18,000-lb single axle load, but the former causes 17 times more loss in pavement life. Other design procedures, such as those proposed by Portland Cement Association (PCA) and other empirical and theoretical pavement design procedures used in other parts of the world, also indicate that the effect on pavement life increases as a power function of axle load (TRB 1986, pp. 162). Thus, if a truck heavier than the design one is permitted on the roadway, it would significantly reduce the service life of existing pavements; heavier axle loads would result in more rapid ESAL accumulations and shorten the interval of pavement resurfacing needed to maintain a desired level of serviceability.

Moreover, at the time of resurfacing, an increased overlay thickness would be required otherwise subsequent resurfacing intervals would be shortened.

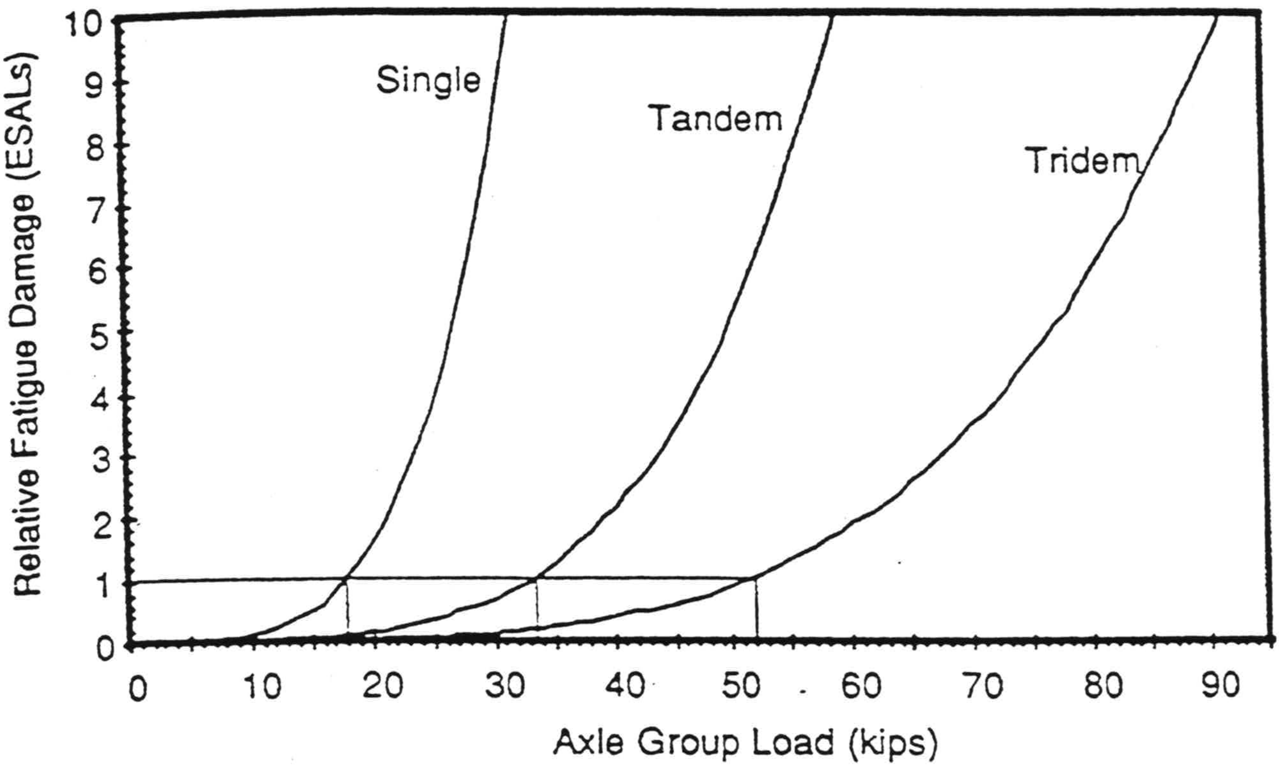


Fig. 4.1 Relative fatigue damage of PCC pavements versus axle load (Gillespie et al. 1993, pp. 13).

Note: Slab thickness : 10 inch; axle (tandem and tridem) spacing: 4.25 ft.

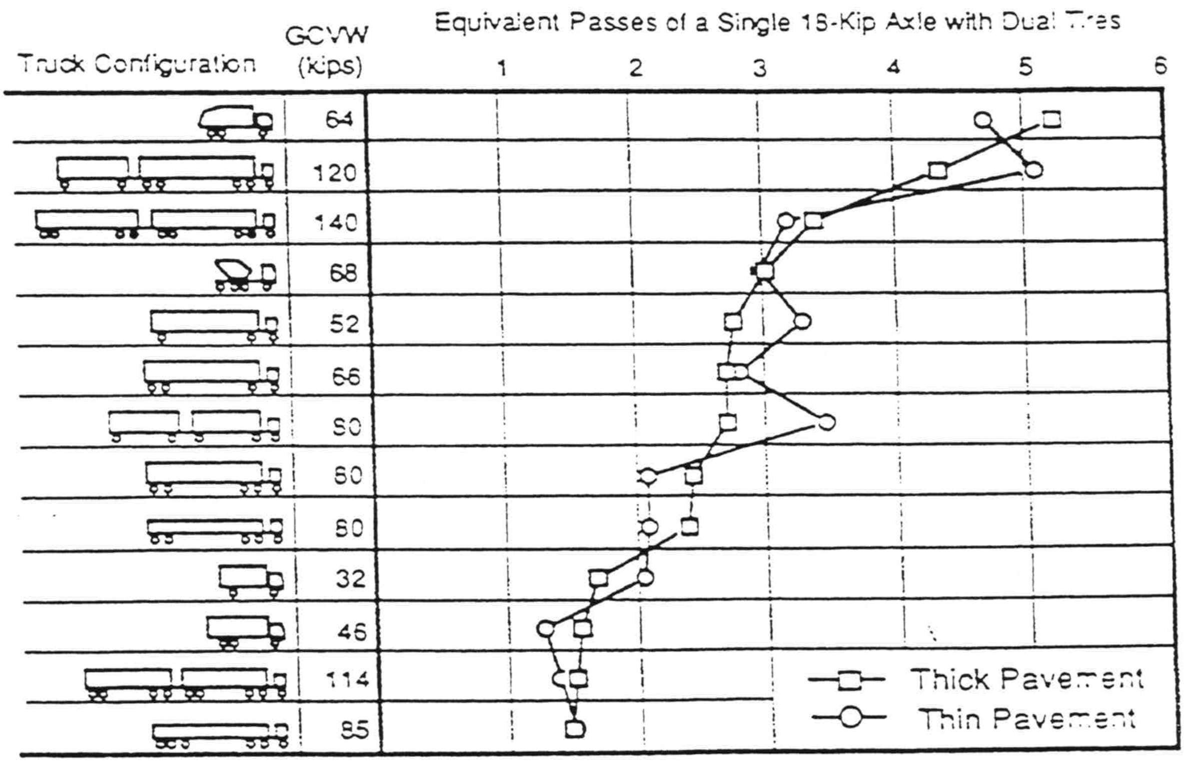


Fig. 4.2 Relative PCC pavements fatigue damage over a range of trucks and pavement thickness (Gillespie et al. 1993, pp. 14).

Note: Thin pavement: 7-in. slab on an 8-in. granular subbase; thick pavement: 12-in. slab on an 8-in. granular subbase.

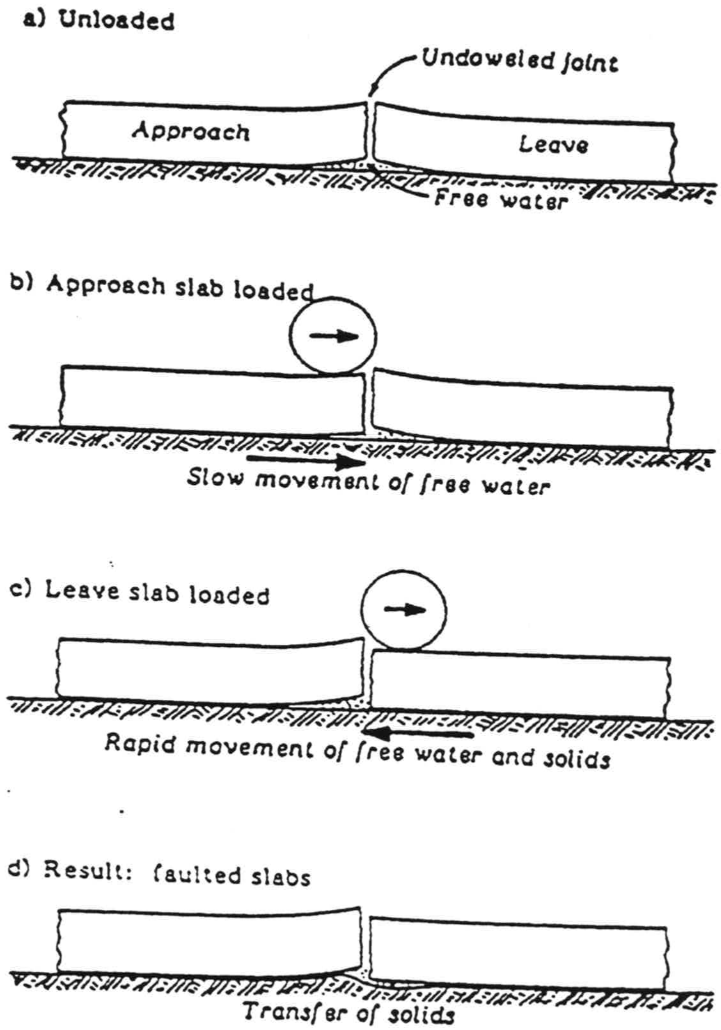


Fig. 4.3 Faulting caused by pumping of solids in PCC pavement joints (Mcghee 1995, pp. 29)

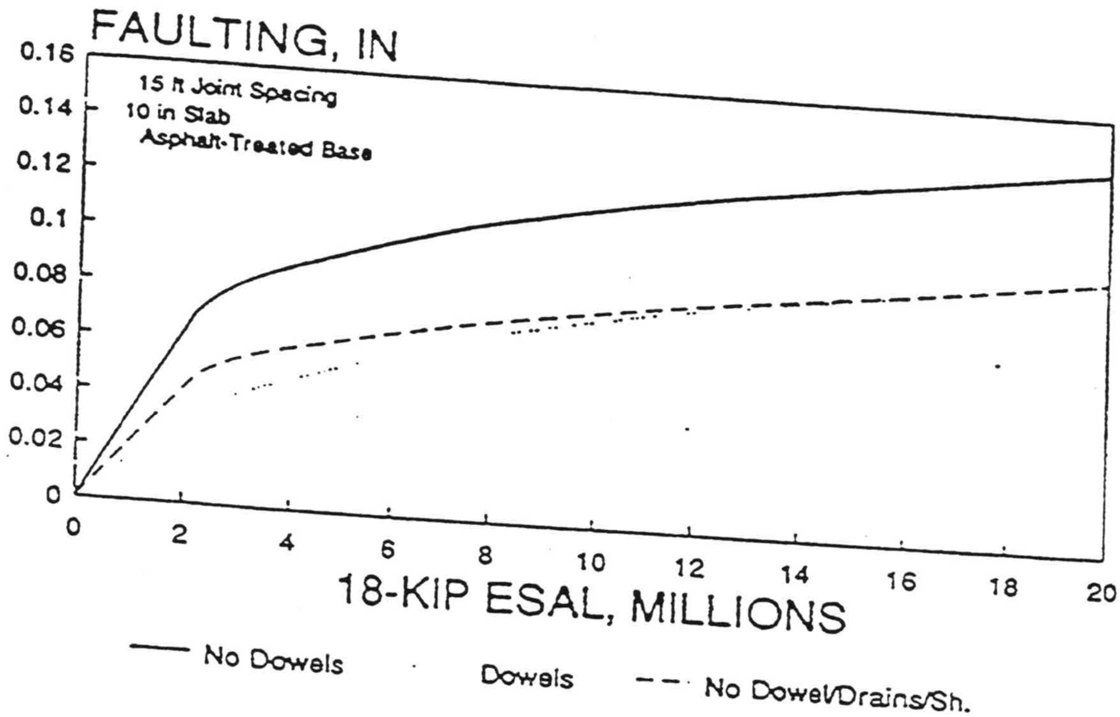
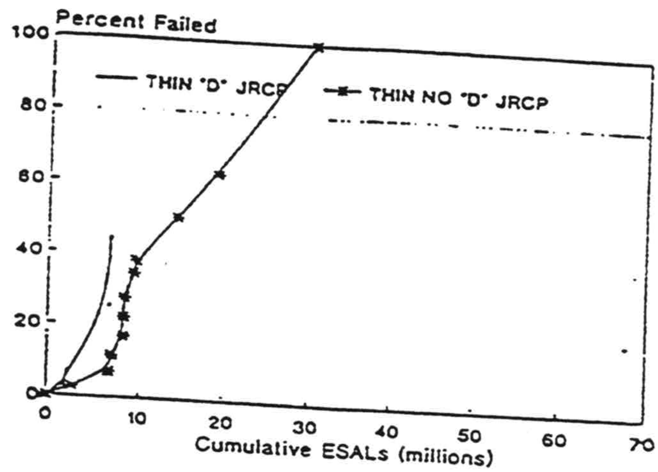
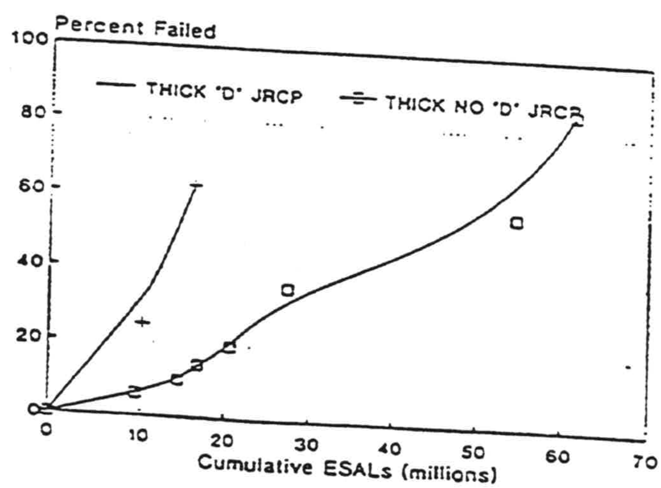


Fig. 4.4 Sensitivity of transverse joint faulting of PCC pavements to traffic volume. (Darter et al. 1991, pp.146).



