AUBURN

SAMUEL GINN
COLLEGE OF ENGINEERING

CORRELATION BETWEEN TRUCK WEIGHT, HIGHWAY INFRASTRUCTURE DAMAGE AND COST

Final Report

Submitted to
Federal Highway Administration
400 7th St SW
Washington, D.C. 20590

In fulfillment of:
DTFH61-05-Q-00317

Subject No.
70-71-5048

Prepared by
Dr. David H. Timm, P.E.
Gottlieb Associate Professor of Civil Engineering
email: timmdav@eng.auburn.edu

Dr. Rod E. Turochy, P.E.
Associate Professor of Civil Engineering
email: rturochy@eng.auburn.edu

Kendra D. Peters
Civil Engineering Research Assistant
email: peterkd@auburn.edu

October 29, 2007

Highway Research Center
Harbert Engineering Center
Auburn, Alabama 36849

www.eng.auburn.edu/research/centers/hrc.html

CORRELATION BETWEEN TRUCK WEIGHT, HIGHWAY INFRASTRUCTURE DAMAGE AND COST

Final Report

Submitted to
Federal Highway Administration
400 7th St SW
Washington, D.C. 20590

In fulfillment of:
DTFH61-05-Q-00317

Subject No.
70-71-5048

Prepared by
Dr. David H. Timm, P.E.
Gottlieb Associate Professor of Civil Engineering
Auburn University
Tel: 334-844-6282
email: timmdav@eng.auburn.edu

Dr. Rod E. Turochy, P.E.
Associate Professor of Civil Engineering
Auburn University
Tel: 334-844-6271
email: rturochy@eng.auburn.edu

Kendra D. Peters
Civil Engineering Research Assistant
Auburn University
Tel: 334-844-6273
email: peterkd@auburn.edu

October 29, 2007

AUBURN UNIVERSITY
HIGHWAY RESEARCH CENTER
238 HARBERT ENGINEERING CENTER
AUBURN, AL 36849-5337
334-844-5250

ACKNOWLEDGEMENTS

This research was conducted through the Highway Research Center at Auburn University under the direction and sponsorship of the Office of Freight Management and Operations at the Federal Highway Administration. The authors wish to specifically acknowledge Tom Kearney, Bill Mahorney, Mike Onder and Julie Strawhorn of FHWA for their support and cooperation on this project. The collective efforts of state transportation agency employees who completed and returned surveys as part of this project are also appreciated.

EXECUTIVE SUMMARY

The national highway infrastructure is under constant pressure to carry heavier trucks more frequently. According to the Federal Highway Administration (FHWA), from 1997 to 2006, state issuance of overweight permits exceeding Federal and grandfathered state limits have grown by 13%. In 2003, over 3 million "routine" permits were issued. These permits did not require anything beyond filing paperwork and paying appropriate fees for the permit to be issued. These permits typically allow a carrier to operate at some designated heavier axle or gross vehicle weight.

In addition to routine permitting, there has also been recent discussion of raising federal legal limits to allow for heavier trucks. Since 1974, the federal legal weight limit for trucks on interstate highways in the United States has been 80,000 lbs gross vehicle weight (GVW). The trucking industry is advocating increasing that limit to improve fleet efficiency, and with good reason. The Department of Transportation has estimated that freight movement will increase 87% over the next 20 years. This will greatly challenge our existing infrastructure and alternative approaches to carrying freight should be considered.

At the spring 2006 meeting of the American Association of State Highway and Transportation Officials (AASHTO), the American Trucking Association (ATA) proposed an increase of the GVW limit to 97,000 lbs. In order to satisfy the "federal bridge formula", this would require an additional axle at the trailer rear instead of the typical tandem axle set. The new proposed weight limit would potentially reduce congestion, improve fuel economy and save shippers and motor carriers billions of dollars annually. Specifically, research conducted by Americans for Safe and Efficient

Transportation (ASET)[1] showed that there could be an estimated 10 million fewer miles traveled per year by large trucks, a savings of 1.9 billion gallons of fuel and $15 billion (U.S.) saved in annual shipping costs. In addition to these benefits, the United States would have truck dimensions in harmony with those of Mexico and Canada in accordance with the North American Free Trade Agreement (NAFTA). Conversely, there are several arguments against the weight increase, including decreased safety, loss of profit from the trucking industry, and damage to highway infrastructure. Although there are practical arguments on either side of the first two issues, the third issue could be resolved by analyzing the actual pavement impacts that this change would create.

While the demands on the highway infrastructure are generally well known, there is not currently a widely-applied approach to assessing the impact of these demands on the highway pavement infrastructure in terms of accelerated damage and cost. Therefore, the objectives of this study were to:

1. Identify practices that are proven effective to assess highway infrastructure damage caused by overweight trucks.
2. Identify state-utilized corrective measures to ensure compliance with load regulations.
3. Establish a method by which to measure highway infrastructure damage, and associated cost, caused by overweight trucks.

These objectives were accomplished by first conducting a comprehensive literature review investigating various approaches to computing infrastructure damage and cost. Then, a survey of state departments of transportation evaluated current practice to permit and regulate truck weights. Finally, an analysis framework was developed that

[1] ASET, "Congress Can Encourage Safer and More Efficient Trucks," Americans for Safe and Efficient Transportation, http://www.aset-safety.org/congress/aset4.html, accessed October 29, 2007.

combined the Mechanistic-Empirical Pavement Design Guide (MEPDG) and life cycle cost analysis to determine pavement damage and cost. The framework was demonstrated through a number of alternative loading scenarios that included shifting entire weight distributions, permitting specific axles and considering increasing the legal limit to 97,000 lb as proposed by the American Trucking Association.

The literature review began by examining current weight regulations from both a state and federal perspective. It was found that weight regulations are highly state specific and that many states have legal limits exceeding federal limits for GVW, single and tandem axles, respectively. Most states also issue so-called "Routine Permits" that require no special investigation, but only payment of a permitting fee to operate at a heavier weight. It was also found that GVW and axle weights are considered separately in permitting.

Regarding permitting fees, it was found that the fee structure has not historically been damage-based. Despite general knowledge of significant costs to the transportation infrastructure due to heavy loads, permitting fees are usually established to recover administrative costs only and are often set by state legislatures.

An investigation of the number of overloads indicated that they are significant on a nationwide basis. Current estimates of overloads, from government agencies, range from 15-30% of all trucks. While this is significant, one study conducted in New York surveyed trucking companies and found a 50% overload percentage, which was deemed an unconservative estimate since trucking companies may not wish to reveal operations in excess of legal limits. Therefore, the actual percentage may be even higher. Clearly, more well-defined information regarding the occurrence of overloads is warranted.

Pavement damage assessment due to overloads has traditionally been based upon an empirical, equivalent single axle load (ESAL), approach or a mechanistic-empirical (M-E) approach. Using the ESAL approach, agencies have accounted for increased pavement damage and faster deterioration rates by computing the relative damaging effect of the heavier loads. The M-E approach has relied upon mechanistic modeling of typical or specific pavement sections with heavier loads to predict the increased rate of deterioration. Both approaches have been used with reasonable success. Given that pavement engineering is progressing toward mechanistic-empirical pavement design, using an M-E approach is considered state-of-the-art.

The last portion of the literature review focused upon cost analysis. It was found that standard economic models, whereby the net present value of various scenarios can be evaluated, were common. As part of any economic analysis, it is important to also weight the potential economic benefits of increased weight limits due to more efficient trucking operations.

To document current practices regarding overweight vehicle permit fees, a questionnaire was sent to a targeted individual in each state, either with the state transportation department or highway patrol. Twenty-seven responses were received, for a response rate of 54%. The length of the survey, as well as the wide variety of expertise required to thoroughly complete it, are assumed to have affected the response rate – in many states, the effort required to complete the questionnaire required the participation of at least three individuals, representing the state's permit, pavement design, and bridge offices.

In most states, fees for overweight permits are based on some combination of gross vehicle weight, axle weight and configuration, and mileage. A fee structure based on groups of GVW values and mileage is most common (30% of states), while a fee structure based solely on mileage is next most common (26%). Other factors that occasionally affect permit fees are commodity type and trip duration. 44% of states noted that their fee structure is set by legislative act; however, since this information appeared incidentally or as supplements to completed questionnaires, and was not a specific question in the survey, it is suspected that the true figure is higher. In their conversations with the researchers, some survey respondents noted that they have little or no impact permit fees and that the fees themselves do not accurately reflect the cost of damage to the infrastructure associated with overweight vehicles. Most states (78%) now have online application processes. Regarding coordination among states, other than the New England group, which appears to be the only working example with substantial uniformity of regulations among states and permits recognized in multiple states, efforts to increase uniformity and simplify processes for permit applicants appear to consist mainly of discussions.

As part of the permit application process, most states (74%) conduct analyses of the impact of the subject vehicle on pavements, bridges, or both. Analysis approaches vary widely, but the Bridge Analysis Rating System (BARS) and a variety of agency-internal approaches are most common. Six states (22%) reported using or experimenting with mechanistic-empirical approaches, and most states report typically involving their bridge design units, and many the pavement design units. To reduce impact of overweight vehicles on the infrastructure, 48% of states noted that they may require alternate routing, axle configuration, or travel periods. Although sharing of data obtained through monitoring and enforcement programs with offices responsible for pavement and/or bridge design is common (59% of states), site visits to evaluate damage are rare. While 78% of states responded that they do *not* conduct field investigations of damage, interpretation of the comments provided by the other 22% of states responding to this question suggest that site-specific, field-level evaluations of infrastructure damage are very rare.

There is an incredible diversity among states with respect to factors affecting weight limits associated with climate and commodity type. For example, changes in legal weight limits for the spring thaw period are very common in the northern states. The variety of commodities specifically addressed in regulations (typically statutes) may reflect the importance of various industries to their respective state's economies.

Most states have an apparently small number of locations at which weights are regularly monitored, both for traffic monitoring purposes as well as for enforcement. Several states reported having no weigh-in-motion stations in traffic ("mainline" as opposed to "roadside"); however, it is suspected by the researchers that in most of these cases, the person or persons responding to the survey are not involved in traffic monitoring (more likely the responsibility of transportation planning unit rather than a permitting unit of a transportation agency). Therefore, the extent of traffic monitoring equipment deployed in the field, based solely on the survey results, is probably underestimated.

Based upon findings from the literature review and survey of states' practices, an infrastructure damage and cost analysis framework was developed. The framework utilized the newly-developed MEPDG to compute pavement damage under standard and alternative loading scenarios. Using the MEPDG, a pavement rehabilitation cycle (i.e., time until terminal level of pavement distress) can be determined under the standard loading configuration. The same pavement can then be simulated under the alternative loading scenario to estimate the change in rehabilitation cycle relative to the standard. Standard techniques in life cycle cost analysis (LCCA) can then be used to determine the ratio of cost under the alternative condition relative to the standard. This cost ratio was termed the "Cost Factor."

A number of loading scenarios were evaluated with the framework described above. The first involved shifting entire weight distributions toward heavier axle weights. The second investigated permitting specific axles, and the third evaluated changing the GVW from 80,000 lb to 97,000 by adding a third axle to a conventional tandem axle. All three scenarios examined both flexible and rigid pavements and considered a wide range of traffic volumes (250 to 8,000 trucks/day).

The first portion of the investigation showed that even small changes in the weight distribution (i.e., shifting axle weights by 3,000 lb) can have significant impacts on pavement damage and cost. A five year reduction in pavement performance and costs increasing by a factor of 1.5 to 2 were common for even this modest load increase.

The second portion of the investigation found that cost increases became significant when the volume of permitted axles exceeded 10% of the total number of legally loaded axles. Also, there did not appear to be large differences between flexible and rigid pavements in terms of reductions in pavement life or increasing cost factors due to increasing the number of permitted axles.