(a) Thin AC overlays



(b) Thick AC overlays

Fig. 4.5 ESAL survival curves for AC overlays of JRCP (Hall et al. 1991, pp. 185-186)

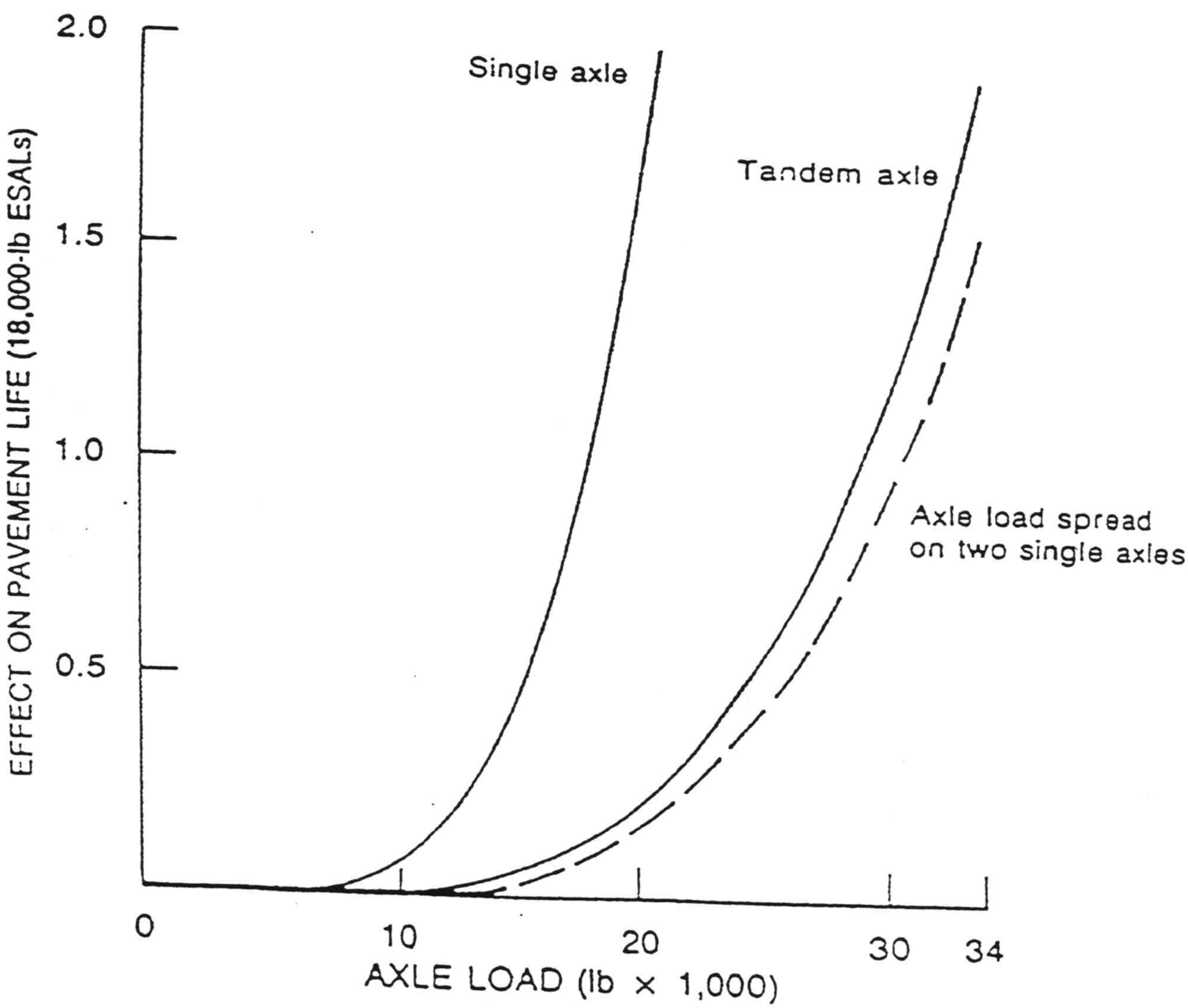


Fig. 4.6 Effect of axle load on PCC pavement life (TRB 1986, pp. 163).

Note: Slab thickness (D) = 9 in., and $p_t = 2.5$.

CHAPTER V

EFFECT OF TRUCK LOADING ON BRIDGE DETERIORATION

5.1 INTRODUCTION

Highway bridges can undergo significant deterioration due to a number of factors such as corrosion caused by de-icing agents, shrinkage cracking, thermal effects and the passage of heavy trucks. Although it is evident that heavy trucks increase the deterioration process of highway bridges, the extent to which all damage can be attributed to heavy trucks is difficult to quantify. Heavy truck damage can be categorized by overstress (exceeding the design stress) and fatigue (damage due to repeated loading).

This chapter summarizes the findings of a literature search regarding the effect of heavy trucks on bridge deterioration. In addition, results of simple overstress and fatigue analyses involving heavier Canadian trucks are presented. Finally, conclusions are drawn regarding the potential damage to Oklahoma's inventory of highway bridges.

5.2 OKLAHOMA INTERSTATE HIGHWAY BRIDGES

There are 6,669 bridges on the state highway system in Oklahoma, 1,582 bridges on the urban routes within cities (off system urban) and 14,062 bridges on the rural road system (off system). The conditional categorization of these bridges is performed through extensive field inspection following FHWA specifications (ODOT 1996a). As summarized in Table 5.1, eighty-two percent of Oklahoma's state highway bridges are currently considered adequate, ten percent are structurally deficient, and eight percent are considered functionally obsolete. A bridge is deemed structurally deficient if it harbors moderate cracking, while bridges with insufficient lane width and/or height are considered functionally obsolete (ODOT 1996b).

Table 5.1 Summary of Bridge needs of Oklahoma state (ODOT 1996a)

Highway System	Total No. of Bridges	Adequate bridges	Structurally deficient Bridges	Functionally Obsolete Bridges
State	6,669	5,457	678	534
Off System Urban	1,582	1,088	311	183
Off System Rural	14,062	6,928	6,242	710

There are approximately 1100 bridges within interstate highways I-35, I-40, I-44, I-235, I-240, I-40B and I-444 in Oklahoma, of which approximately 52 percent are concrete and 48 percent are composite concrete deck with steel I-beam girders (hereafter termed steel bridges). As summarized in Table 5.2, the majority of bridges with lengths under 75 ft are single and multi-span concrete culverts. The preponderance of bridges with lengths between 75 and 200 ft are concrete or steel, while most of the bridges greater than 200 ft long are steel. Prestressed concrete bridges are limited in number.

Table 5.2 Summary of Bridge Composition Expressed as a Percentage (ODOT 1996b)

Bridge Length	Steel	Prestress Concrete	Reinforced Concrete	Concrete Culvert
under 75 ft	0.4	0	1.0	26
75 to 200 ft	28	2.4	18	0.5
over 200 ft	20	2.5	1.1	0

The majority of highway bridges in Oklahoma are continuous span type. As summarized in Table 5.3, the majority of reinforced concrete bridges have span lengths less than 50 ft while steel bridge span lengths range from 25 ft to over 100 ft.

Table 5.3 Summary of Bridge Span Length Expressed as a Percentage (ODOT 1996b)

Span Length	Steel	Prestressed Concrete	Reinforced Concrete	Concrete Culvert
less than 25 ft	0	0	2	26
25 to 50 ft	11	0.3	14	0
50 to 75 ft	17	0.4	4	0
75 to 100 ft	14	2.7	0.5	0
over 100 ft	5.2	1.2	0.5	0

5.3 AASHTO BRIDGE DESIGN

The bridge design process is broadly divided into two steps: (i) determination of maximum shear force and bending moment for each member, and (ii) selection of a suitable section to resist the design stresses. During analysis, a load similar to the legal load is considered and magnified to represent a rare combination of multiple presence of overloads, impacts and load distribution. This magnification of the design load, or safety factor, is selected such that there is a very small probability that it will be exceeded during the design life of the bridge. An additional safety factor is introduced when selecting suitable member sections to account for inconsistent size, shape and quality of materials, as well as the unpredictable effect of weather and environment. The selection of safety factors depends on the importance of the structure, e.g., damage to a bridge pier may cause collapse of the whole structure while deck damage may be limited locally.

5.3.1 Loads

According to AASHTO guidelines, bridges should be designed for dead load, live load, impact or dynamic effect of live load, wind loads and other forces, e.g., longitudinal forces, centrifugal forces, thermal forces, buoyancy, earthquake forces, etc. The live load specified by AASHTO (1992), similar to AASHO (1966), consists of standard idealized trucks or of live loads which are equivalent to a series of trucks (equivalent lane loads). Two types of standard idealized trucks are provided: H-loading and HS-loading, representing respectively, a two-axle tractor and a two-axle tractor plus a single-axle semi-trailer. AASHTO stipulates a 20,000-lb single axle weight limitation; for tandem axles spaced between 40 and 96 inches the weight limitation is 34,000-lb. Equivalent lane loads consist of a uniformly distributed load (640 lb/ft for an equivalent HS20) and a concentrated load placed to produce the maximum moment (18 kip for an equivalent HS20) or the maximum shear (26 kip for equivalent HS20). When the bridge span is long (typically greater than 90 ft), the effect of equivalent lane load dominates over the single design truck. The other significant load in bridge design is the impact load which represents the dynamic effect of live load. Impact load is calculated as the fraction of live load with a maximum allowable factor of 0.30.

5.3.2 Truck Configuration

Because trucks with many configurations and weights operate within the United States, AASHTO specifies a single fictitious truck according to the purpose of the bridge (e.g. HS20 is the design truck for interstate bridges while H15 is the design truck for primary road bridges). These "umbrella" loadings represent the effect of each legal truck by balancing the effects of gross vehicular weight and truck length. Axle spacing is as important as axle weight in the design of bridges, e.g. a short truck would generate greater bending stresses on a bridge member than a longer truck of equal weight. To demonstrate the relationship between truck weight and configuration, Sorensen and Robledo (1992) determined critical lengths in simply supported spans, beyond which a specific truck would produce overstress, for different combinations of nine-axle twin-trailer trucks. The combinations included 15 percent length and 15 percent gross vehicular weight variations. It was shown that the critical span length increased if the truck length was increased or if the gross vehicular weight was decreased. It should be noted that if the truck length and weight are increased simultaneously the critical span remains essentially unchanged (Sorenson and Robledo 1992).

Federal bridge formula

In addition to the axle weight limitations, for the purpose of regulating variations in truck configuration, AASHTO introduced a Federal Bridge Formula in 1974; it is given by:

$$W = 500 [L \times N / (N-1) + 12 N + 36] \qquad (5.1)$$

where W = maximum weight in pounds carried on any group of two or more axles, including any and all weight tolerance, L = distance in feet between the extremes of any group of two or more consecutive axles, and N = number of axles under consideration. The Federal Bridge formula is applicable for trucks up to 80,000 pounds gross vehicular weight.

The objective of the Federal Bridge formula, or Formula B, is to limit axle weights and spacings such that trucks will not overstress highway bridges. For example, critical axle spacings (minimum allowable) for the 5-axle truck shown in Fig. 5.1, are calculated considering axle combinations 1 to 3, 2 to 5, and 1 to 5.

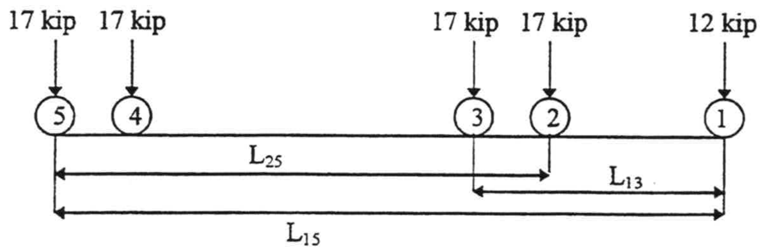


Fig. 5.1 Five-Axle Truck

Calculation of L_{13}

$W = 12{,}000 + 17{,}000 + 17{,}000 = 46{,}000$; $N = 3$

From Formula B: $46{,}000 = 500[\,L \times 3/2 + 12 \times 3 + 36]$; $\quad \therefore \mathbf{L_{13} = 13\ ft}$

Calculation of L_{25}

$W = 17{,}000 + 17{,}000 + 17{,}000 + 17{,}000 = 68{,}000$; $N = 4$

From Formula B: $68{,}000 = 500[\,L \times 4/3 + 12 \times 4 + 36]$; $\quad \therefore \mathbf{L_{25} = 39\ ft}$

Calculation of L_{15}

$W = 12{,}000 + 17{,}000 + 17{,}000 + 17{,}000 + 17{,}000 = 80{,}000$; $N = 5$

From Formula B: $80{,}000 = 500[\,L \times 5/4 + 12 \times 5 + 36]$; $\quad \therefore \mathbf{L_{15} = 51\ ft}$

Therefore, the axle spacings of this 80,000 lb. truck must meet or exceed these calculated values to comply with Formula B. Formula B allows for five percent overstress on HS20 bridges and 30 percent overstress on H15 bridges. It should be noted that a number of alternative formulas to the Federal bridge formula have been examined to circumvent the 80,000 lb. gross vehicular weight limit. In particular, the alternative formulas suggested by the Texas Transportation Institute (TTI) allow slightly greater gross vehicular weights in single unit and short combination trucks (TRB 1990a, pp. 6).

5.3.3 Rating Methods

Rating methods are adopted to determine the safe load capacity of new and existing bridges. A standard safety check of girders according to AASHTO (1989) should satisfy the following criterion:

$$D + L\,(1 + I) \leq C \qquad (5.2)$$

where D = stress induced by dead load, L = stress induced by live load, I = impact factor, and C = prescribed limiting or safe level of bending stress.

Within the traditional inventory rating procedure, the safe level of bending stress C for steel bridges is commonly limited to 55 percent of the yield strength of steel. For concrete bridges, C is limited to 35 to 40 percent of the compressive strength of concrete. When considering additional loading, the operating rating procedure is adopted. The AASHTO Manual for Maintenance Inspection of Bridges (1983), defines an operating rating in which the safe level of stress can be as great as 75 percent of the yield strength of steel or between 50 to 60 percent of the compressive strength of concrete (AASHTO 1994). This rating method is considered when posting maximum allowable load limits to bridges. Among 46 states, eight routinely conduct inventory rating, twenty-six conduct operating rating and twelve use a flexible or intermediate level (TRB 1990a, pp. 94). In Oklahoma, inventory rating is followed for legal trucks and operating rating is followed for overload permit trucks (see Section 2.2.3).

5.4 DAMAGE OF BRIDGE MEMBERS

Common mechanisms of crack generation in bridge members can be attributed to shrinkage of concrete, corrosion, thermal effects, overstress and fatigue. Various field observations and tests have indicated that traffic-load induced bridge damage initiates as deck cracking and is later evidenced through cracking in beams, girders and columns (James et al. 1988, pp. 8).

5.4.1 Deck Damage

Transverse flexural cracking in concrete is the primary mode of deck damage (Kostem 1985). Longitudinal deck cracking may also occur on the lower surface of the slab near the mid-spacing of the girders due to two-way action. Following the initiation of flexural damage, cracks may propagate through the bridge slab due to further addition of heavy loads. The Ontario Bridge Design Code has identified that failure of reinforced concrete bridge decks is primarily due to punching shear rather than flexure (James et al. 1988, pp. 9). This may be because the presence of large in-plane compressive forces due to the restraint of bridge deck expansion under loading increases the bridge deck flexural capacity; thus, the controlling factor becomes punching shear. Consequently, the mechanism of progressive deck damage is related to flexure, while in situations involving bridge restraint, ultimate capacity may be governed by punching shear.

5.4.2 Beam and Girder Damage

Beam and girder damage usually occurs after substantial deck damage. Flexural damage occurs near the midspan, while flexure-induced shear causes cracks at the interface of the deck and the beams.

5.4.3 Detail Damage

Very often, the damage observed in steel bridges occurs in the details, i.e., cover plates, stiffeners and welded connections. Cracking in details initiates due to repetitive loading and owing to such fatigue, the design life of steel bridges is reduced.

5.5 EFFECT OF OVERSTRESS ON BRIDGES

Overstress, attributed to the passage of heavy trucks, occurs when a bridge member is loaded beyond its design stress. To evaluate the impact of heavy trucks on possible bridge overstress, a computer program based on influence line analysis was developed in this study to determine the maximum bending moment induced by the presence of specific truck configurations over simply supported bridge spans. Analyses were performed for the HS20 and several Canadian trucks shown in Figs. 5.2 through 5.5. Based on the regulations depicted in Figs. 5.2 to 5.4, configurations for maximum and minimum effects of all three Canadian trucks were determined and are shown in Fig. 5.5. For simplicity, axle weights for both maximum and minimum combination trucks were kept identical, however, axle spacing was varied.

Maximum bending moments were determined considering bridge span lengths ranging from 40 to 150 ft at 10 ft intervals. Large vehicles spread out the load to more than one span for continuous bridges with span lengths less than 40 ft; the selection of 150 ft as the maximum simply supported span length is representative for all highway bridges in Oklahoma.

Presented in Figure 5.6 are the bending moment difference ratios (BMDR) for each truck type, given by:

$$\text{BMDR} = \frac{(\text{BM for truck of interest - BM for HS20})}{\text{BM for HS20}} \times 100\,\% \qquad (5.4)$$

A positive BMDR indicates the design stress has been exceeded. As bridges are designed for an inventory level of stress (zero line in Fig. 5.6), consideration of operating stress levels would permit an increase of approximately 13 percent BMDR. From Fig.5.6, it is evident that the TST(min) truck would cause less damage than the HS20 trucks for all span lengths. However, the remaining five Canadian trucks analyzed are capable of considerable overstress damage. Over all span lengths considered, the B-Train(max) and TST(max) trucks impart stresses in excess of the inventory level, by as much as 50 percent for span lengths near 90 ft. The operating level of stress is exceeded for span lengths greater than 55 ft. Stresses induced by the B-Train(min), ACTD(max) and ACTD(min) exceed the inventory level for span lengths greater than 60-70 ft and exceed the operating stress level for spans greater than 75-90 ft.

Given the inventory of bridge span lengths in Oklahoma (see Table 5.2), for the trucks analyzed, it appears that concrete culvert and reinforced concrete bridges are not at risk to overstress damage. However, with over 75 percent of Oklahoma's steel bridges incorporating span lengths in excess of 50 feet, considerable overstress damage is possible.

5.6 EFFECT OF FATIGUE ON BRIDGES

5.6.1 Manifestation of Fatigue Damage

Significant fatigue damage is commonly observed in steel bridges and rarely in concrete bridges (Moses 1989, TRB 1990a). Fatigue distresses in steel bridges usually occur within the concrete deck and in the steel details (e.g. cover plates, welded connections, etc.).

In an effort to evaluate the nature and extent of damage to bridge structures in Texas, the Texas Transportation Institution (TTI) investigated 24 different bridge types. TTI personnel investigated visible damage to the wearing surface, the bridge deck bottom, the supporting girders and diaphragms, as well as bearings, bents and columns. The study revealed that flat slab/concrete girder bridges remained in good condition after decades of service while prestressed concrete box girder and prestressed concrete deck/girder bridges underwent little visible damage after 10 years of service. However, composite reinforced concrete decks with steel I-beam girders suffered significant damage. Among the noted distress were grid-like cracks on the bottom of the bridge deck, fatigue cracks in welded connections, and corrosion at the deck and beam joints. Bearings, bents and columns were found in good condition and few visible cracks were found in diaphragms (James et al. 1988, pp. 38).

To further evaluate the effect of truck loading on fatigue damage, TTI investigated two companion composite concrete and steel I-beam bridges which carried an identical average volume of daily truck traffic. However, the southbound bridge accommodated mostly heavily loaded trucks while the northbound bridge accommodated mainly empty trucks. Consequently, significant differences in deck cracking were observed between the southbound and northbound bridges.

Crack densities in the bridge decks at various transverse locations are shown in Fig. 5.7. It is evident that the bridge deck crack density on the southbound bridge is greater than that of the northbound bridge. In addition, crack densities in the center and outermost lanes, those mostly used by trucks, are greater than that observed in the left lane. This further suggests that heavy trucks are causing the fatigue damage. The observed relationship between bridge deck crack density and number of 40, 50, and 60 kip truck passages is shown in Fig. 5.8, while the noted relationship between crack density and 10, 12, 14, 16, and 18 kip single-axle passages is shown in Fig. 5.9. While it is intuitive that crack density should increase with the number of truck passages or the single-axle passages, Figs 5.8 and 5.9 reveal two striking results regarding the effects of heavier trucks. First, the number of truck passages required to achieve a level of crack intensity is strongly weight dependent. For example, to achieve a given level of crack density (Fig. 5.9), 12 million 10-kip axle passages were required versus less than 1 million 18-kip axle passages. Second, the rate of crack increase is much greater for heavier truck and axle weights. For example, to increase the crack density 12 percent, 16 million 10-kip axle passages are required versus less than one million 18-kip axle passages.

Similar observation have been made in Oklahoma, where eastbound interstate bridges accommodating loaded trucks exhibit greater deck cracking than westbound bridges that see mainly empty trucks (ODOT 1996b).

Fatigue damage increases in a cumulative manner with the passage of each vehicle, and decreases the expected life of the bridge in the process. Moreover, cracks initiated through repetitive loading in concrete decks and welded connections of steel beams of composite bridges may also induce further damage or collapse due to stress resulted from a single truck loading (James et al. 1988, pp. 19).

5.6.2 Evaluation of Fatigue Damage of Existing Bridges

Given the widespread fatigue-related damage observed in steel bridges, researchers have developed evaluation techniques to quantify the effects of repetitive loading on existing bridges. Through an NCHRP effort, a simplified method to evaluate the fatigue life of existing uncracked and/or unrepaired steel highway bridges was developed. According to NCHRP Report 299 (Moses et al. 1987), the remaining safe fatigue life (Y_f) of steel bridges is determined by:

$$Y_f = \frac{K \times 10^6}{T_a \, C \times (R_s \, S_r)^3} - a \qquad (5.5)$$

where,
S_r = effective stress range in ksi,
T_a = lifetime average daily truck volume in outermost lane,
K = detail constant,
C = number of stress cycles per truck passage,
R_s = reliability factor (1.35 for redundant members and 1.75 for non-redundant members),
a = present age of bridge in years.

Equation (5.5) illustrates the significant relationship between truck weight (accounted for through the effective stress range) and the fatigue life of steel bridges. From Equation (5.5), it is evident that doubling the effective stress range reduces the remaining safe life by a factor of eight. The effective stress range (S_r) is calculated from stress-range histograms, prepared through strain values collected from site data (Moses et al. 1987, pp.11) and is determined by:

$$S_r = (\Sigma f_i \, S_{ri})^{1/3} \qquad (5.6)$$

where,
f_i = fraction of stress ranges within a stress interval i, and
S_{ri} = midwidth of stress interval i.

Each stress interval in the stress-range histogram represents the difference between maximum and minimum stresses resulting from the passage of one truck. Therefore, if the percentage of heavier trucks increases in the traffic composition, the effective stress range would increase and thus reduce the fatigue life of the bridge.

Another important factor when considering the fatigue life of a bridge is life-time average truck traffic volume which depends on the present truck traffic volume and its growth rate. As expected, it is evident from Equation (5.5) that an increase of truck traffic volume will reduce the fatigue life. Each truck which crosses over a bridge produces one or more stress cycles. These stress cycles are included in Equation (5.5) as factor C. Short span bridges (under 40 ft) may be subjected to more than one stress cycle per vehicle passage since front and rear axles cause separate loadings (Moses 1989, pp.2-19). The NCHRP Report 299 (Moses et al. 1987, pp.72) recommends a value of C greater than 1.0 for spans under 40 ft.

5.7 SUMMARY

Damage to existing bridges can result from overstress or fatigue due to the passage of heavy trucks. Through an analysis of potential overstress due to heavy trucks, it was determined that while the majority of reinforced concrete and concrete culvert bridges are not susceptible to overstress damage, the majority of steel bridges may undergo significant overstress due to the passage of heavy trucks. Steel bridges are also susceptible to fatigue damage and subsequent decreased life when subjected to increased truck loading.