The third portion of the investigation found no practical difference in pavement damage between the 80,000 lb and 97,000 lb GVW loading scenarios when the weight increase is caused by an additional axle added to the rear tandem set. Mechanistic analyses were conducted outside of the MEPDG to confirm that only slight differences in pavement response (i.e., strain) existed between the tandem and triple axles. Therefore, there would not be any expected reduction in pavement life or increase in cost.

It must be noted that the above findings represent a limited number of conditions. One of the key findings was that though there were trends in the data, the results seemed to be somewhat pavement-specific. Therefore, a framework, rather than a single metric such as $/pound should be used to evaluate the effects of overloads on pavement damage and cost. Further recommendations include:

1. Identify and characterize other loading scenario conditions to be simulated with the MEPDG.
2. Explore the effect of changing analysis period and interest rate on cost factors.
3. Establish a methodology to more accurately predict changes in load spectra resulting from changes in legal limits, or from permitting heavier axles.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

BACKGROUND

Although truck size and weight legislation in the U.S. has existed for nearly a century, it was only in the early 1960s, at the American Association of State Highway Officials (AASHO) Road Test, that pavement engineers and researchers began to precisely quantify the effects of axle loads on pavement damage. The so-called fourth-power relationship between pavement damage and axle load has been well-documented since then and has been characterized by a change in pavement serviceability resulting from the application of a known number of equivalent single axle loads (ESALs). More recently, efforts have been made toward characterizing pavement damage in terms of actual distresses through mechanistic-empirical (M-E) pavement analysis techniques. This approach more robustly predicts pavement distress as a function of stress or strain levels within the pavement cross-section.

With advances in pavement design and distress analysis come greater opportunity to evaluate the impact of changes in truck weight policies and permitting processes. There is some interest in the effects of raising legal limits as well as maximum weights for routine permitting (weights for which a minimal review process is followed for the issuance of overweight permits) on roadway infrastructure. Additionally, a recent proposal from the freight industry would increase legal gross vehicle weight limits from 80,000 to 97,000 pounds on Interstate highways. The increased load would be carried by an additional axle in the trailer rear axle group (essentially, changing this axle group from a 34,000 pound tandem to a 51,000 pound tridem axle). The M-E pavement analysis process can be used to evaluate the impacts of such changes.

While damage assessment is an important component in overload analysis, assigning costs to the damage is equally critical. The costs of maintaining a roadway at some acceptable level of service can be estimated by applying life-cycle cost analysis concepts to the additional infrastructure damage associated with overweight vehicles, shifts in legal weight limits, and routine permit values.

As Federal and State agencies continue to review and develop their own legal limits, permitting and fining regulations in the context of pavement and bridge design, there is a need for guidance and recommendations concerning best practices to correlate infrastructure damage and resultant costs to overweight or exceptionally heavy vehicles.

A wide variety of factors, such as overweight permit regulations set by transportation agencies and state legislatures, types of products commonly hauled in a particular state, enforcement resources, as well as traffic patterns and climatological factors, affect the loading and routing of overweight vehicles. For these reasons, an equally wide range of procedures for issuing and managing these permits has developed among the states. Analyses such as those described above could potentially lead to increased uniformity in the permitting process and associated fees.

Many government agencies and the freight industry are stakeholders in determining infrastructure damage and associated costs of changes in weight limit policies. Therefore, the development of a framework to analyze the impacts of such changes, using M-E analysis techniques, is needed. However, the diversity of conditions to which roadways are subjected provides support for a framework that can ultimately be applied in a case-specific situation. Because of the variety of case-specific parameters, such as pavement structure and history, subgrade material quality, traffic loading, and climate, the impacts of changes in weight limits and axle loads as well as the impacts of individual heavy vehicles passes cannot be generalized. Rather than expressing the cost of overload damage in a single number (such as dollars per pound per mile), the analysis framework developed in this research can be used to determine appropriate cost factors for specific scenarios. These cost factors can be then used for a variety of purposes from budgeting tools for state transportation agencies to support for establishment of permitting fees to recover the costs associated with specific scenarios.

OBJECTIVES

This research project was undertaken to address the issue of infrastructure damage and associated costs. The main objectives were:

1. Identify practices that are proven effective to assess infrastructure damage caused by overweight trucks.
2. Identify state-utilized corrective measures to ensure compliance with load regulations.
3. Establish a method by which to measure infrastructure damage, and associated cost caused by overweight trucks.

SCOPE OF WORK

These objectives were accomplished through review of pertinent literature, a determination of current practices regarding the permitting process, and the development of a framework to analyze the impacts of changes in axle load distributions on pavement life and the associated life cycle costs required to maintain an acceptable level of pavement serviceability.

The literature review examined truck weights from both a state and federal perspective. Major pieces of legislation that have affected truck and axle weight limits were documented. The literature review also examined current legal limits and permitting procedures in addition to the incidence of overloads and enforcement practices. Pavement infrastructure damage assessment studies were studied, including both empirical and mechanistic-empirical approaches. Finally, methods of economic analysis in relation to truck weights and infrastructure damage were documented.

The current practice of state agencies was determined primarily through a survey and supplemented through follow-up communications and the findings of the literature review. The survey focused on permit processes and the basis of permit fees, techniques used for infrastructure damage analysis and assessment, and weight limits and enforcement.

To accomplish the third objective, a framework for conducting damage and cost assessments was developed. This framework included methods for quantifying pavement damage and associated cost. The framework was demonstrated by comparing a "baseline" case representing typical conditions on Interstate highways to hypothetical cases representing changes in the vehicle fleet. Such hypothetical changes included shifts in the distribution of axle weights, or load spectra, based on changes in weight limits, load spectra shifts based on changes in "routine permit" weights, and fleet changes based on the 97,000-pound configuration mentioned previously. For a limited set of hypothetical yet broadly representative cases, a knowledge base of damage and cost as a function of changes in load spectra was developed. These cases include combinations of variations in pavement type and thickness, subgrade soil strength, and traffic loads. The effects of these cases, using M-E pavement analysis principles (specifically the MEPDG software), on pavement life and associated life cycle costs were quantified. Pavement life was characterized by pavement distresses such as fatigue cracking and rutting reaching unacceptable levels.

The core of this report is organized into three chapters focusing on the literature review, current practice, and the development and application of the analysis framework. The analysis framework is applied to three types of scenarios: shifts in load spectra, routine permit limits, and the proposed 97,000-pound vehicle. Changes in pavement damage, service life, and life cycle costs are noted. Conclusions and recommendations follow these chapters.

CHAPTER 2

LITERATURE REVIEW

INTRODUCTION

This literature review examines truck weights from both a state and federal perspective. The review begins by highlighting major U.S. legislation that has affected truck and axle weight limits. A discussion is then provided regarding current legal limits and permitting procedures. The incidence of overloads and enforcement practices are then described. Pavement infrastructure damage assessment studies are presented, including both empirical and mechanistic-empirical approaches. The final section discusses methods of economic analysis in relation to truck weights and infrastructure damage.

HISTORY OF FEDERAL SIZE AND WEIGHT LEGISLATION

The origin of truck size and weight limits in the U.S. can be traced to 1913 when Maine, Washington D.C., Massachusetts and Pennsylvania established legal limits to protect pavements and bridges from premature distress (*1*). By 1933, each state had established its own set of weight limits that were highly state-specific. The state-limits were considered acceptable under a grandfather clause when the Federal-Aid Highway Legislation Act of 1956 established a maximum gross vehicle weight (GVW) of 73,280 lb (*1*). Through the same act, the federal government also established legal limits for single axles ($\leq$ 18,000 lb), tandem axles ($\leq$ 32,000 lb) and vehicle width ($\leq$ 96 in.) (*2*).

The federal legal limits were in effect for the next 19 years until the Federal-Aid Highway Amendment Act of 1974 increased the GVW to 80,000 lb. This measure was meant to improve fuel economy and was considered a trade-off for lowering the speed limit to 55 mph (*1*). In addition to the GVW increase, both single and tandem axle limits were increased by 2,000 lb (*2*). These weight limits have been in effect since 1974. However, prior to 1982, states could enforce lower legal limits if they chose which created a logistical problem for many trucking companies that were forced to follow lower legal limits in so-called "barrier states" along the Mississippi River that enforced a legal limit of 73,280 lb (*2*). Abdel Halim and Saccomanno (*3*) cited NCHRP Report 198 (*4*) which suggested that "failure to adopt consistent limits among states in the United States has imposed additional costs on both trucking operations and road administration."

To address the problem of non-uniform regulations on the interstate system, the federal government enacted the Surface Transportation Assistance Act (STAA) of 1982 (*2*). This act set uniform standards enforced on all Interstate routes with the above-mentioned weight limits. It also fixed size limits such that twin trailers were limited to 28 ft long by 102 in. wide and semi-trailers were limited to 48 ft long. The STAA also raised the federal fuel tax by five cents per gallon to offset the infrastructure costs. This tax increase was based upon the 1982 Federal Highway Cost Allocation Study which found that the trucking industry was not paying its share of pavement, bridge, and other infrastructure costs (*2*).

The most recent legislation regarding truck size and weight was implemented in 1991. The Intermodal Surface Transportation Efficiency Act (ISTEA) placed a freeze on using longer combination vehicles (LCVs), a popular vehicle type in the western U.S. (*2*). The freeze prevents states from expanding any LCV operations or designation of new routes that may support LCV usage (*2*). Despite the freeze, there is intense debate regarding the usage of LCV vehicles on a wider basis which prompted an investigation into the impacts of LCVs in a recent Federal Highway Administration (FHWA) report (*2*).

CURRENT WEIGHT LIMITS AND PERMITTING

Weight Limits

The current federal weight regulation for the interstate system limits GVW to 80,000 lb, single axle weights to 20,000 lb and tandem axle weights to 34,000 lb (*5*). While these limits apply to the Interstate system, each state has its own set of regulations pertaining to the non-Interstate system. Table 2.1, from Volume II of the Comprehensive Truck Size and Weight Study (*5*), summarizes truck weight limits and permitting limits for each state as of 1994. Even a cursory examination of the table, and the following thirty footnotes, gives a clear indication that the application of legal limits is highly state-specific. For example, only seven states (Arkansas, Iowa, Mississippi, Montana, Tennessee, Utah and Virginia) apply the federal limits statewide. However, even amongst these states, weight limits for routine permits are different between each state. Clearly, each state has developed its own set of regulations to meet the needs of the local economy and trucking industry.

Figure 2.1, derived from data in Table 2.1, summarizes legal GVW in states where the maximum on "Other Highways" within the state exceeds the Interstate limit. Eleven states fall in this category and the maximums range from 5% to 61% above the Interstate limit. In a similar manner, Figures 2.2 and 2.3 represent state maximums for single and tandem axles relative to Interstate limits, respectively. Figure 2.2 indicates that only six states exceed the limit and range from 10% to 12% above the Interstate limit. Figure 2.3 contains more states and a wider range of maximum tandem axle weights; 6% to 18% above the Interstate limit. It is interesting to note that there were not consistent trends within a state regarding vehicle and axle limits. For example, Nevada has the highest GVW limit, yet adheres to the Interstate limits for single and tandem axles. Another example is Alabama which has the lowest maximum GVW of the states that exceed the Interstate limit, follows the maximum federal limit for single axles, but allows tandem axles up to 18% above the federal limit.

It is also important to note that a number of states have Interstate and Other Highway limits that are equivalent, but higher than the federal-mandated limits. Connecticut, for example, limits single axles to 22,400 lb on both Interstate and Other Highway routes. Tandem axles in Connecticut are limited to 36,000 lb on both systems. New Mexico is an example where limits on GVW, single and tandem axles are consistent between both Interstate and Other Highways, but are higher than the nationwide limits.

Table 2.1 1994 Vehicle Weight Limits (5).