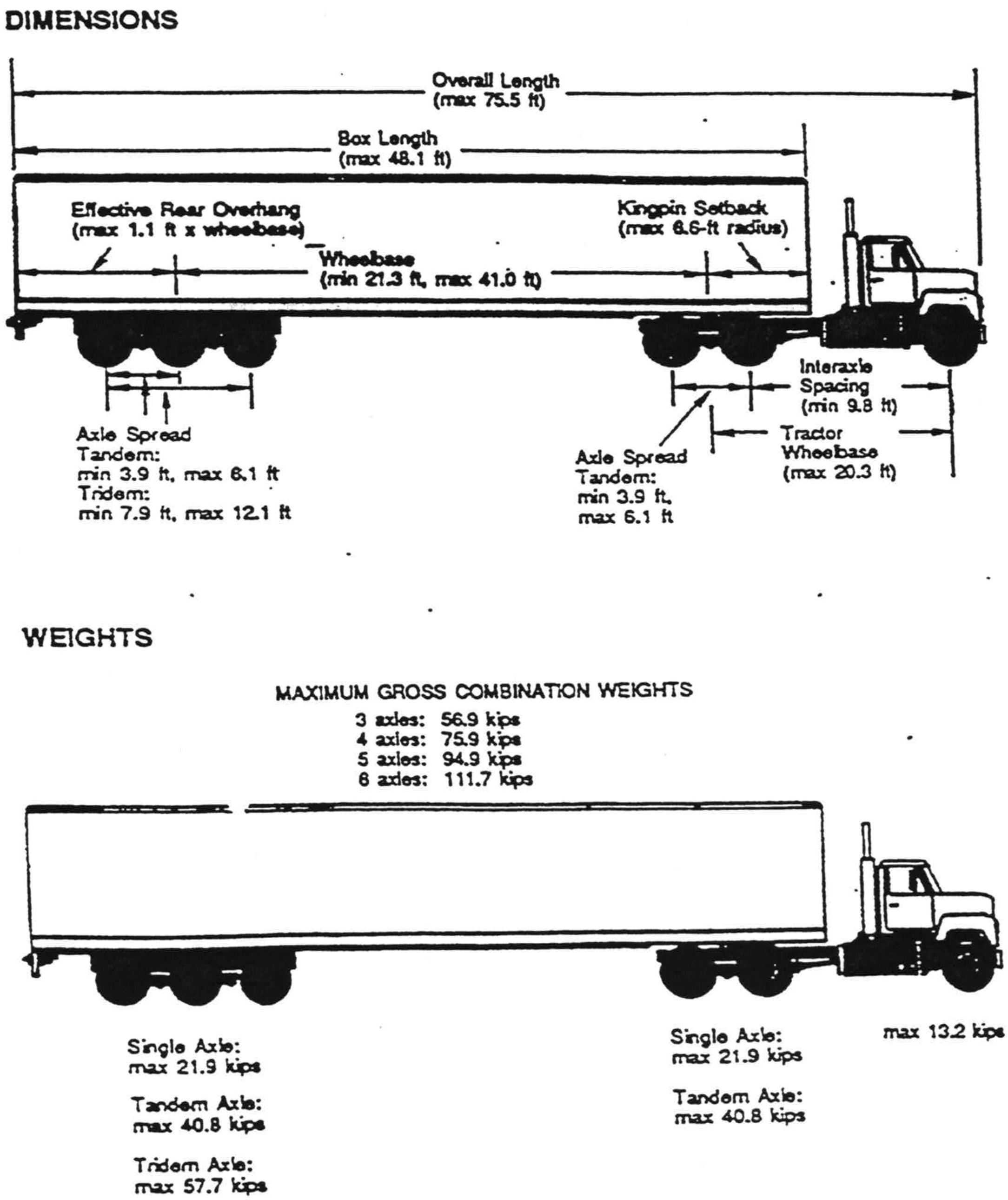
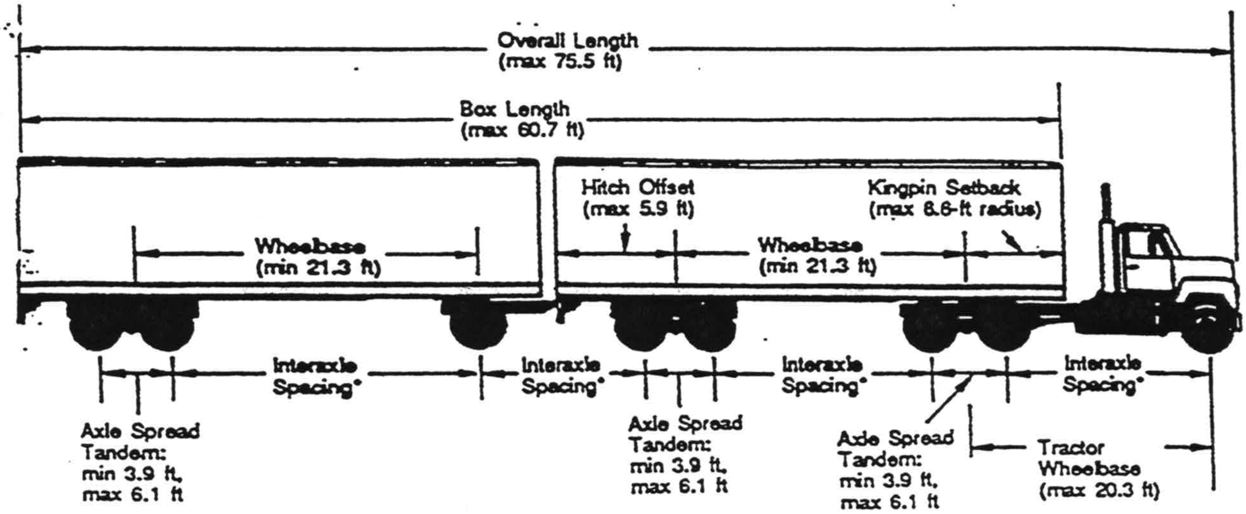


Fig. 5.2 Canadian interprovincial limits for tractor-semitrailers (TST) (RTAC 1988)

DIMENSIONS



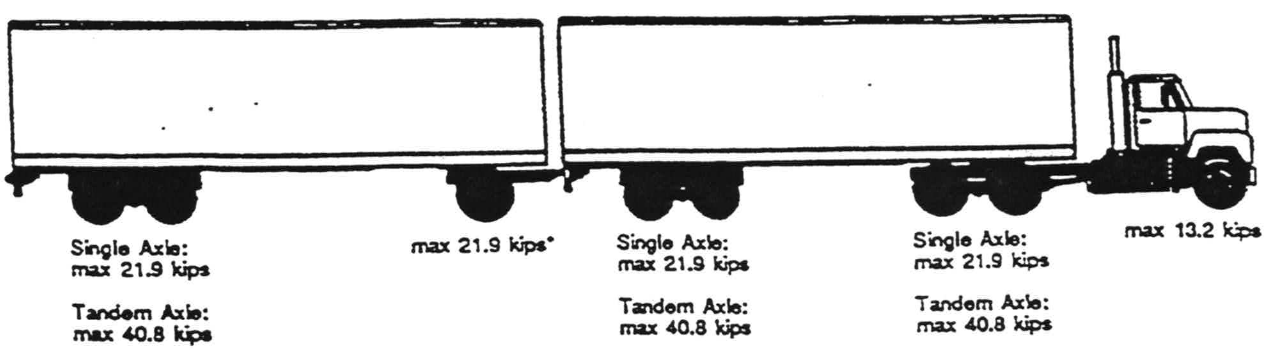
*Interaxle spacing:

Single–single, min 9.8 ft
Single–tandem, min 9.8 ft
Tandem–tandem, min 16.4 ft

WEIGHTS

MAXIMUM GROSS COMBINATION WEIGHTS

5 axles: 95.4 kips
6 axles: 114.4 kips
7 axles: 128.5 kips
8 axles: 128.5 kips



*Note: Second trailer limited to a maximum weight of 38.4 kips or weight of lead trailer, whichever is lower.

Fig. 5.3 Canadian interprovincial limits for A- and C-train doubles (ACTD) (RTAC 1988)

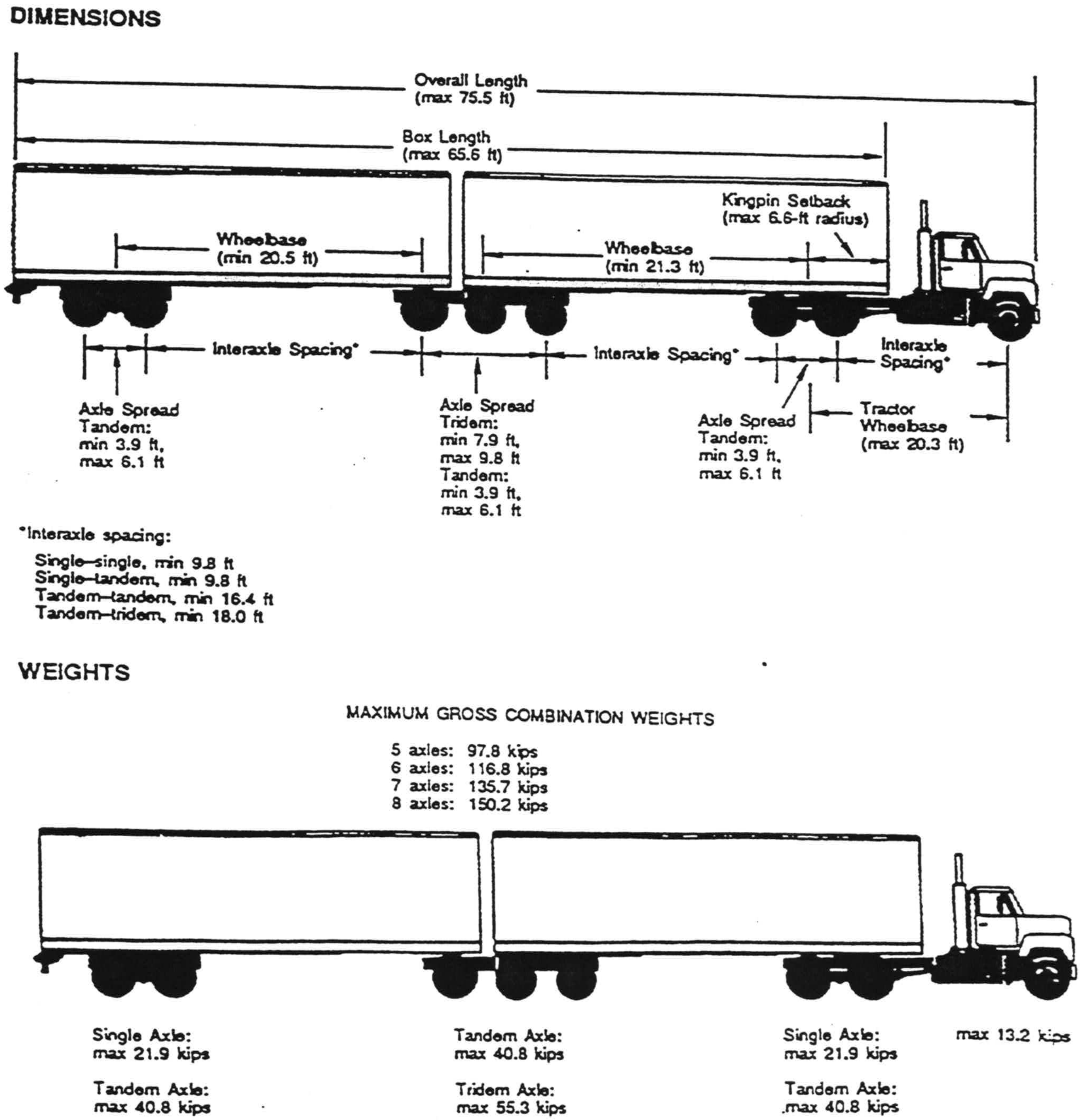
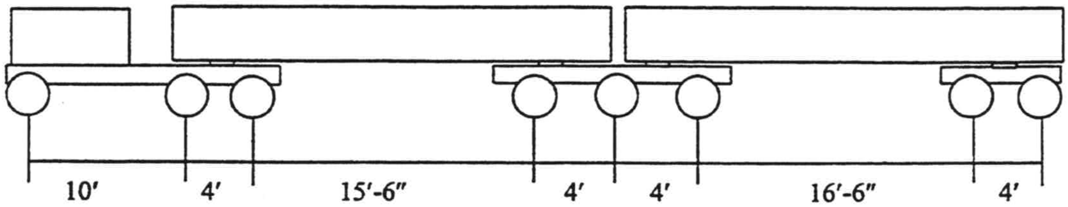
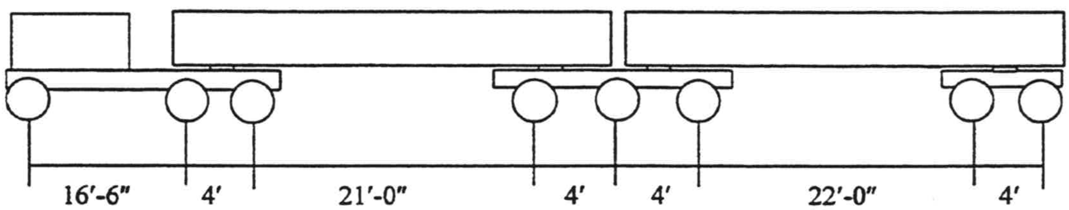


Fig. 5.4 Canadian interprovincial limits for B-train doubles (B-Train) (RTAC 1988)