State	Gross Vehicle		Single Axle		Tandem Axle		FBF "B"		"Routine" Permit		
	"I"	Other Highways	"I"	Other Highways	"I"	Other Highways	"I"	Other Highways	GVW	Single Axle	Tandem Axle
Alabama	80	84	20	20	34	40	Yes	No-WT	110/150	22	44
Alaska	–	90(2)	–	20	--	38	---	Yes	88.6(2)/150	30	50
Arizona	80	80	20	20	34	34	Yes	No-WT	106.5(3)/250	28	46
Arkansas	80	80	20	20	34	34	Yes	Yes	102/134	20	40
California	80	80	20	20	34	34	Yes-mod	Yes-mod	119.8/4(5)	30	60
Colorado	80	85	20	20	36	40	Yes	No	127/164	27	50
Connecticut	80	80	22.4	22.4	36	36	Yes	Yes	120/160	22.4	NS
Delaware	80	80	20	20	34	40	Yes	No-WT	120/120	20	40
D.C.	80	80	22	22	38	38	Yes-mod	Yes-mod	155-248	31	62
Florida	80	80	22	22	44	44	Yes(6)	No-WT	112/172	27.5	55
Georgia	80	80	20.34	20.34	34(7)	37.34	Yes	Yes(6)	100/175	23	46
Hawaii	80.8	88	22.5	22.5	34	34	Yes	No – Case-by-case above normal limits			
Idaho	80	105.5	20	20	34	34	Yes	Yes – Case-by-case above normal limits			
Illinois	80	80(8)	20	20(9)	34	34(9)	Yes	Yes(9)	100/120	20	48
Indiana (10)	80	80	20	20	34	34	Yes	Yes	108/120	28	48
Iowa	80	80	20	20	34	34	Yes	Yes	100/160	20	40
Kansas	80	85.5	20	20	34	34	Yes	Yes	95/120	22	45
Kentucky	80	80(11)	20	20	34	34	Yes	Yes	96/140	24	48
Louisiana	80(12)	80(12)	20	22	34	37	Yes	No	108/120	24	48
Maine	80	80(13)	20(14)	22.4	34	38	Yes-mod	No	130/167	25	50
Maryland	80	80	20(15)	20(15)	34(15)	34(15)	Yes	Yes	110/110	30	60
Massachusetts	80	80	22.4	22.4	36	36	Yes	Yes	99/130	NS	NS
Michigan (16)	80	80	20	20	34	34	Yes	Yes	80/164	13	26
Minnesota	80	80(17)	20	18	34	34	Yes	Yes-mod	92/144	20	40
Mississippi	80	80	20	20	34	34	Yes	Yes	113/190	24	48
Missouri	80	80 (18)	20	20(18)	34	34(18)	Yes	Yes(18)	92/120	20	40

State	Gross Vehicle		Single Axle		Tandem Axle		FBF "B"		"Routine" Permit		
	"I"	Other Highways	"I"	Other Highways	"I"	Other Highways	"I"	Other Highways	GVW	Single Axle	Tandem Axle
Montana	80	80	20	20	34	34	Yes	Yes	105.5/126	20	48
Nebraska	80	95	20	20	34	34	Yes	Yes	99/110	20	40
Nevada	80	129(19)	20	20	34	34	Yes	Yes	110(20)/(21)	28	50.4
New Hampshire	80	80	20(15)	22.4	34(15)	36	Yes	No	130/150	25	50
New Jersey	80	80	22.4	22.4	34	34	Yes	No	100(22)/150(22)	25(22)	40(22)
New Mexico	86.4	86.4	21.6	21.6	34.32	34.32	Yes-mod	Yes-mod	104(23)/120	26	46
New York	80	80	20(24)	22.4	34(24)	36	Yes(24)	Yes(24)	100/150	25	42.5
North Carolina	80	80	20	20	38	38	Yes-mod	Yes-mod	94.5/122	25	50
North Dakota	80	105.5	20	20	34	34	Yes	Yes	103/136	20	45
Ohio	80	80	20	20	34	34	Yes	No	120/120	29	46
Oklahoma	80	90	20	20	34	34	Yes	Yes	95/140	20	40
Oregon	80	80	20	20	34	34	Yes/mod	Yes-mod	90/105.5	21.5	43
Pennsylvania	80	80	20(25)	20(25)	34(25)	34(25)	Yes(25)	Yes(25)	116/136	27	52
Rhode Island	80	80	22.4	22.4	36	36	Yes-mod	Yes-mod	104.8(21)	22.4	44.8
South Carolina	80	80	20	22	34(26)	39.6	Yes(26)	No	90/120	20	40
South Dakota	80	129(19)	20	20	34	34	Yes	Yes	116(27)/(21)	31	52
Tennessee	80	80	20	20	34	34	Yes	Yes	100/160	20	40
Texas	80	80	20	20	34	34	Yes-mod	Yes-mod	106.1(28)/200	25	48.125
Utah	80	80	20	20	34	34	Yes	Yes	100/123.5	20	40
Vermont	80	80	20	22.4	34	36	Yes	Yes	108(29)/120	24	48
Virginia	80	80	20	20	34	34	Yes	Yes	110/150	25	50
Washington	80	105.5	20	20	34	34	Yes	Yes	103/156	22	43
West Virginia	80	80(30)	20	20	34	34	Yes	Yes	104/110	20	45
Wisconsin	80	80	20	20	34	34	Yes-mod	Yes-mod	100/191	20	60
Wyoming	117	117	20	20	36	36	Yes	No	85/135	25	55

Table 2.1 Footnotes

NS...Not specified

WT...Weight table

(1) "Routine" Permit GVW: The first number (left) is the highest weight a 5-axle unit can gross before special (other than routine) review and analysis of an individual movement is required. The second number (right) is the highest gross weight any unit with sufficient axles can gross before special review is required.

(2) State rules allow the more restrictive of the FBF B or axle summation. The 5-axle "routine" permit value is estimated using a truck tractor-semitrailer with a 65' outer bridge (based on a 48' semitrailer).

(3) The 5-axle "routine" permit value is estimated using a truck tractor-semitrailer with two 5' tandems @ 47.25K each + a 12K steering axle.

(4) Estimate based on State weight table values for a 4' tandem (drive) @ 46.2K, a rear tandem at the 60K maximum, and a 12.5K steering axle.

(5) Maximum based on the number of axles in the combination.

(6) FBF applies if GVW exceeds 73.28K.

(7) If GVW is less than 73.28K, the tandem axle maximum is 40.68K.

(8) On Class III and non-designated highways the maximum is 73.28K.

(9) On non-designated highways the single axle maximum is 18K, the tandem axle maximum is 32K, and the Bridge formula does not apply.

(10) On the Indiana Toll Road the single axle maximum is 22.4K, the tandem axle maximum is 36K, and the maximum practical gross is 90K.

(11) The maximum gross weight on Class AA highways is 62K, on Class A highways 44K.

(12) 6- or 7-axle combinations are allowed 83.4K on the Interstate System, and 88K on other State highways.

(13) A 3-axle tractor hauling a tri-axle semitrailer has a maximum GVW of 90K.

(14) If the GVW is less than 73.28K, the single axle maximum is 22K.

(15) If the GVW is 73K or less, the single axle maximum is 22.4K, and the tandem axle maximum 36K.

(16) Federal axle, gross and Bridge formula limits apply to 5-axle combinations if the GVW is 80K or less. For other vehicles and GVWs over 80K other limits apply. State law sets axle weight controls which allow vehicles of legal overall length to gross a maximum of 164K.

(17) Most city, county and township roads are considered "9-Ton Routes" with a maximum gross vehicle of 73.28K.

(18) On highways other than Interstate, Primary, or other designated, the single axle maximum is 18K, the tandem axle maximum 32K, the Bridge formula is modified, and the GVW maximum is 73.28K.

(19) The maximum is directly controlled by the FBF. Given the State's length laws, the maximum practical gross is 129K.

(20) The 5-axle "routine" permit value is estimated using a truck tractor-semitrailer with a 12.5K steering axle, a 47.25K drive tandem (5' spacing from State weight table), and a 50.4K spread tandem (8' spacing from the State weight table).

(21) A determination is made on a case-by-case basis.

(22) All "routine" permit values are calculated using 10" wide tires and a maximum 800 pounds/inch of tire width loading value.
(23) The 5-axle "routine" permit value is estimated using a truck tractor-semitrailer with two 46K tandems + a 12K steering axle.
(24) If the GVW is less than 71K, the single axle maximum is 22.4K, the tandem axle maximum 36K, and a modified Bridge formula applies.
(25) If the GVW is 73.28K or less, the single axle maximum is 22.4K, the tandem axle maximum 36K, and the Bridge formula does not apply.
(26) If the GVW is 75.185K or less, the tandem axle maximum is 35.2K, and the Bridge formula does not apply.
(27) The 5-axle "routine" permit value is estimated using a truck tractor-semitrailer with two 52K tandems + a 12K steering axle.
(28) The 5-axle "routine" permit value is estimated using a truck tractor-semitrailer with a 13K steering axle, a 45K drive tandem, and a 48.125K spread tandem. Both tandem weight values are from the State weight chart.
(29) The 5-axle "routine" permit value is estimated using a truck tractor-semitrailer with two 48K tandems + a 12K steering axle.
(30) The maximum GVW on non-designated State highways is 73.5K, and on county roads 65K.

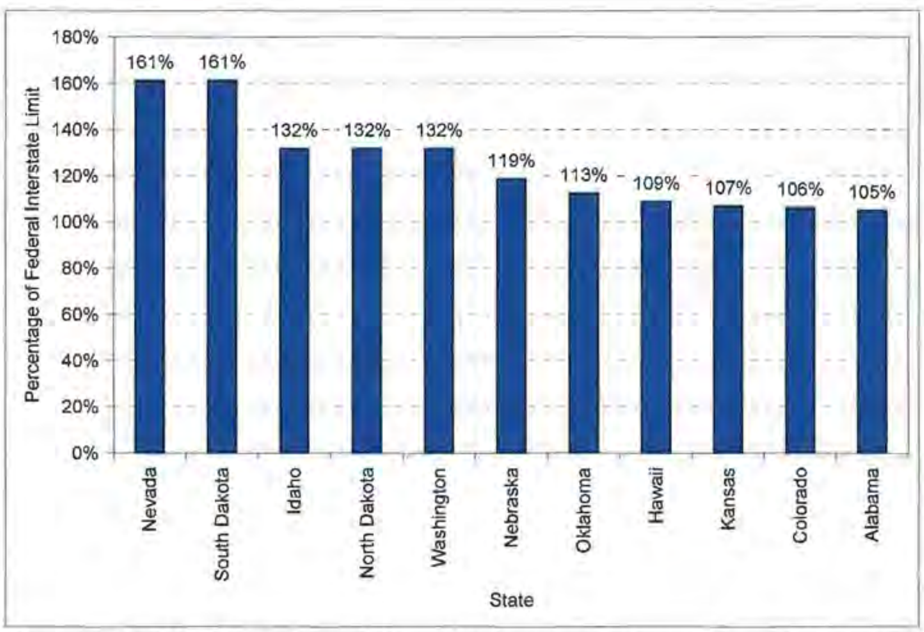


Figure 2.1 GVW Limit on Other Highways as Percentage of Interstate Limits (*Source: 5*).