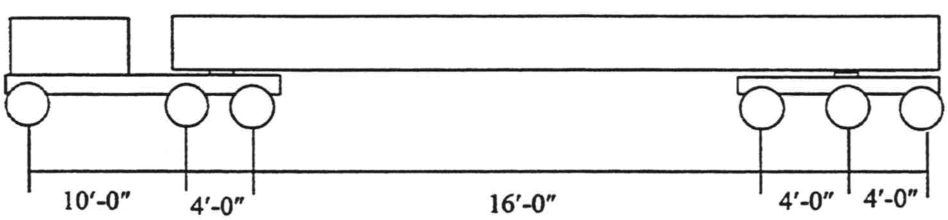


(a) B-Train (max)

Front axle: 13.2 kip
Tandem axle: 18.73 kip
Tridem axle: 16.19 kip

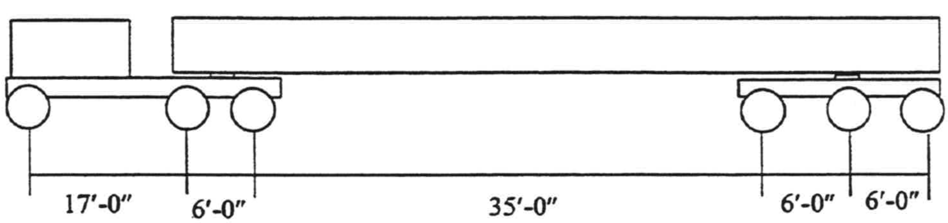


(b) B-Train (min)



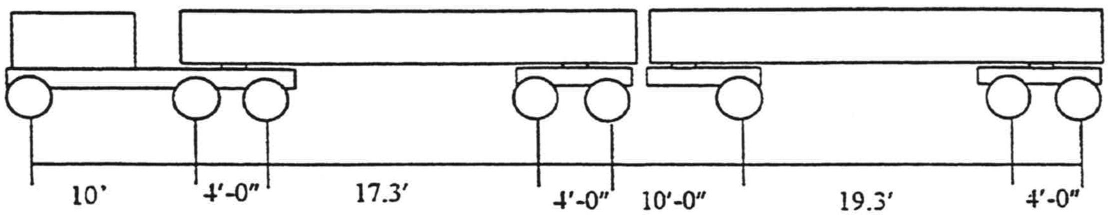
Front axle: 13.2 kip
Tandem axle: 18.73 kip
Tridem axle: 16.19 kip

(c) TST (max)



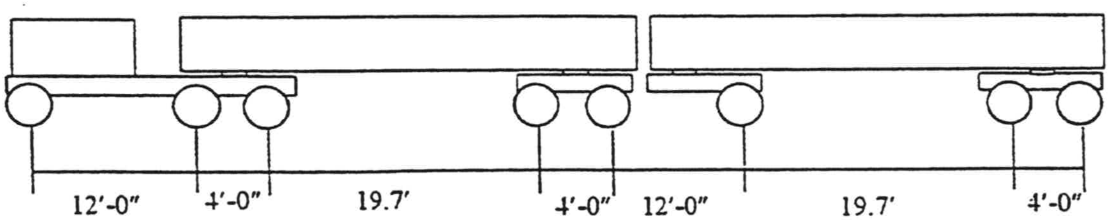
(d) TST (min)

Fig. 5.5 Configuration of Different Canadian Interprovincial Trucks



(e) ACTD (max)

Front axle: 13.2 kip
Tandem axle: 16.47 kip
Single axle: 16.47 kip



(f) ACTD (min)

Fig. 5.5(Contd.) Configuration of Different Canadian Interprovincial Trucks

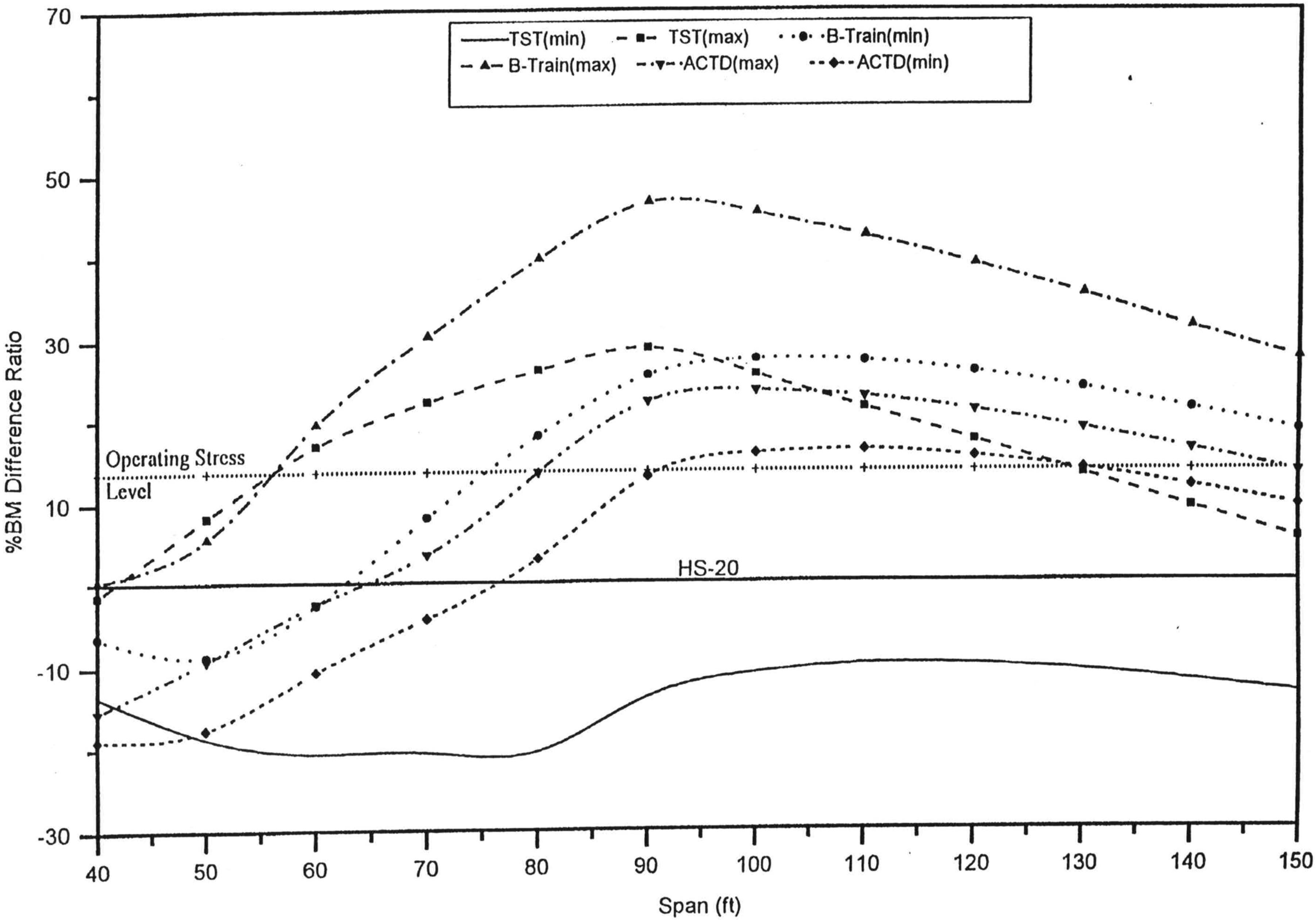


Fig. 5.6 BMDR vs. Span Length

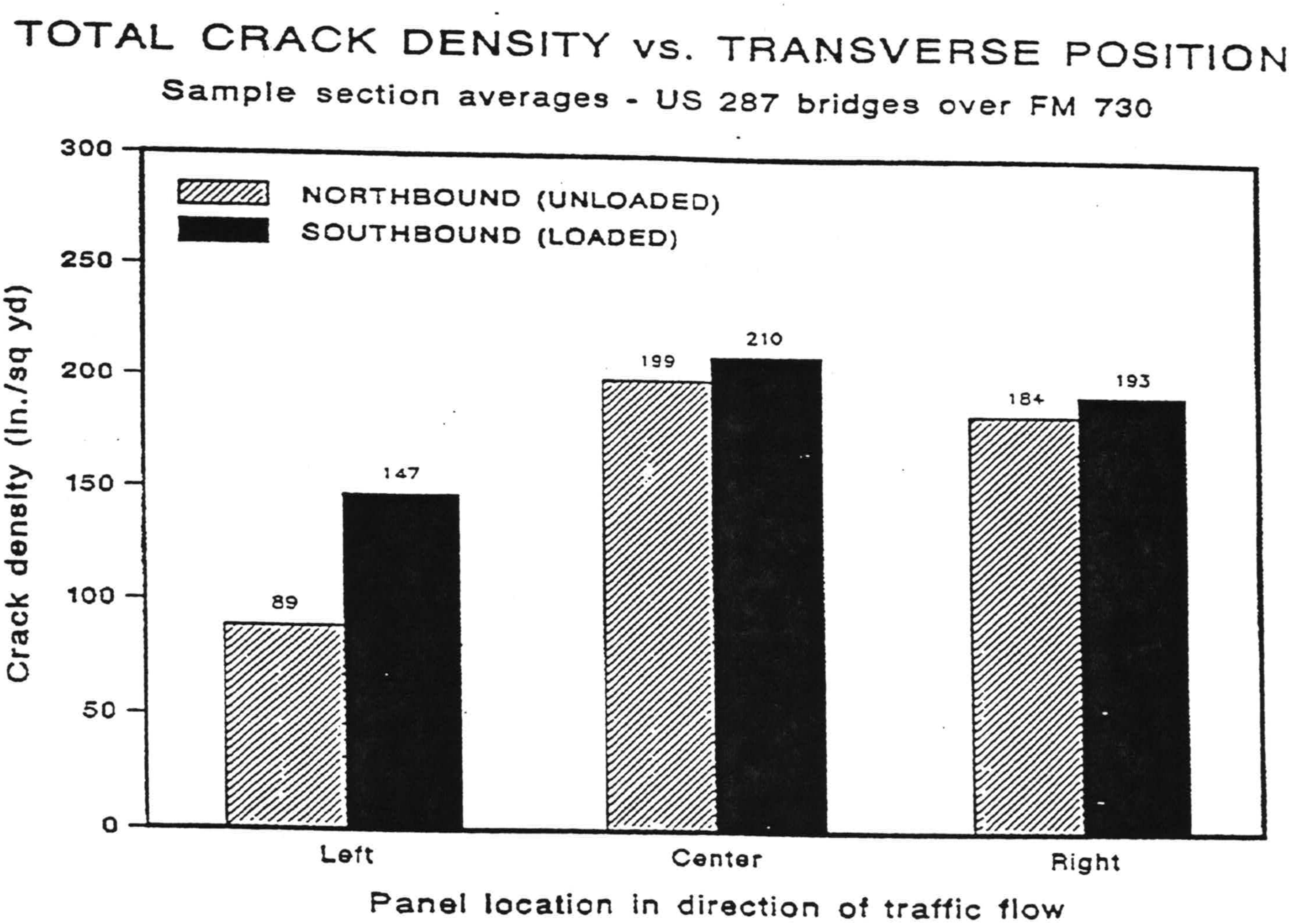


Fig. 5.7 Transverse distribution of total crack density (James et al. 1988)

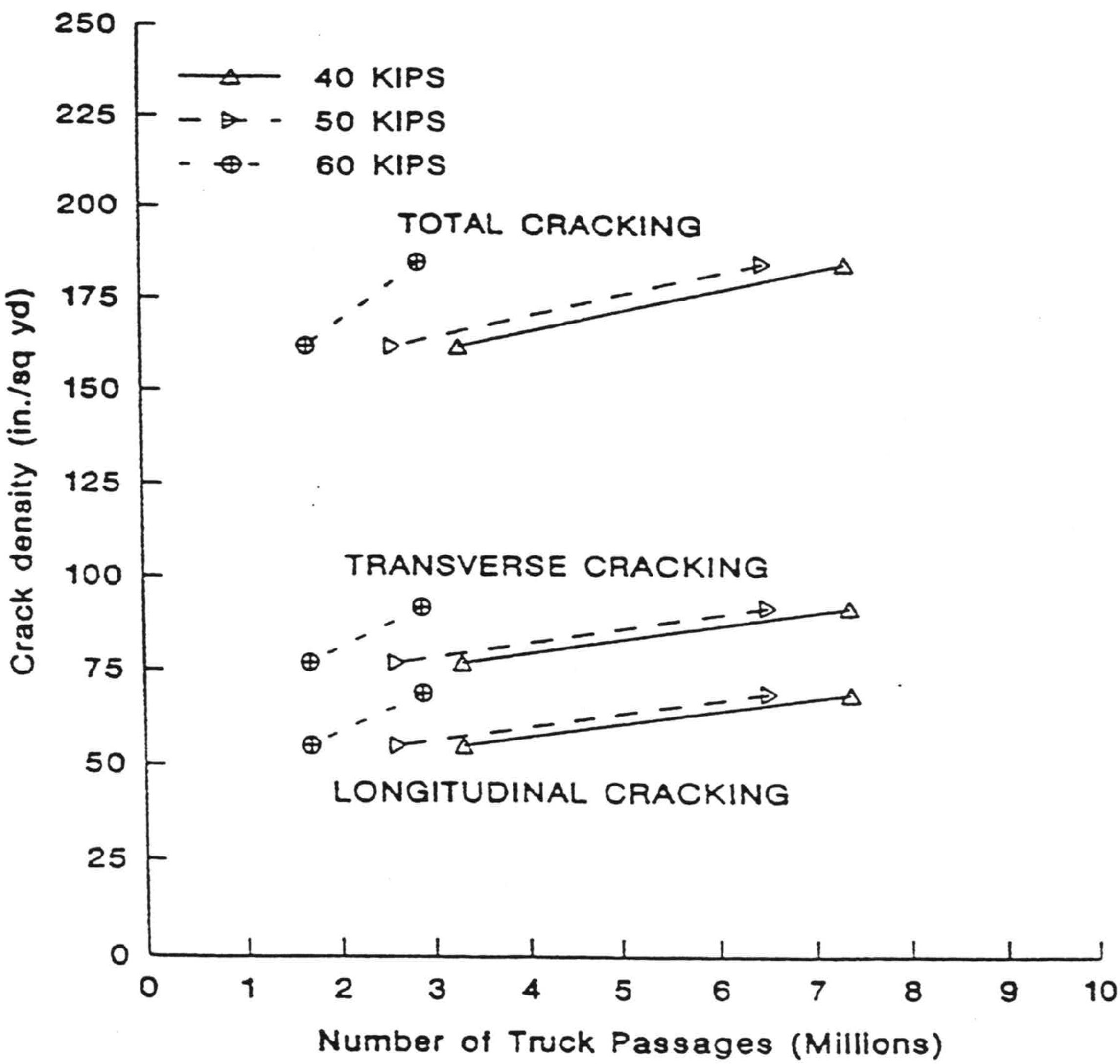


Fig. 5.8 Average deck cracking correlated to number of different GVW truck passages (James et al. 1988)