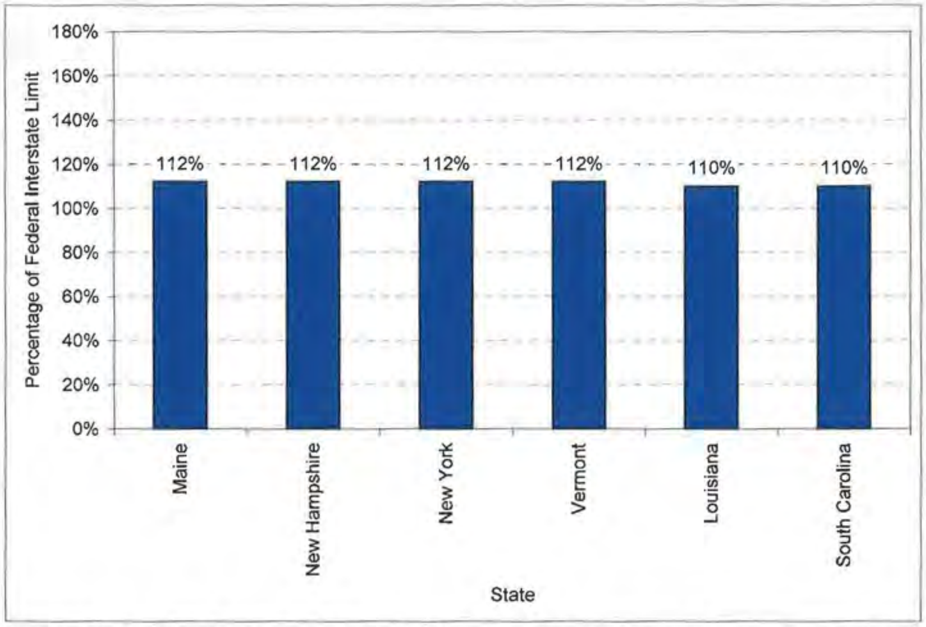


Figure 2.2 Single Axle Limit on Other Highways as Percentage of Interstate Limits (*Source: 5*).

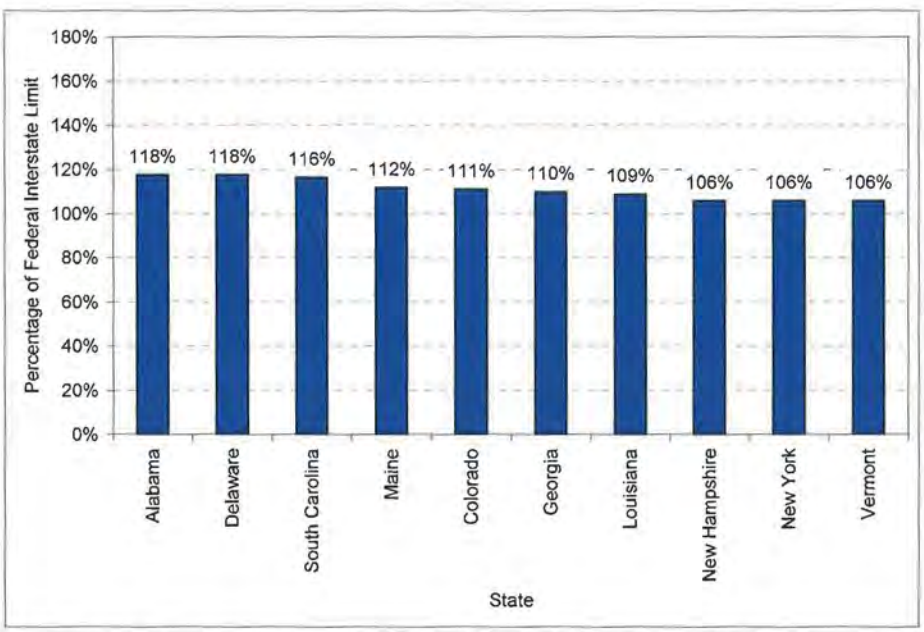


Figure 2.3 Tandem Axle Limit on Other Highways as Percentage of Interstate Limits (*Source: 5*).

Further examination of Table 2.1, and the accompanying footnotes, clearly indicate that each state has its own set of unique load limits and permitting regulations. Even within a particular state, there are regulations that are not uniformly applied. For example, Indiana Toll Roads allow a single axle maximum of 22,400 lb, a tandem axle maximum of 36,000 lb, and a maximum practical GVW of 90,000 lb. As a consequence, trucking companies have to meet the challenge of conforming to the respective weight laws whether operating in an inter- or intrastate mode.

There are more examples of unique state regulations to be found in other literature. The following is sample of state-specific regulations:

- Minnesota allows axle weights 10% above legal during winter months when the roads are frozen and less susceptible to load-induced damage (*6*).
- Idaho allows tandem axles up to 37,800 lb on non-interstate routes provided the GVW is less than 79,000 lb (*7*).
- New Jersey, while allowing single axle weights up to 22,400 lb on all routes, has a provision for trucks to register at lower GVW and pay a correspondingly lower registration fee (*8*).
- Maine has a provision for forest product and construction material hauling which allows up to 100,000 lb on 6-axle trucks (*6*).
- Montana allows B-train LCV's to operate at the Canadian legal limit of 137,800 lb on some routes (*6*).
- Michigan allows up to 164,000 lb GVW on 11 axles (*6*).
- Ohio allows trucks from Michigan, operating at Michigan legal limits, to utilize state roads to access the Ohio and Indiana turnpikes (*6*).

- New Hampshire allows 99,000 lb on 6 axles plus a 5% tolerance (*6*).

Texas has a well-documented load-zoning program. Within the state, there are approximately 17,250 miles of the 78,000 mile network subjected to load-zoning where the GVW limit is 58,240 lb (*9*). Built primarily in the 1950's, these routes are primarily "farm-to-market" and represent approximately 2/5 of the farm-to-market system (*10*). The limit is based upon bridge capacities and there are approximately 4,000 bridges on the farm-to-market system rated to the 58,240 lb maximum (*10*).

Routine Permitting

The data in Table 2.1 also indicate that every state has "Routine" permitting whereby trucking companies can apply and obtain a permit with no further investigation needed. Figure 2.4 summarizes these data as percentages above the federal limits on interstate highways for 5-axle GVW, other vehicle GVW, single axle weights and tandem axle weights. The data indicate that permitting for 5-axle GVW, single and tandem axles ranges predominantly from 10% to 50% above the federal maximums. Permitting for other vehicles GVW covered a much broader range with most in the 50% to 150% above the federal limits. It was unclear from the table presented in the Truck Size and Weight Report (*5*) why certain states listed the federal maximum as a "Routine Permitting" weight for single axles. It was also unclear why Wyoming's "Routine Permitting" weights were below their own maximums. These anomalies deserve further investigation.

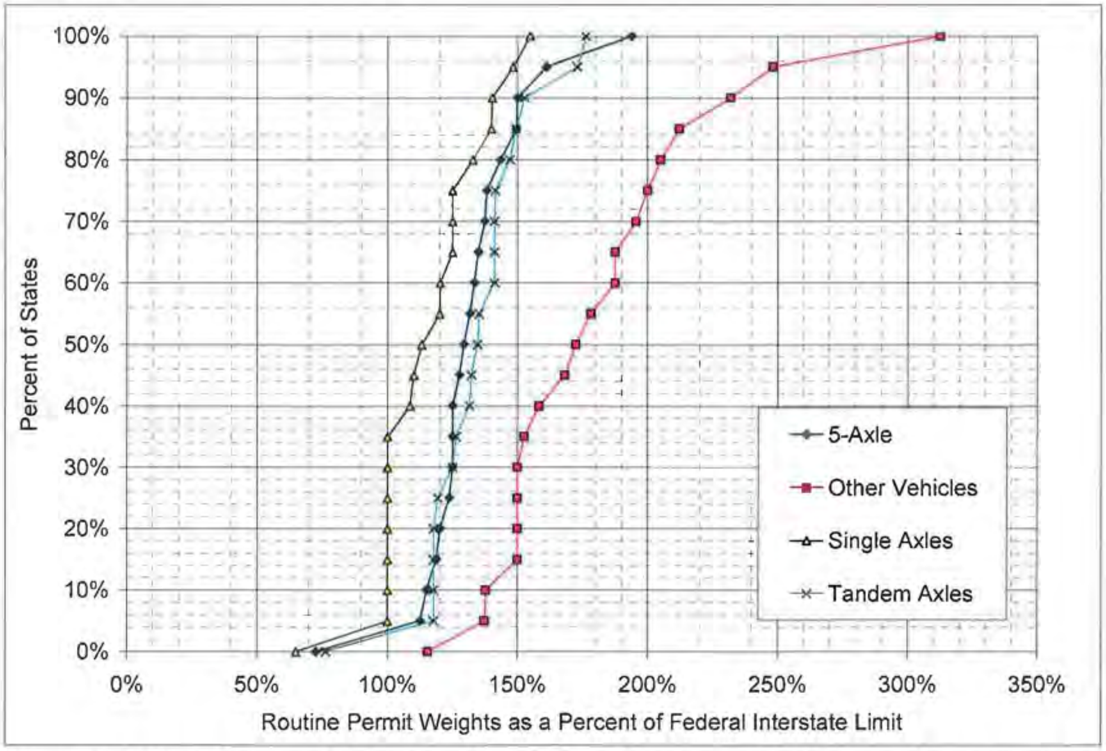


Figure 2.4 Distribution of Routine Permitting Maximum Weights as Percentage of Federal Limit on Interstates (*Source: 5*).

The data in Table 2.1 also appear to indicate that the permitted GVW, single and tandem axle weights are generally independent of one another. To illustrate this point, the single axle, tandem axle and other vehicle routine permitted weights were plotted against the permitted weights for 5-axle vehicles in Figure 2.5. Linear regression was performed on the data and the resulting equations and R^2 values are shown in the figure. There is generally poor correlation, as indicated by the low R^2, between these permitted weights. This seems to indicate that states consider GVW and axle weight limits separately in the context of permitting.

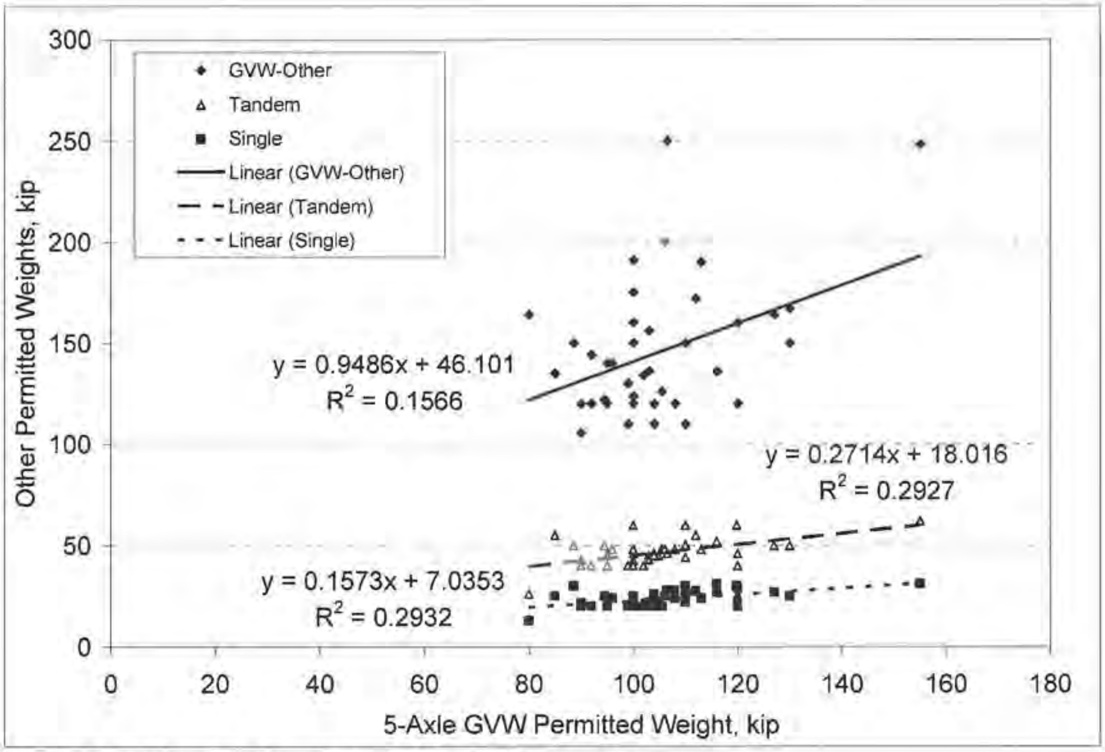


Figure 2.5 Routine Permitted Weight Comparison.

A study conducted by Straus and Semmens (*11*) quantified the number of permits issued in fiscal year 2003. While the total number of permits was reported, more useful quantities were the number of permits normalized by million vehicle-miles traveled (VMT) and normalized by the number of permits per 1,000 trucks. Figures 2.6 and 2.7 rank the states from fewest to greatest permits according to each method of normalization. If the two graphs are compared, it is apparent that the states are not necessarily ranked in the same order, however Figure 2.8 shows the strong correlation between the two methods so that either may serve as a good indicator of the number of permits in a particular state. In plotting the data, two outliers were noted; North Dakota and Washington, D.C. Straus and Semmens make no note of this in their paper; however, the deviation from the norm raises suspicion as to the validity and should be viewed with caution. Therefore, as shown in the figure, two linear regression analyses were performed. The regression that excludes these two outliers clearly indicates a strong correlation between the two methods of normalization.

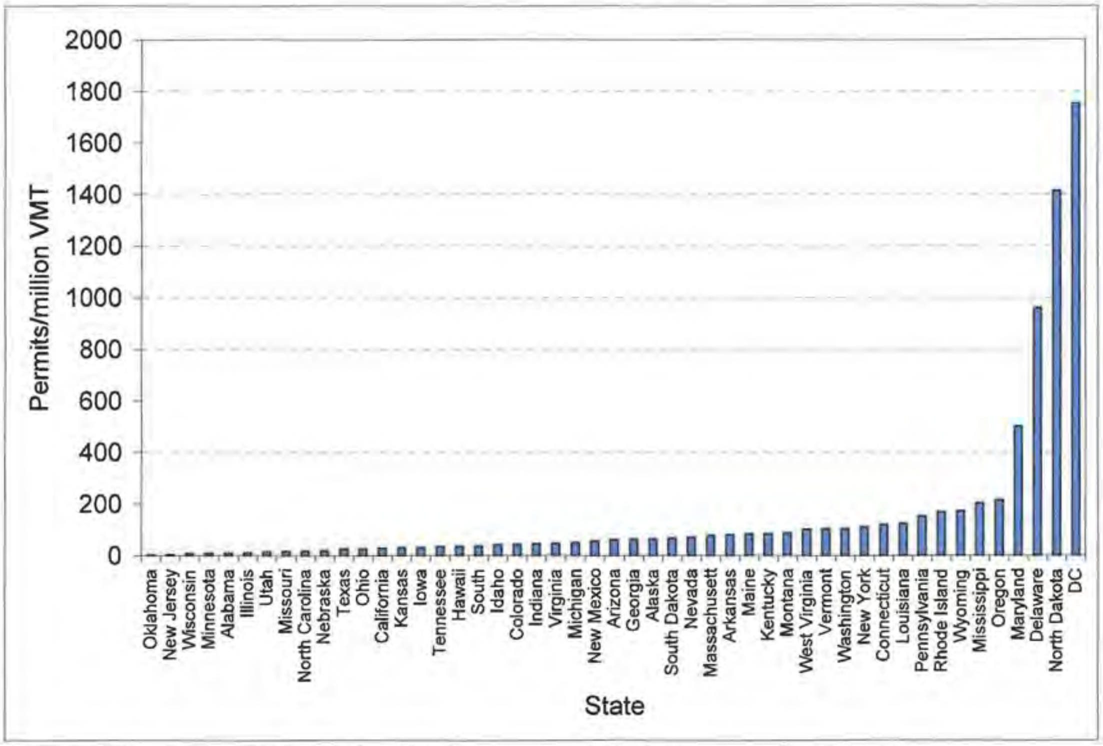


Figure 2.6 Permits per Million Vehicle Miles Traveled (*Source: 11*).

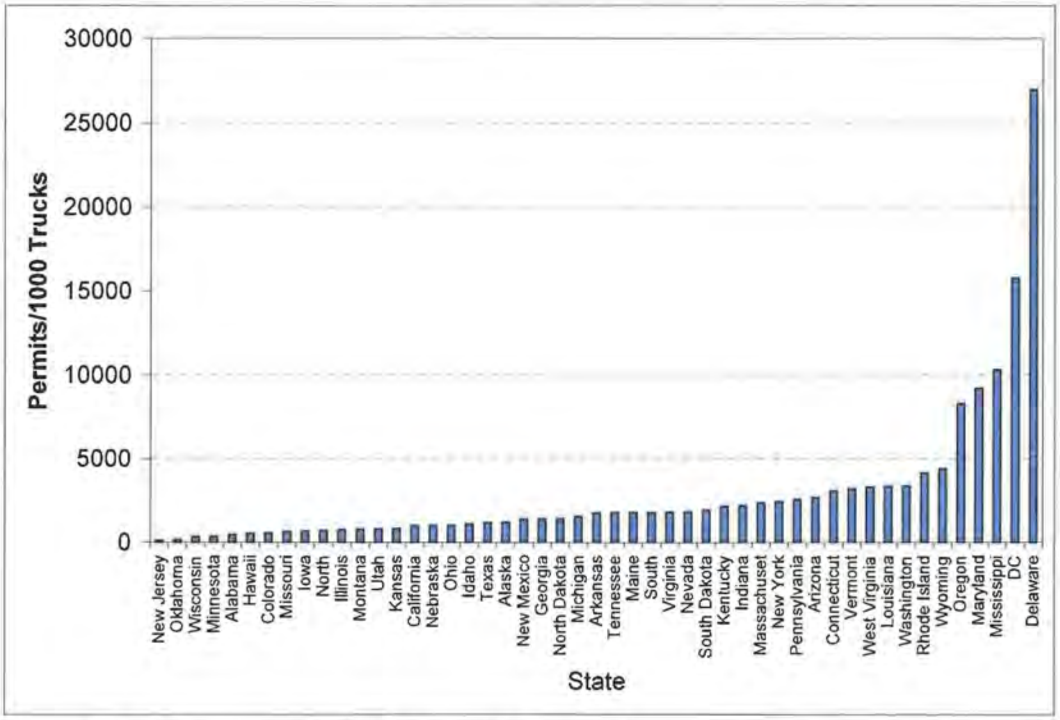


Figure 2.7 Permits per 1000 Trucks (*Source: 11*).

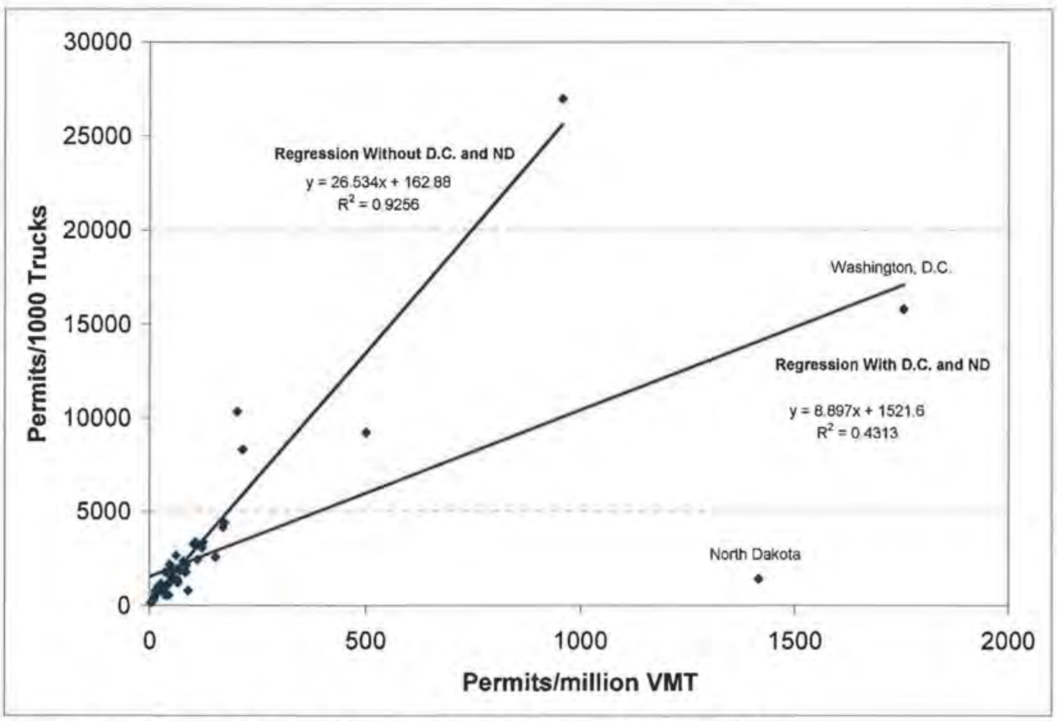


Figure 2.8 Permit Normalization Comparison (*Source: 11*).

An interesting comparison can be made between the number of permits issued as a function of state legal axle limits. One may expect that states adhering to the federal limit would need to grant more permits to meet the demands of the trucking industry. This is true, based upon GVW, as shown in Figure 2.9 where the average number of permits in states adhering to the federal GVW limit is more than double the average number of permits from other states. This was not the case for single axles, where the averages were approximately equal. More puzzling were the averages for the tandem axles. The average number of permits was nearly doubled for states where the state limit exceeded the federal limit when compared to states following the federal limit. This could be related to the legal limit ranges for GVW and tandem axles. Figure 2.1 showed the range of GVW legal limits ranging from 105% to 161% of the federal limit. In contrast, Figure 2.3 indicated a narrower range of 106% to 118% of the federal limit for tandem axles. It may be that the higher GVW limits reduce the need for permitting, while the less liberal tandem limits increase the need for permitting on a state-by-state basis.

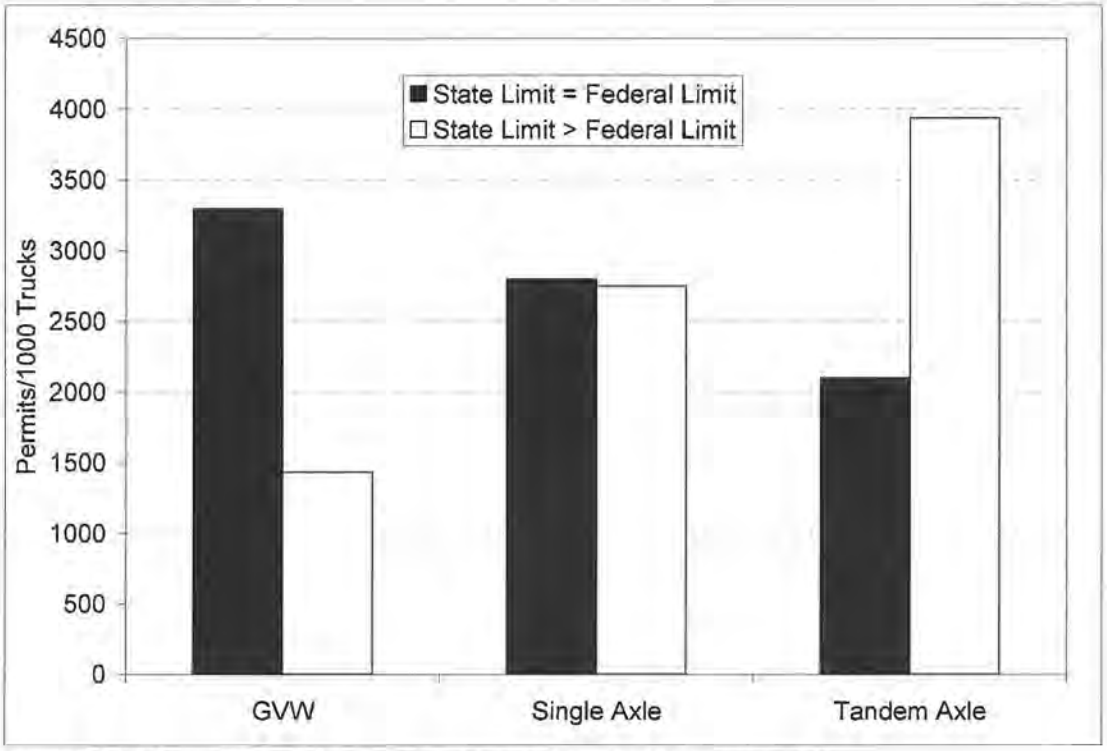


Figure 2.9 Permits as a Function of State Limits (*Source: 5, 11*).