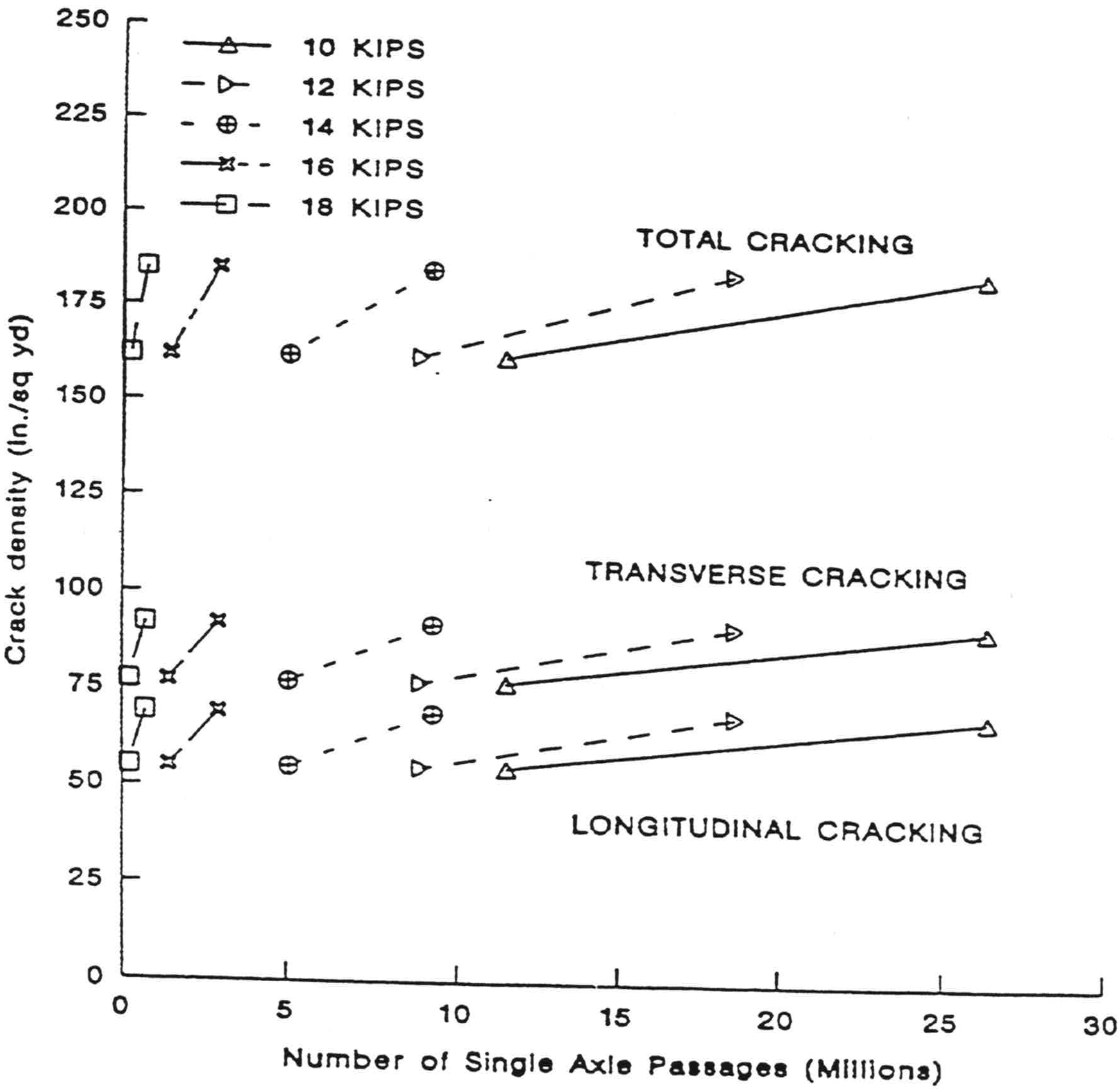


Fig. 5.9 Average deck cracking correlated to number of different weight axle passages (James et al 1988).

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER STUDY

The material presented in this report is based on a comprehensive literature search of current and past journals, periodicals, and publications in transportation engineering. This chapter summarizes the conclusions from the study and highlights areas where further research is needed.

6.1 CONCLUSIONS

- Federal and state governments fund highway expenditures through user and non-user taxes in the form of fuel taxes and registration fees. Truck user fees and taxes do not appear to be equitable with the extent that these heavy vehicles cause damage to the transportation infrastructure.
- In Oklahoma, the allowable maximum load for a single axle is 20,000 lb. For a tandem-axle and a tridem-axle these limits are 34,000 lb, and 42,000 lb, respectively. The gross vehicle weight (GVW) is limited to 80,000 lb for Interstate and federally designed highways and 90,000 lb for all other state highways and supplemental roads. About 20% of the vehicles operating on federal-aid highways have axle or gross weight in excess of legal limits, contributing to rapid deterioration of pavements.
- The current vehicle size and weight limits are not uniform throughout the States in the U.S. On Interstate highways, a single axle load limit varies from 20 kips to 22.5 kips (Vermont), a tandem-axle load limit varies from 34 kips to 44 kips (Florida),

and a GVW limit varies from 80 kips to 86.4 kips (New York). Similarly, the axle load limits as well as the GVW limits in Mexico and Canada are quite different from those in the United States. The GVW limit of 7-axle Tractor-Twin-Trailers in Mexico is approximately 1.75 times greater than the limits in the U.S. The GVW limits of Canadian Tractor-Twin-Trailers (7 axles) are approximately 1.5 times higher than the U.S. limits. If these trucks are allowed to travel on the existing roads and bridges in Oklahoma, they are likely to accelerate the deterioration of our transportation infrastructure system.

- The current Special Permit System for overweight vehicles in Oklahoma is based on the gross vehicle weight: a flat fee of $20 plus $5 for every 1,000 lb overweight. This permit system does not reflect the actual damages caused by the overloaded trucks to our transportation infrastructure system.
- The penalty structure for illegal overloaded trucks is not related to the extent of pavement damage. Also, illegally overloaded trucks often escape fines because of failure of the administrative procedures.
- Approximately 152 miles of the Oklahoma Interstate highway pavements are currently considered critical, and about 10 miles are considered tolerable. More road mileage is likely to become structurally inadequate with the implementation of the NAFTA because of the increased frequency of the heavier Canadian and Mexican trucks on the U.S. Interstate highways.
- Both AC and PCC pavements in Oklahoma are designed according to the AASHTO (1986) guidelines. The projected traffic load is determined by converting the axle-

loads during a 20-year analysis period into 18-kip equivalent single axle loads (ESAL factors based on the AASHO Road Test). The ESAL factors vary sharply with axle load, following approximately a fourth power relationship. The AASHO Road Test included only single axle and tandem axle loads up to 30 kips and 48 kips, respectively. Load equivalency factors suggested by AASHTO for single and tandem axle loads higher than those used in the Road Test as well as for tridem axles are primarily based on extrapolations and do not appear to have any physical and experimental basis. Also, the current design methods do not appear to adequately address any unexpected growth in truck traffic intensity.

- The imparted ESAL of any truck on PCC pavement is more than that on AC pavement. For example, a five-axle tractor-semi-trailer (3-S2) imparts 4.04 ESALs on PCC pavement (D=10 in.; p_t=2.5) compared to 2.37 ESALs on AC pavement (SN=5; p_t=2.5).
- The conventional 3-S2 are the most common type of truck on the U.S. highways (70% of all trucks). Approximately 92% of the total ESALs on rural Interstate highway pavements are contributed by 3-S2 type trucks. The corresponding contribution for all (rural and urban) highway pavements is about 80%.
- The AASHTO equivalency factors can be used to estimate the effects of heavy trucks on pavement damage. Truck traffic carrying heavy weights plays a major role in fatigue damage and rutting damage of AC pavement.
- The fatigue damage of pavements is highly dependent on the truck axle load. The fatigue damage is found to be proportional to the axle load raised to the fourth

power. When the current single axle load limit of 20 kips is increased by 10% to 22 kips, the resulting fatigue damage is increased by 46%. As a result of this damage, the remaining life of existing pavements decreases rapidly as a power function of load.

- The primary cause of permanent deformation due to common truck traffic in AC pavement is the GVW and axle loads. Severe rutting in AC pavement can be caused when axle loads in trucks are too high (above legal limits). The permanent deformation of AC pavement due to a legal axle load is usually limited within the asphalt layer. To the contrary, only 5% of the permanent deformation for a higher axle load develops within the asphalt layer, 10% within the base course and the remaining in the subgrade layer. Faulting and pumping of PCC pavement are directly influenced by the repetitive, heavy axle loads of which loaded trucks are an integral part.
- Heavy vehicles (GVW more than 7,700 lb) are responsible for approximately 99% of the total traffic-related damage to pavements. An 80 kip truck weighing the equivalent of 20 automobiles has the same damaging potential as 9,600 automobiles.
- Tandem and tridem axles are very effective means for increasing truck load capacity without significantly increasing the damage potential. A tandem-axle load limit could be increased as much as 40,000 lb with no more damage than that imposed by two widely separated single axles, each carrying 20,000 lb.
- The serviceability of pavements is shortened by the action of heavy trucks. To prevent the premature roadway failure, a high level of maintenance is mandatory.

- Documented case studies and detailed analyses have shown that heavy trucks are responsible for significant damage to bridges. Concrete decks and details in steel girder bridges are most susceptible to fatigue damage. Concrete bridges have exhibited few signs of fatigue distress. As a consequence, greater truck loads and volume are likely to increase fatigue damage of steel bridges.
- Through influence line analysis, it was found that bridges with span lengths greater than 50 ft are susceptible to overstressing by heavy trucks, i.e. Canadian trucks which exceed U.S. weight limits. Since concrete culvert and concrete girder bridges in Oklahoma have span lengths below this level, it may be concluded that these bridges are not susceptible to overstressing by the Canadian trucks.
- The majority of steel bridges in Oklahoma have span lengths greater than 50 ft. According to the analysis conducted herein, these bridges may be susceptible to significant damage due to overstressing.

6.2 RECOMMENDATIONS FOR FURTHER STUDY

- The current permit issuance policy, fee structures, and fine enforcement issues in Oklahoma need to be carefully reviewed and changed as appropriate. Research is needed to quantify the damage potential of various types of trucks and other vehicles so that an appropriate fee structure could be established.
- The observations made in the present study is primarily based on the information available in the public domain literature. Because the design, construction, maintenance, and management practices can vary significantly among various States in the U.S., specific conclusions about the impact of loaded trucks on transportation

infrastructure deterioration cannot be made from a general literature survey. A more comprehensive study can be undertaken in which the existing data (e.g., traffic data, sufficiency rating, structural condition, maintenance cost, etc.) be analyzed and new data collected to evaluate the specific impacts of loaded trucks on our transportation infrastructure system.

- Because the Canadian and the Mexican trucks have the potential to accelerate damage to the transportation infrastructure in Oklahoma, it is recommended that a comprehensive study be undertaken to assess the specific damage potentials and their significance. Such a study would involve field data collection, laboratory testing as well as numerical modeling to avoid any "surprises" that may result from the implementation of the NAFTA.
- Impact and advantages of Turner trucks should be evaluated considering field data and numerical modeling.
- The current practice of using strains at the bottom of the surface course in an asphalt concrete pavement as the indicator of fatigue damage is not justified. To better establish the mechanisms of fatigue damage in AC and PCC pavements due to heavy axle loads, field observations and numerical/analytical modeling efforts are necessary, focusing on stresses and strains throughout the pavement layers.
- Dynamics of loaded trucks should be considered to evaluate the actual damage potential of different types of trucks on pavements and bridges.