Special Permitting

The above discussion has documented so-called "Routine" permitting. These permits require no special analysis prior to the state granting the permit. There are cases, however, where more detailed analysis is required. One such program is the super-heavy freight program in Texas (*12*). According to Texas Department of Transportation (TXDOT) policy, bridge analysis is required when the GVW exceeds 254,000 lb. Pavement analysis is required when the GVW exceeds 500,000 lb (*12*). The most common weights are 600,000 to 700,000 lb, so both bridge and pavement analysis are typically required. Though permits have been granted for distances ranging from 1 to 710 miles, 20 to 40 miles are most common (*12*). The goal of either analysis is to prevent structural failure of infrastructure due to the super-heavy move. In the event of failure, infrastructure damage becomes the responsibility of the carrier. TXDOT has an established procedure for determining the amount of infrastructure damage caused by these super-heavy moves. A detailed discussion will be included in the "Infrastructure Damage Assessment" of this literature review.

The number of super-heavy permits has grown over the past ten years in Texas. In 1994, Jooste and Fernando reported 60-70 permit requests per year (*13*). They cited primarily dragline components, oil pressure vessels and electric transformers among these requests. More recently, Chen et al. (*12*) reported 288 permits in 2001 and 364 permits in 2002. They reported approximately 78% of super-heavy moves were power plant related, 17% were oil-refinery related and 5% were mining related.

While the super-heavy permitting program applies to any type of carried freight, there are examples of states allowing permits to support particular industries. Louisiana is one such state that allows special permits for particular industries (*14*). Roberts and Djakfar (*14*) cited three particular permits; the agronomic/horticultural permit, the cotton module permit and the harvest season or natural forest products permit. Table 2.2 lists various permit types available in Louisiana, as of 1998 (*14*). The most unique permit is the cotton module permit that pertains to specially-designed vehicles that transport bailed cotton to the gin. This three-axle vehicle has replaced the traditional two-axle wagon commonly used in Louisiana to transport cotton from the field to the gin (*14*).

Table 2.2 Louisiana and Federal Statutes on Vehicle Weights for Selected Commodities, 1998 (*after 14*).

Commodity	Permit Type	Permit Applies to	Max GVW Limit, lb	Steering Axle Limit, lb	Single Axle Limit, lb	Tandem Axle Limit, lb
Sugarcane	Agronomic/	IH[a]	100,000	--	--	48,000
	Horticultural	Non-IH[b]	100,000	12,000	--	48,000
Rice	Agronomic/	IH	83,400	--	22,000	37,000
	Horticultural	Non-IH	100,000	12,000	--	48,000
Cotton	Cotton Module	IH	Not allowed	Not allowed	Not allowed	Not allowed
		Non-IH	68,000	--	20,000	48,000
Timber	Natural Forest	IH	83,400	--	20,000	35,200
	Products	Non-IH	86,600	--	20,000	40,000

[a]Interstate Highway
[b]Non-Interstate Highway

Permitting Fees

When reviewing the literature, many studies estimate the amount of infrastructure damage caused by overweight vehicles. The following are just a small sample of estimated costs:

- In 1987, Terrell and Bell reported annual costs due to overloaded trucks on the federal-aid highway system approaching \$1 billion/year (*15*).
- In 1995 FHWA estimated overloaded trucks in the U.S. cost taxpayers \$160-\$670 million/year in pavement damage (*11*).

Despite the general understanding that overloaded trucks can cause significant pavement damage, the fee structure has historically not been infrastructure damage-based (*5*). According to the Truck Size and Weight study, the associated permitting fees are usually established to recover the administration costs of the permitting program itself, including enforcement activities (*5*). The fees themselves have undergone little change from 1989 to 2000. In 1989, state permit fees for an 84,000 GVW vehicle ranged from \$6-\$61 (*5*).

As part of the Truck Size and Weight study, two states were cited as having a permitting fee structure that reflected the level of infrastructure damage (*5*). Minnesota, as depicted in Table 2.3, sets permitting fees on a per-mile basis as a function of the load level and calculated equivalent single axle load (ESAL) damage (*5*). The data have also been plotted in Figure 2.10 to illustrate the slightly non-linear relationship between fees and loading level. One would expect a more dramatic effect of load on permitting fees based on ESAL calculations since ESALs follow a fourth-power relationship between damage and load. It is also important to note the decreasing fees for a given axle group weight as the load is spread among more axles.

Table 2.3 Minnesota Overweight Axle Group Cost Factors ($/mile) Single Trip Permits (*5*).

Number of Pounds	2 Axles at 8 Feet Or Less	3 Axles at 9 Feet Or Less	4 Axles at 14 Feet Or Less
0 - 2,000 Pounds	0.12	0.05	0.04
2,001 - 4,000 Pounds	0.14	0.06	0.05
4,001 - 6,000 Pounds	0.18	0.07	0.06
6,001 - 8,000 Pounds	0.21	0.09	0.07
8,001 - 10,000 Pounds	0.26	0.1	0.08
10,001 - 12,000 Pounds	0.3	0.12	0.09
12,001 - 14,000 Pounds	Not Permitted	0.14	0.11
14,001 - 16,000 Pounds	Not Permitted	0.17	0.12
16,001 - 18,000 Pounds	Not Permitted	0.19	0.15
18,001 - 20,000 Pounds	Not Permitted	Not Permitted	0.16
20,001 - 22,000 Pounds	Not Permitted	Not Permitted	0.2

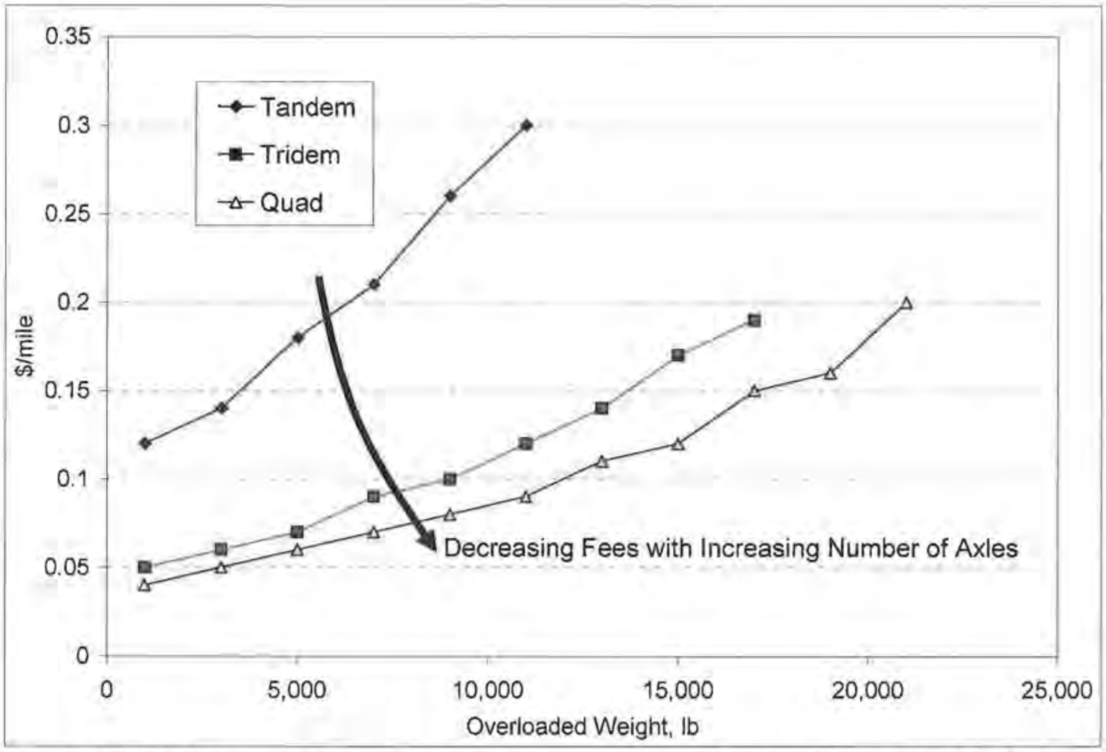


Figure 2.10 Minnesota Overweight Axle Group Cost Factors ($/mile) Single Trip Permits (*Source: 5*).

The other state cited by the Truck Size and Weight study was Washington (*5*). In 1995, legislation was passed to increase the per mile overweight permit fees to account for damage and administrative costs. The two-tiered fee structure in Washington includes a flat and per-mile fee. Before 1995, the per mile fee was capped at $2.80 for any weights exceeding 80,000 lb. After 1995, the fee structure changed to $2.82/mile for 80,000 to 100,000 lb and $4.25/mile for weights exceeding 100,000 lb. There is an additional fee of $0.50/mile for each additional 5,000 lb above 100,000 lb.

Perhaps the most unique permitting/taxing scheme can be found in Oregon which operates a so-called "weight-mile" tax. According to Rufolo et al. (*16*), the first objective of the system is to "apportion the cost of road construction and repair in an equitable manner among those who affect the standards needed for construction and cause for repair." The second objective is to "discourage the use of heavy axle loads, and thus reduce the wear and tear on the road system." The weight-mile tax reflects this philosophy by levying higher taxes on more heavily loaded trucks and by actively discouraging heavily loaded axles by offering tax incentives for using more axles per truck when the GVW exceeds 80,000 lb (*16*).

Within Oregon, the weight-mile tax applies to any trucks operating above 26,000 lb. Fuel taxes do not apply to these vehicles, while any vehicle under 26,000 lb is subjected to a fuel tax (*16*). All trucks wishing to operate above 26,000 lb must declare a maximum operating weight, and are subjected to a tax rate ($/mile) based only on the GVW up to 80,000 lb. Between 80,000 lb and 105,500 lb, the number of axles per truck in addition to the declared weight determine the tax rate. Within each weight class, the rate declines as the number of axles per vehicle increases. The rate reflects the expected reduction in road damage (*16*). The weight-mile tax allows up to 9 axles per vehicle.

The highest tax rate in Oregon applies to 5-axle vehicles operating at 98,000 lb. Under these conditions, as of 2002, the tax rate was $0.192/mile (*16*). Figure 2.11 illustrates how the tax changes as a function of axles, for a range of maximum operating weights. Another perspective is presented in Figure 2.12 where tax rates are shown as a function of maximum weight. It is important to note that Figures 2.11 and 2.12 represent only three weight categories while the actual tax system contains 2,000 lb increments from 80,000 lb to 105,500 lb.

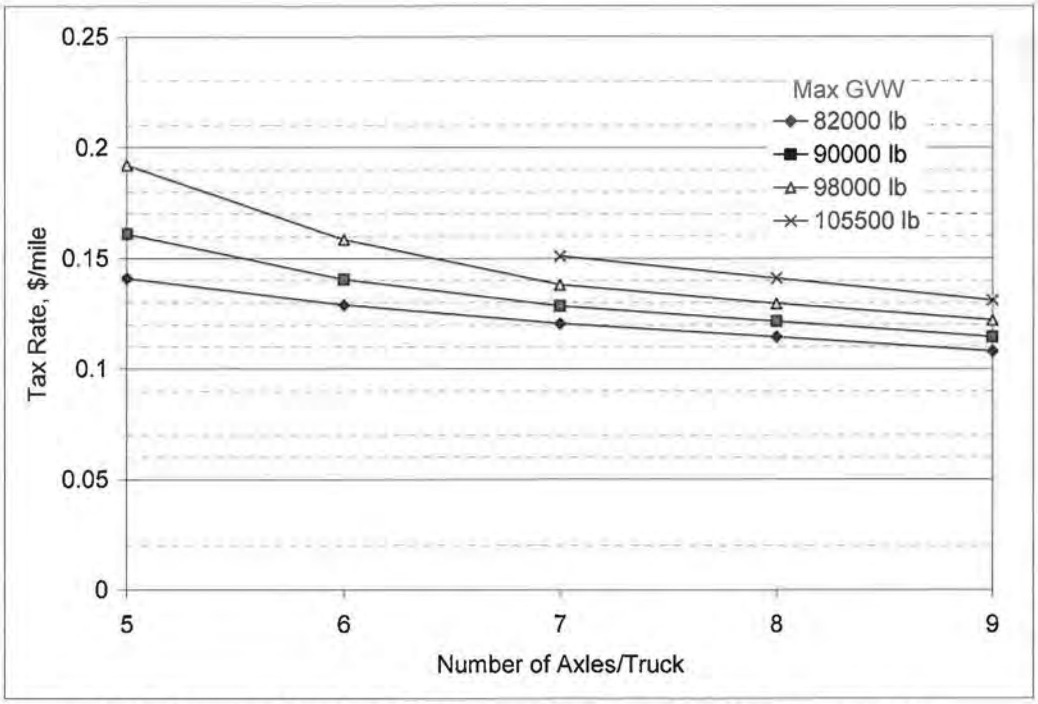


Figure 2.11 Oregon Weight-Mile Tax vs Number of Axles (*source: 16*).

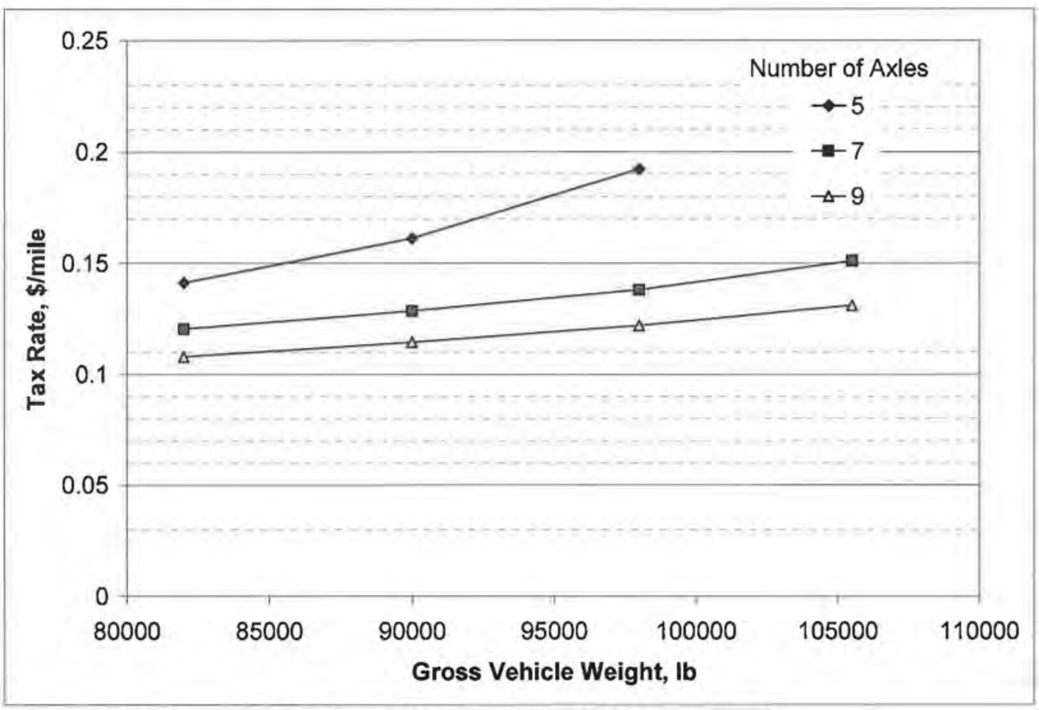


Figure 2.12 Oregon Weight-Mile Tax vs. GVW (*source: 16*).

The primary motivation for the Oregon tax system was a belief that motor carriers would adjust their fleet, as a function of the taxation, to maximize their profitability. This, in turn, would help mitigate the damaging effects of heavily loaded vehicles on the highway infrastructure. In the course of their study, Rufolo et al. (*16*) examined whether the weight-mile tax reduction for vehicles over 80,000 lb led to an increase in the number of axles per truck in a particular weight class. Through an examination of weight-mileage data and interviews of trucking companies, it was concluded that the mile-tax system has only minor effects on the number of axles per truck (*16*). It was concluded, based upon the interviews, that taxes are relatively unimportant when determining the type of equipment used. Regulatory and safety issues tended to play a larger role in determining the number of axles per truck.

The above discussion highlights the complexity of intra- and interstate legal weight limits and permitting practices. There are certainly many subtle nuances and regional-specific regulations that have evolved over time as the trucking industry has matured and new legislation and international trade agreements have been developed. Within this evolutionary process, there are many factors interacting as illustrated in Figure 2.13, which was developed as part of the Federal Truck Size and Weight Study (*5*). These factors are at play whether state, regional or nationwide truck size and weight regulations are under consideration.

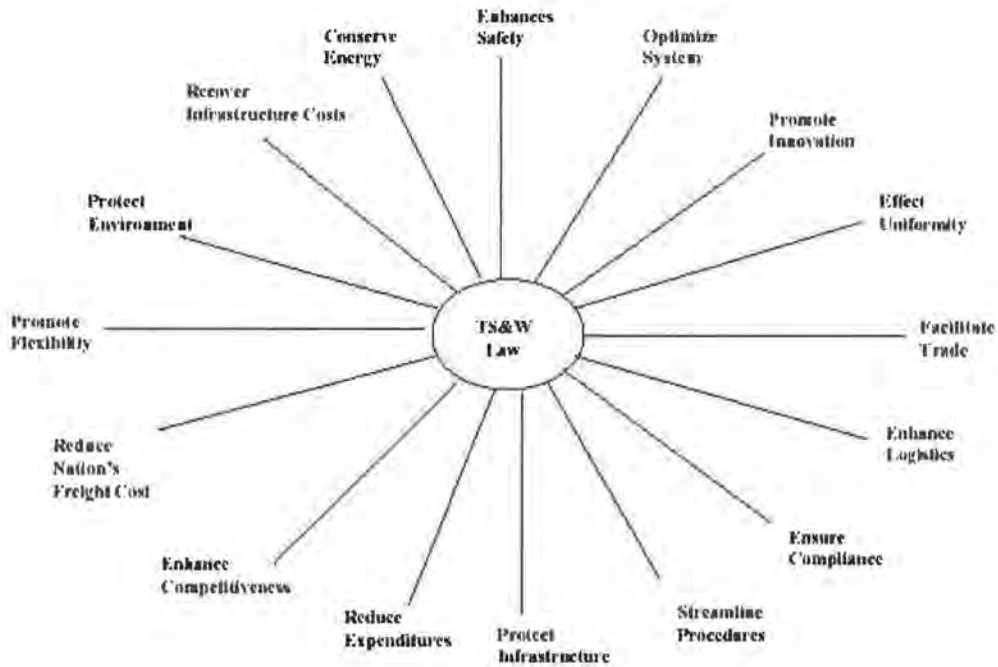


Figure 2.13 Forces Affecting Federal Truck Size and Weight Law (*5*).

VEHICLE OVERLOADS AND ENFORCEMENT

While every U.S. state and the federal government imposes strict legal weight limits, there is a prevalence of overweight violations on a yearly basis. Over the past 25 years, a number of studies have attempted to quantify the number of violators to better characterize the actual truck weights, and corresponding infrastructure damage, resulting from these overloads. The following subsections detail several of these studies with focus placed on the numbers and types of violators and enforcement practices. Later sections of the literature review will focus on infrastructure damage assessment.

Barros (*8*) conducted a study to determine overweight violators in New Jersey. He reported over 9,000 weight violations in 1981 based upon citations issued by the New

Jersey State Police. However, as Barros points out, scale avoidance is a common practice and the number of citations issued is likely significantly less than the actual number of overweight vehicles. To quantify the number of actual overweight vehicles, Barros (*8*) used loadometer tables to estimate 30 million truck trips per year in New Jersey. He then used a combination of portable scale data, personal correspondence with FHWA contacts and data obtained from an instrumented bridge on I-80 in Pennsylvania, to estimate that approximately 20% of all trucks in New Jersey are overloaded. This equates to approximately 6 million overloaded vehicles per year which dwarfs the 9,000 citations issued.

The overweight fine schedule, prior to 1983, in New Jersey is shown in Table 2.4 (*8*). Barros noted, based upon data from 1981, that 91% of violators were charged less than $200 while 9%, who are charged more, comprise approximately 25% of the total revenue (*8*). This observation is consistent with the fact that the heavier trucks cause exponentially more damage and should be fined accordingly. However, Barros found that the cost of enforcement alone ($1.7 million) exceeded the revenue generated through fining ($1.1 million). In other words, the fine schedule did not even cover the cost of enforcement, without mentioning infrastructure costs resulting from overweight vehicles. Barros estimated infrastructure damage costs of approximately $19 million on an annual basis. He concluded that changes to the fine schedule, greater enforcement or both were needed to cover the costs of enforcement and infrastructure damage.

Table 2.4 New Jersey Fine Schedule for Overweight Trucks Before September 1983 (*after 8*).

Excess Weight, lb	Penalty, $
0 to 2,500	50.00
2,500.1 to 10,000	0.02/excess lb above legal limit
> 10,000.1	0.03/excess lb above legal limit

The study by Barros indicated that approximately 0.15% (9,000/6,000,000) of all overweight vehicles in New Jersey were fined. In the same year, an interesting study by Wyatt and Hassan (*17*) examined the effects of enforcement level on percentage of overloads in Saskatchewan. They reported violation rates exceeding 15% for all types of loaded trucks when there is no enforcement at static scales. This was consistent with the 20% reported by Barros (*8*). Further, they found that the violation rate dropped to 3% when the probability of apprehension approached 10% (*17*). Applying these statistics to New Jersey would result in the issuance of approximately 600,000 citations; well beyond their capabilities in 1981. Beyond this level of enforcement, Wyatt and Hassan reported little change in the violation rate.

Wyatt and Hassan (*17*) also examined enforcement using WIM devices to measure weights of short haul trucks. Contrary to their findings from static weigh stations, they found 26% of all trucks were overloaded under normal enforcement operations (20 hrs/week) which only increased to 33.2% overloads under zero enforcement.

A different approach was taken by Meyburg et al. (*18*) to determine the number of overloaded vehicles in New York. Using paper surveys issued directly to trucking companies, they found that almost 50% of the statewide truck fleet operated above the permitted weight limits (*18*). While this percentage is quite high, the accuracy is unknown since it is not necessarily in the best interest of the trucking industry to voluntarily disclose such statistics. This was evident by the relatively low response rates. In three surveys issued (Summer 1990, Winter 1991 and Fall 1991) the response rates were 33.1%, 36.7% and 28.9%, respectively (*18*).

Perhaps the most recent and comprehensive examination of vehicle overloads and enforcement was published by Straus and Semmens (*11*). Their paper cites the U.S. General Accounting Office (GAO) 1979 estimate of 15% overloads nationwide (*19*) costing U.S. taxpayers approximately \$160-\$670 million per year (*20*). Since then, the estimates have gone up and as of 1987, estimated nationwide overloads were up to 25% (*15*). As part of their study, Straus and Semmens compiled overweight violation data from 2003 by state (*11*). Figures 2.14 and 2.15 express their data according to violations per million miles traveled and violations per 1000 trucks, respectively. For clarity, the states have been ordered from fewest to greatest number of violations.