- Since most of the steel bridges are susceptible to overstressing due to the passage of heavy loaded trucks, according to the preliminary analysis conducted herein, future detailed analysis of such bridges is recommended.
- Fatigue damage is very prominent for steel bridges. Thus, detailed fatigue life evaluation of such bridges for the expected truck traffic composition is also suggested.

AASHO (1946). "Recommended Policy on Maximum Dimensions and Weights of Motor Vehicles to be Operated Over the Highways of the United States." *American Association of State Highway Officials*, Washington, D.C.

AASHO (1966). "Standard Specification for Highway Bridges." *American Association of State Highway Officials*, Washington, D.C.

AASHTO (1983). "Manual for Maintenance Inspection of Highway Bridges." *American Association of State Highway and Transportation Officials*, Washington, D.C.

AASHTO (1984). "Our Highways: Why Do They Wear Out? Who Pays for Their Upkeep?" AASHTO Subcommittee on State Highway and Transportation Officials, *American Association of State Highway and Transportation Officials*, Washington, D.C.

AASHTO (1986). "AASHTO Guide for Design of Pavement Structures." *Published by the American Association of State Highway and Transportation Officials*, 444 N. Capitol Street, N.W., Suite 249, Washington, D.C.

AASHTO (1989). "Standard Specification for Highway Bridges." *American Association of State Highway and Transportation Officials*, Washington, D.C.

AASHTO (1992). "Standard Specification for Highway Bridges." *American Association of State Highway and Transportation Officials*, Washington, D.C.

AASHTO (1993). "AASHTO Guide for Design of Pavement Structures 1986." *Published by the American Association of State Highway and Transportation Officials*, 444 N. Capitol Street, N.W., Suite 225, Washington, D.C.

AASHTO (1995). "Report of The Subcommittee on Truck Size and Weight of The AASHTO Joint Committee on Domestic Freight Policy." An AASHTO document developed within the special committee on Intermodal Issues and Domestic Freight Policy with commentary by AASHTO Member Departments and Committees, *American Association of State Highway and Transportation Officials*. Washington, D.C.

Backlund, R. E. and Gurver, J. E. (1990). "Heavy Trucks on Highways: An Important Pavement Issue." *Transportation Research Record*, 1272, Transportation Research Board, National Research Council, Washington, D. C.

Barron, C. J., Jessup, E. L., and Casavant, K. L. (1994*). "A Case Study of Motor Vehicles Violating Special Weight Permits in the State of Washington." Final Technical Report, Phase I, Research Project No. 13, *Washington State Transportation Commission, Department of Transportation*, Washington.

Bennett, C. J. (1921). "Load Limitations for Primary and Secondary Roads." *Public Roads*, Vol. 3, No. 34.

CBO (1978). "Highway Assistance Programs: A Historical Perspective." *Background paper*, Washington, D.C.

Ceran, T., Daniel, Mann, Jonson, Mendenhall, and Newman, R. B. (1995). "Maintenance Considerations in Highway Design." *National Cooperative Highway Research Program*, Report 349, Transportation Research Board, National Research Council, National Academy Press, Washington, D.C.

Csagoly, Hoolowka and Dorton (1978). "The True Behavior of Thin Concrete Bridge Slabs." *Transportation Research Record*, 664, Vol. 1, National Research Council, Washington, D.C.

Darter, M. I., Smith, K. D., and Peshkin, D. G. (1991). "Field-Calibrated Mechanistic-Empirical Models for Jointed Concrete Pavements." *Transportation Research Record*, 1307, Transportation Research Board, National Research Council, Washington, D.C.

Dicleli, M. and Bruneau, M. (1995). "Fatigue-Based Methodology for Managing Impact of Heavy-Permit Trucks On Steel Highway Bridges." *Journal of Structural Engineering*, ASCE, Vol. 121, No. 11.

DOT (1981). "An Investigation of Truck size and Weight Limit." Final Report, Department of Transportation, Washington, D.C.

Duffey, E. (1918). "New York Advocates Placing Reasonable Limits Upon Total Load of Motor Trucks." *Public Roads*, Vol. 1, No.2.

Fekpe, E. S. K., Clayton, A. .M., and Haas, R. C. G. (1995). "Evaluating Pavement Impacts of Truck Weight Limits and Enforcement Levels." *Transportation Research Record*, 1508, Transportation Research Board, National Research Council, Washington, D. C.

FHWA (1981). "A pavement Moisture Accelerated Distress Identification Manual." FHWA/RD-81/080, volume 2, *Federal Highway Administration*, US Department of Transportation, Washington, D.C.

Forsyth, R. A. (1993). "Pavement Structural Design Practices." *National Cooperative Highway Research Program*, Synthesis of Highway Practice 189, Transportation Research Board, National Research Council, Washington, D.C.

GAO (1991). "Transportation Infrastructure: Preserving the Nation's Investment in the Interstate Highway System." *Report to the Committee on Public Works and Transportation*, House of Representatives, General Accounting Office, Washington, D.C.

Gillespie, T. D., Karamihas, S. M., Sayers, M. W., Nasim, M. A., Hansen, W., and Ehsan, N. (1993). "Effects of Heavy-Vehicle Characteristics on Pavement Response and

Performance." *National Cooperative Highway Research Program,* Report 353, National Academy Press, Washington, D.C.

Hall, K. T., Darter, M. I., and Hall, J. P. (1991). "Performance of Asphalt Concrete Resurfacing of Jointed Reinforced Concrete Pavement on Illinois Interstate Highway System." *Transportation Research Record,* 1307, Transportation Research Board, National Research Council, Washington, D.C.

ICC (1941). "Federal Regulation of the Sizes and Weight of Motor Vehicles." 77th Congress, 1st Session, *House Document 354*, Washington , D.C.

James, R. W., Zimmerman, R. A., and Loper, J. H. (1988). "Effects of Repeated Heavy Loads on Highway Bridges." Texas Transportation Institute, Research Report, 462-1F, *Texas State Department of Highways and Public Transportation,* Austin, Texas, Laboratory Report 434.1, Lehigh University.

Kostem, C. N. (1985). "Overloading of Prestressed Concrete I-Beam Highway Bridges (Final Report 1977-1983)." *FHWA Report*: FHWA/PA-84/015.

Ksaibati, K., and Staigle, R. (1995), "Faulting Performance Modeling for Undoweled Plain Concrete Pavements." *Transportation Research Record,* 1482, Transportation Research Board, National Research Council, Washington, D.C.

Lee, K. W. and Peckham W. L. (1990). "Assessment of Damage Caused to Pavements in New England." *Transportation Research Record,* 1286, National Research Council, Washington, D.C.

Mcghee, K. H. (1995). "Design ,Construction, and Maintenance of PCC Pavement Joints." *National Cooperative Highway Research Program*, Synthesis of Highway Practice 211, Transportation Research Board, National Research Council, National Academy Press, Washington, D.C.

Moses, F. (1989). "Effect on Bridges of Alternative Truck Configurations and Weights." *Transportation Research Board*, Draft Final Report, National Research Council, Washington, D.C.

Moses, F., Schilling, C. G. and Raju, k. S. (1987). "Fatigue Evaluation Procedure for Steel Bridges." *National Cooperative Highway Research Program* 299, Transportation Research Board, National Research Council, Washington, D.C.

ODOT (1991). "State of Oklahoma County Roads Design Guidelines Manual." *Developed jointly by ODOT and the Association of County Commissioners of Oklahoma.*

ODOT (1994). "Total Road Mileage." Planning Division/Current Planning Branch; *Oklahoma Department of Transportation*, 200 N.E. 21st Street, Oklahoma City, Oklahoma 73105.

ODOT (1995a). "Highway Needs Study and Sufficiency Rating Report." Needs Study Section, Planning Division, Volume I, *Oklahoma Department of Transportation*, 200 N.E. 21st Street, Oklahoma City, Oklahoma 73105.

ODOT (1995b). "Oklahoma Traffic Characteristics Report." Analysis of Traffic Volumes and Vehicle Classification Data at Permanent Traffic Counter Locations, Plannimg Division, Traffic Studies Branch, *Oklahoma Department of Transportation*, 200 N.E. 21st Street, Oklahoma City, Oklahoma 73105.

ODOT (1996a). "Summary Bridge Report." Bridge Division, *Oklahoma Department of Transportation*, 200 N.E. 21st Street, Oklahoma City, Oklahoma 73105.

ODOT (1996b). "Personal Communication." Bridge Division, *Oklahoma Department of Transportation*, 200 N.E. 21st Street, Oklahoma City, Oklahoma 73105.

OECD (1988). "Heavy Trucks, Climate and Pavement Damage." *Road Transport Research*, Organization for Economic Co-operation and Development, OCED, Paris, France.

OS (1991). "Oklahoma Statutes 1991: Comprising All Laws of a General and Permanent Nature Including Laws and Amendments Passed by the First Extraordinary and First Regular Sessions of the Forty-Third Legislature, 1991." *Edited and Published Under the Direction of the Supreme Court*, Volume 3, Titles 27 to 56, Eminent Domain- Poor Persons, Oklahoma Statutes Annotated Classification, West Publishing CO., St. Paul, Minn.

Paterson, W. D. O. (1987). "Road Deterioration and Maintenance Effects." Models for Planning and Management, *A World Bank Publication*, The Johns Hopkins University press, Baltimore and London.

Reno, A. T. and Stowers, J. R. (1995). "Alternatives to motor fuel taxes for financing surface transportation improvements." *National Cooperative Highway Research Program*, Report 377. Transportation Research Board, National Research Council, Washington, D.C.

Sinha , K. C. and Fwa ,T. F. (1993). "Framework for Systematic Decision Making in Highway Maintenance Management." *Transportation Research Record*, 1388, National Research Council, Washington, D.C.

Sorensen, H. C., and Robledo, F. M. (1992). "Turner Truck Impact on Bridges." *Washington Department of Transportation*, Final report, Washington, D.C.

Southgate, H. F., Deen R. C., and Mayes, J. G. (1983). "Strain Energy Analysis of Pavement Designs for Heavy Trucks." *Transportation Research Record*, 949, National Research Council, Washington, D.C.

Terrell, R. D., and Bell, C. A. (1987). "Effects of permit and illegal overloads on pavements." *National Cooperative Highway Research Program*, Synthesis of Highway Practice, 131, Transportation Research Board, National Research Council, Washington, D.C.

Thompson, M. R. and Nauman, D. (1993). "Rutting Rate Analysis of the AASHTO Road Test Flexible Pavements." *Transportation Research Record*, 1384, National Research Council, Washington, D.C.

Tilly, G. P. (1978). "Fatigue Problem in Highway Bridges." *Transportation and Road Research Laboratory*, TRR No. 664, pp. 93-101.

TRB (1986). "Twin Trailer trucks." *Transportation Research Board*, Special Report 211, National Research Council, Washington, D.C.

TRB (1988). "A Look Ahead: Year 2020." *Proceeding of the Conference on Long-Range Trends and Requirements for the Nation's Highway and Public Transit Systems, Transportation Research Board*, Special Report 220, National Research Council, Washington, D.C.

TRB (1989). "Providing Access for Large Trucks." *Transportation Research Board*, Special Report 223, National Research Council, Washington, D.C.

TRB (1990a). "Truck Weight Limits: Issues and Option." *Transportation Research Board*, Special Report 225 National Research Council, Washington , D.C.

TRB (1990b). "New Trucks for Greater Productivity and Less road Wear." *Transportation Research Board*, Special Report 227, National Research Council, Washington, D.C.

USDOT (1982*). "Final Report on the Federal Highway Cost Allocation Study." Report to the Secretary of Transportation to the Congress, *US Department of Transportation*, Federal Highway Administration, Government Printing Office, Washington, D.C.

USDOT (1983*). "Working Paper on Alternatives to Tax on Use of Heavy Trucks." *US Department of Transportation*, Federal Highway Administration, Government Printing Office, Washington, D.C.

Walter, P. K. (1989). "Heavy vehicle evaluation for overload permits." *Transportation Research Record*, 1227, National Research Council, Washington, D.C.

Wright, P. H. (1996). "Highway Engineering." *John Willy & Sons, Inc*. New York, USA.

Zaghloul, S. and White T. (1993). "Use of a Three-Dimensional, Dynamic Finite Element Program for Analysis of Flexible Pavement." *Transportation Research Record*, 1388, National Research Council, Washington, D.C.

Zaghloul, S. M., White, T. D., and Kueczek, T. (1994a). "Evaluation of Heavy Load Damage on Concrete Pavements Using Three-Dimensional, Nonlinear Dynamic Analysis." *Transportation Research Record, 1449*, Transportation Research Board, National Research Council, Washington, D.C.

Zaghloul, S. M., White, T. D., Ramirez, J. A., White, D. W. and Prasad, N. (1994b). "Computerized Overload Permitting Procedure for Indiana." *Transportation Research Record,* 1448, Transportation Research Board, National Research Council, Washington, D.C.

* denotes cross-reference.

APPENDICES

APPENDIX A

LOAD EQUIVALENCY FACTORS FOR DIFFERENT AXLES

Load equivalency factors (LEF) of any axle load and axle configuration can be defined as the ratio of its number of repetitions to cause the same reduction in PSI as one application of an 18-kip single axle load. LEFs for different axle load are presented in Table A-1 through A-6 which are reproduced below from AASHTO (1986, pp. D-3, D-6, D-9).

Table A-1 Load Equivalency Factors for flexible pavements, single-axle and SN=5 (AASHTO 1986, pp. D-3, D-6, D-9).

Axle load (kips)	Terminal serviceability (p_t)		
	2.0	2.5	3.0
2	0.0002	0.0002	0.0002
4	0.0029	0.0020	0.0020
6	0.0090	0.1000	0.0120
8	0.0310	0.0340	0.4000
10	0.0790	0.0880	0.1010
12	0.1740	0.1890	0.2120
14	0.338	0.3600	0.3910
16	0.6030	0.6230	0.6510
18	1.0000	1.0000	1.0000
20	1.5700	1.5100	1.4400
22	2.3500	2.1800	1.9700
24	3.4000	3.0300	2.6000
26	4.7700	4.0900	3.3300
28	6.5200	5.3900	4.1700
30	8.7000	7.0000	5.1000
32	11.5000	8.9000	6.3000
34	14.9000	11.2000	7.6000
36	19.0000	13.9000	9.1000
38	24.0000	17.2000	11.0000
40	30.0000	21.1000	13.1000
42	37.2000	25.6000	15.5000
44	45.7000	31.0000	18.4000
46	55.7000	37.2000	21.6000
48	67.3000	44.5000	25.4000
50	81.0000	53.0000	30.0000

Table A-2 Load Equivalency Factors for flexible pavements, tandem-axle and SN=5 (AASHTO 1986, pp. D-4, D-7, D-10).

Axle load (kips)	Terminal Serviceability (p_t) 2.0	2.5	3.0
2	0.0000	0.0000	0.0000
4	0.0002	0.0003	0.0000
6	0.0010	0.0010	0.0010
8	0.0030	0.0030	0.0030
10	0.0060	0.0070	0.0080
12	0.0130	0.0140	0.0170
14	0.0240	0.0270	0.0320
16	0.0420	0.0470	0.0550
18	0.0690	0.0770	0.0900
20	0.1090	0.1210	0.1390
22	0.1640	0.1800	0.2050
24	0.2390	0.2600	0.2920
26	0.3380	0.3640	0.4020
28	0.4660	0.4950	0.5380
30	0.6270	0.6580	0.7020
32	0.8290	0.8570	0.8960
34	1.0800	1.0900	1.1200
36	1.3800	1.3800	1.3800
38	1.7300	1.7000	1.6600
40	2.1600	2.0800	1.9800
42	2.6600	2.5100	2.3300
44	3.2400	3.0000	2.7100
46	3.9100	3.5500	3.1300
48	4.6800	4.1700	3.5700
50	5.5600	4.8600	4.0500
52	6.5600	5.6300	4.5700
54	7.6900	6.4700	5.1300
56	9.0000	7.4000	5.7000
58	10.4000	8.4000	6.4000
60	12.0000	9.6000	7.1000
62	13.8000	10.8000	7.8000
64	15.8000	12.2000	8.6000
66	18.0000	13.7000	9.5000
68	20.5000	15.4000	10.5000
70	23.2000	17.2000	11.5000
72	26.2000	19.2000	12.6000
74	29.4000	21.3000	13.8000
76	33.1000	23.7000	15.1000
78	37.0000	26.2000	16.5000
80	41.3000	29.0000	18.0000
82	46.0000	32.0000	19.6000
84	51.2000	35.3000	21.3000
86	56.8000	38.8000	23.2000
88	62.8000	42.6000	25.2000
90	69.4000	46.8000	27.4000

Table A-3 Load Equivalency Factors for flexible pavements, tridem-axle and SN=5 (AASHTO 1986, pp. D-5, D-8, D-11).

Axle load (kips)	Terminal Serviceability (p_t) 2.0	2.5	3.0
2	0.0000	0.0000	0.0000
4	0.0001	0.0001	0.0001
6	0.0003	0.0003	0.0003
8	0.0007	0.0010	0.0010
10	0.0020	0.0020	0.0020
12	0.0030	0.0030	0.0040
14	0.0060	0.0060	0.0070
16	0.0090	0.0110	0.0130
18	0.0150	0.0170	0.0200
20	0.0240	0.0270	0.0310
22	0.0350	0.0400	0.0460
24	0.0510	0.0570	0.0660
26	0.0710	0.0800	0.0920
28	0.0980	0.1090	0.1260
30	0.1310	0.1450	0.1670
32	0.1730	0.1910	0.2180
34	0.2250	0.2460	0.2790
36	0.2880	0.3130	0.3520
38	0.3640	0.3930	0.4370
40	0.4540	0.4870	0.5360
42	0.5610	0.5970	0.6490
44	0.6860	0.7230	0.7770
46	0.8310	0.8680	0.9200
48	0.9990	1.0330	1.0800
50	1.1900	1.2200	1.2600
52	1.4100	1.4300	1.4500
54	1.6600	1.6600	1.6600
56	1.9400	1.9100	1.8800
58	2.2500	2.2000	2.1300
60	2.6000	2.5100	2.3900
62	2.9900	2.8500	2.6600
64	3.4200	3.2200	2.9600
66	3.9000	3.6200	3.2700
68	4.4200	4.0500	3.6000
70	5.0000	4.5200	3.9400
72	5.6300	5.0300	4.3100
74	6.3300	5.5700	4.6900
76	7.0800	6.1500	5.0900
78	7.9000	6.7800	5.5100
80	8.7900	7.4500	5.9600
82	9.8000	8.2000	6.4000
84	10.8000	8.9000	6.9000
86	11.9000	9.8000	7.4000
88	13.2000	10.6000	8.0000
90	14.5000	11.6000	8.5000

Table A-4 Load Equivalency Factors for rigid pavements, single-axle and D=10 inch (AASHTO 1986, pp. D-12, D-15, D-18)

Axle load (kips)	Terminal serviceability (p_t)		
	2.0	2.5	3.0
2	0.0002	0.0002	0.0002
4	0.0020	0.0020	0.0020
6	0.0100	0.0200	0.0100
8	0.0320	0.0320	0.0320
10	0.0800	0.0810	0.0810
12	0.1740	0.1750	0.1760
14	0.3370	0.3380	0.3400
16	0.5990	0.6010	0.6000
18	1.0000	1.0000	1.0000
20	1.5800	1.5800	1.5700
22	2.4000	2.3800	2.3500
24	3.5100	3.4500	3.3800
26	4.9700	4.8500	4.7000
28	6.8500	6.6100	6.3100
30	9.2300	8.7900	8.2500
32	12.2000	11.4000	10.5400
34	15.8000	14.6000	13.2000
36	20.1000	18.3000	16.2000
38	25.4000	22.7000	19.8000
40	31.6000	27.9000	23.7000
42	38.9000	34.0000	28.5000
44	47.6000	41.0000	33.9000
46	57.7000	49.2000	40.1000
48	69.4000	58.7000	47.3000
50	83.0000	69.6000	55.6000

Table A-5 Load Equivalency Factors for rigid pavements, tandem-axle and D=10 inch (AASHTO 1986, pp. D-13, D-16, D-19)

Axle load (kips)	Terminal Serviceability (p_t)		
	2.0	2.5	3.0
2	0.0001	0.0001	0.0001
4	0.0005	0.0005	0.0005
6	0.0020	0.0020	0.0020
8	0.0050	0.0050	0.0050
10	0.0120	0.0120	0.01300
12	0.0250	0.0250	0.02600
14	0.0470	0.0470	0.04700
16	0.0810	0.0810	0.08100
18	0.1310	0.1320	0.1320
20	0.2030	0.2040	0.2050
22	0.3040	0.3050	0.3070
24	0.4400	0.4410	0.4430
26	0.6180	0.6200	0.6210
28	0.8500	0.8500	0.8500
30	1.1400	1.1400	1.1400
32	1.5100	1.5000	1.4900
34	1.9600	1.9500	1.9300
36	2.5100	2.4800	2.4500
38	3.1700	3.1200	3.0600
40	3.9500	3.8700	3.7600
42	4.8700	4.7400	4.5800
44	5.9500	5.7500	5.5000
46	7.2000	6.9000	6.5400
48	8.6300	8.2100	7.6900
50	10.2700	9.6800	8.9600
52	12.1000	11.3000	10.3600
54	14.2000	13.2000	11.9000
56	16.6000	15.2000	13.6000
58	19.3000	17.5000	15.4000
60	22.3000	20.0000	17.4000
62	25.6000	22.8000	19.6000
64	29.3000	25.8000	22.0000
66	33.4000	29.2000	24.6000
68	37.9000	32.9000	27.4000
70	42.9000	37.0000	30.6000
72	48.5000	41.5000	34.0000
74	54.6000	46.4000	37.7000
76	61.2000	51.8000	41.8000
78	68.8000	57.7000	46.3000
80	76.6000	64.2000	51.1000
82	85.3000	71.2000	56.5000
84	95.0000	78.9000	62.2000
86	105.0000	87.0000	68.5000
88	116.0000	96.0000	75.3000
90	129.0000	106.0000	83.0000

Table A-6 Load Equivalency Factors for rigid pavements, tridem-axle and D=10 inch (AASHTO 1986, pp. D-14, D-17, D-20)

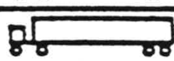
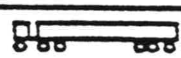
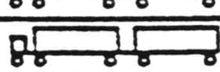
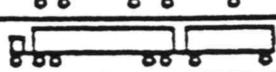
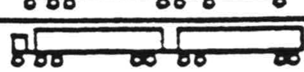
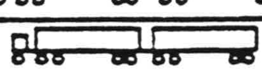
Axle load (kips)	Terminal Serviceability (p_t) 2.0	2.5	3.0
2	0.0001	0.0001	0.0001
4	0.0003	0.0003	0.0003
6	0.0009	0.0010	0.0010
8	0.0020	0.0020	0.0020
10	0.0050	0.0050	0.0050
12	0.0090	0.0090	0.0090
14	0.0160	0.0160	0.0170
16	0.0270	0.0270	0.0280
18	0.0430	0.0440	0.0440
20	0.0660	0.0660	0.0670
22	0.0970	0.0980	0.0980
24	0.1390	0.1390	0.1400
26	0.1930	0.1940	0.1950
28	0.2620	0.2630	0.2650
30	0.3500	0.3510	0.3530
32	0.4590	0.4600	0.4620
34	0.5930	0.5940	0.5950
36	0.7550	0.7560	0.7560
38	0.9510	0.9500	0.9490
40	1.1800	1.1800	1.1800
42	1.4600	1.4500	1.4400
44	1.7800	1.7700	1.7500
46	2.1500	2.1300	2.1000
48	2.5800	2.5500	2.5100
50	3.0700	3.0200	2.9600
52	3.6300	3.5600	3.4700
54	4.2700	4.1600	4.0300
56	4.9900	4.8400	4.6500
58	5.7900	5.5900	5.3400
60	6.6900	6.4200	6.0800
62	7.6900	7.3300	6.8900
64	8.8000	8.3300	7.7600
66	10.0200	9.4200	8.7000
68	11.4000	10.6000	9.7100
70	12.8000	11.9000	10.8000
72	14.5000	13.3000	12.0000
74	16.2000	14.8000	13.2000
76	18.2000	16.5000	14.5000
78	20.3000	18.2000	15.9000
80	22.6000	20.2000	17.4000
82	25.0000	22.2000	19.1000
84	27.7000	24.5000	20.8000
86	30.7000	26.9000	22.6000
88	33.8000	29.4000	24.6000
90	37.2000	32.2000	26.8000

APPENDIX B

COMMON TRUCK WEIGHTS AND DIMENSIONS

To determine the relative pavement damage (fatigue and rutting) over a range of trucks, fifteen truck configurations are considered. The GVW along with axle loads distribution of each truck type is presented in Table B-1. Approximately 30% of the registered heavy vehicles in the United States are tractor-semi-trailers and are responsible for about 70% of the heavy-truck highway mileage (Gillespie et al. 1993 pp. D-18).

Table B-1 Size and Weight Distribution of Different Truck Types (Gillespie et al. 1993 pp. D-16)

Truck Configuration	Configuration Name	GVW (kips)	Axle Load (kips)
	2-Axle Straight Truck	32	12/20
	3-Axle Straight Truck	46	12/34
	3-Axle Refuse Hauler	64	20/44
	4-Axle Concrete Mixture	68	18/38/12
	3-AxleTractor-Semitrailer	52	12/20/20
	4-AxleTractor-Semitrailer	66	12/20/34
	5-AxleTractor-Semitrailer	80	12/34/34
	5-AxleTractor-Semitrailer	80	12/33/33
	5-Axle Tanker	80	12/34/34
	6-Axle Tanker	85	12/34/39
	5-Axle Doubles	80	10/18/17/18/17
	5-Axle Doubles	80	10/20/15/20/15
	7-Axle Doubles	120	12/34/34/20/20
	9-Axle Doubles	140	12/32/32/32/32
	Turner Doubles	114	10/26/26/26/26