It is interesting to note from Figures 2.14 and 2.15 that the state rankings do not match up. For example, Pennsylvania is fourth lowest in Figure 2.14 and second lowest in Figure 2.15. However, both methods of normalization (by VMT and by number of trucks) tend to increase together. To better show the relationship between these two methods of normalizing, consider the data in Figure 2.16. As seen earlier in the permitting data, there are two notable outliers in the figure; Washington, D.C. and North Dakota. As shown in the figure, linear regression was performed both with and without these states included. The regression without D.C. and North Dakota demonstrates a strong correlation between the two methods, as one would expect.

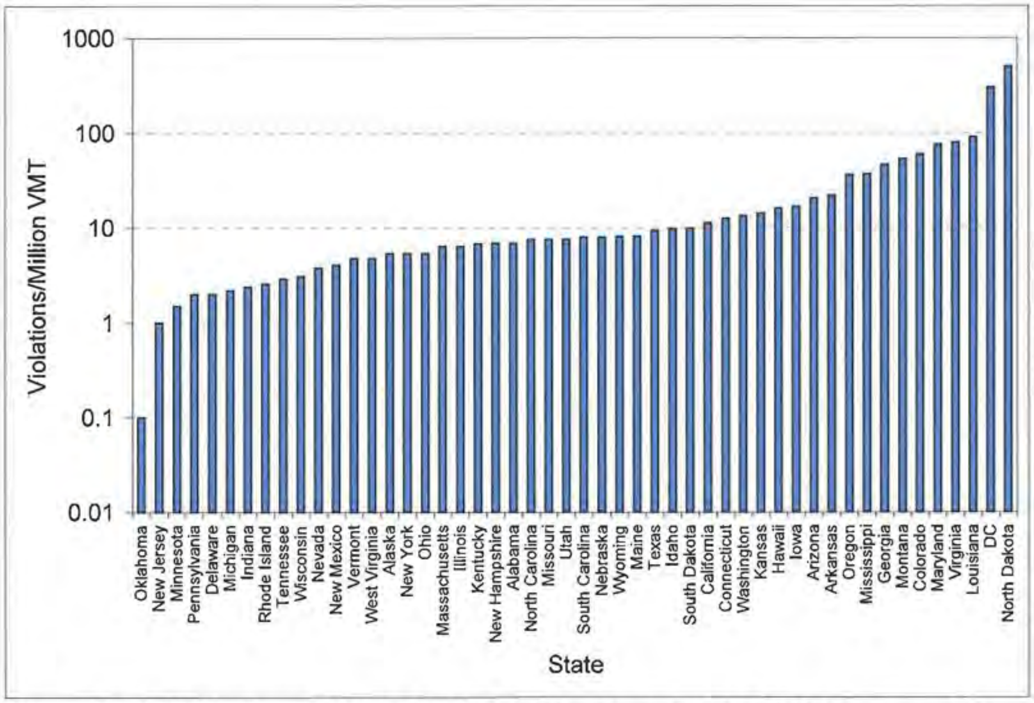


Figure 2.14 Violations/Million Vehicle Miles Traveled in 2003 (*source: 11*).

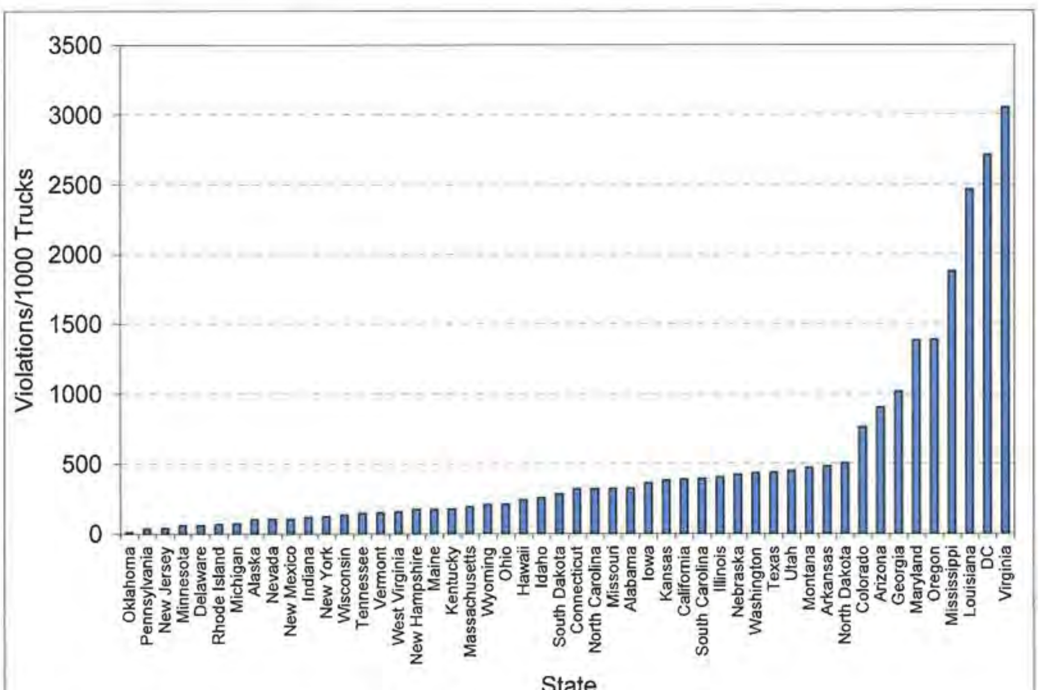


Figure 2.15 Violations/1000 Trucks in 2003 (*source: 11*).

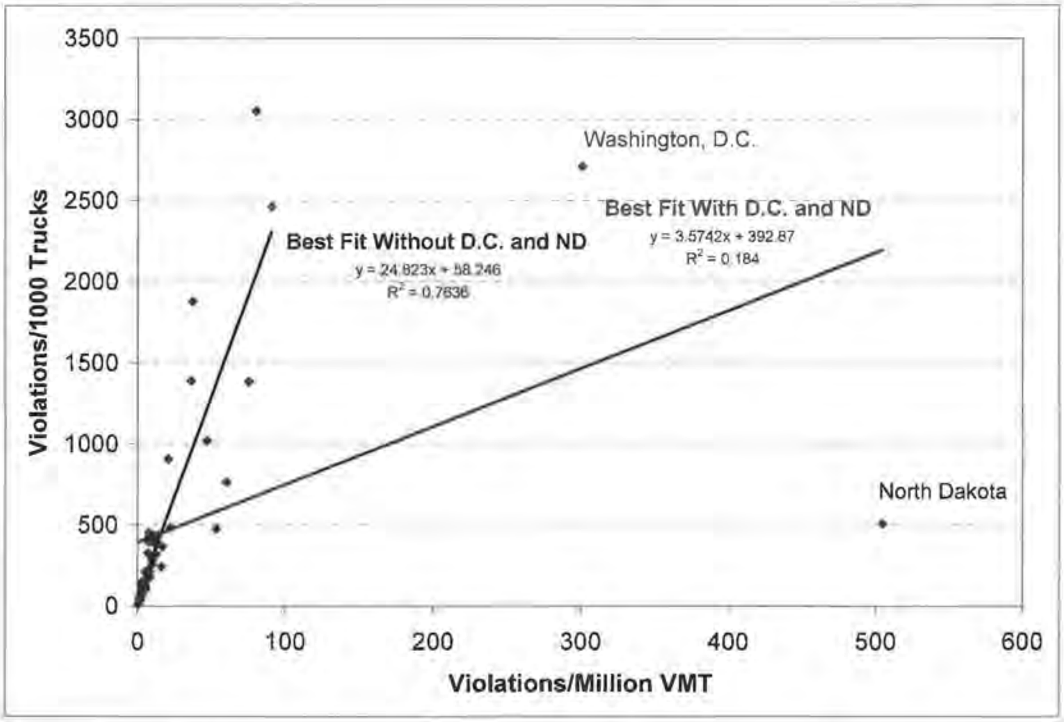


Figure 2.16 Violations/1000 Trucks vs. Violations/Million VMT (*source: 11*).

Another interesting comparison was derived by combining the violation data (*11*) with weight limit data published as part of the Truck Size and Weight study (*5*). Figure 2.17 illustrates the number of violations versus the legal weight limits on non-interstate routes in each state. Cursory inspection of Figure 2.17 indicates that there are generally a higher number of violations for states that adhere to the federal limits (20 kip single, 34 kip tandem, 80 kip GVW). This appears especially true for the gross vehicle weight. To better understand the interaction between limits and violations, the average number of violations were determined for states that followed federal limits versus those that do not. Figure 2.18 summarizes the findings and it is clear that GVW has the most significant impact on the number of violations. There are nearly double the number of GVW violations in states where the state and federal limits are equivalent (i.e., 80,000 lb).

Conversely, there are fewer violations for single and tandem axles where the federal limit equals the state limit.

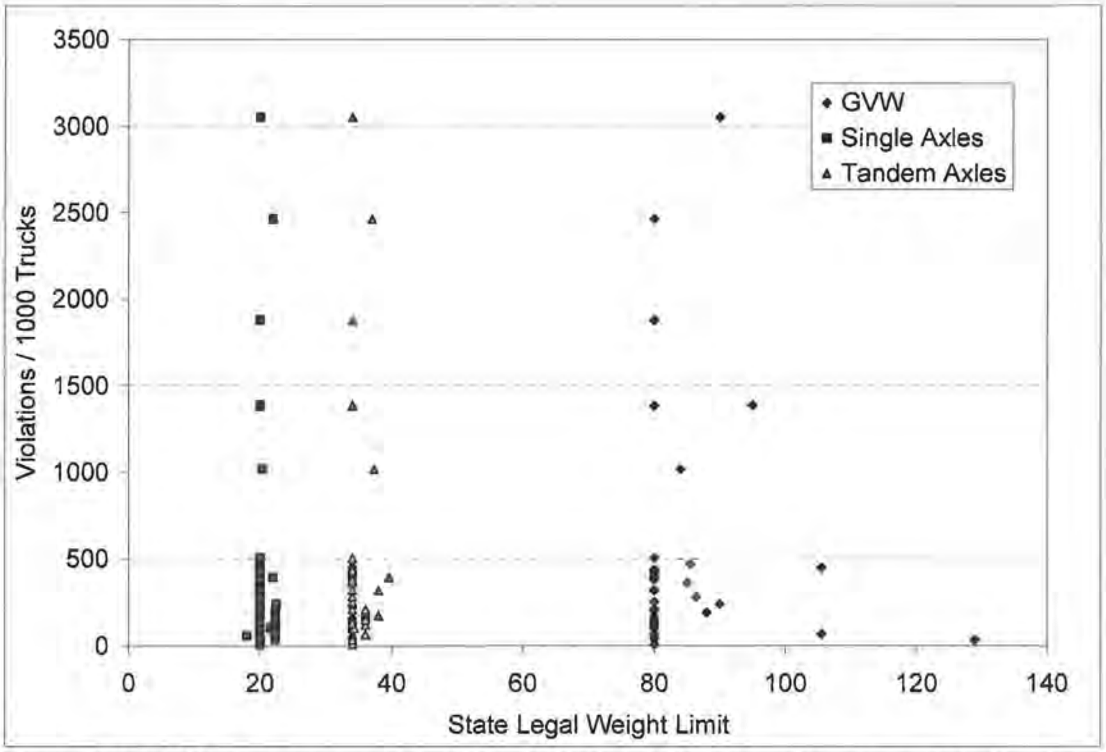


Figure 2.17 Violations versus State Legal Weight Limits on Non-Interstate Routes (*Source: 5, 11*).

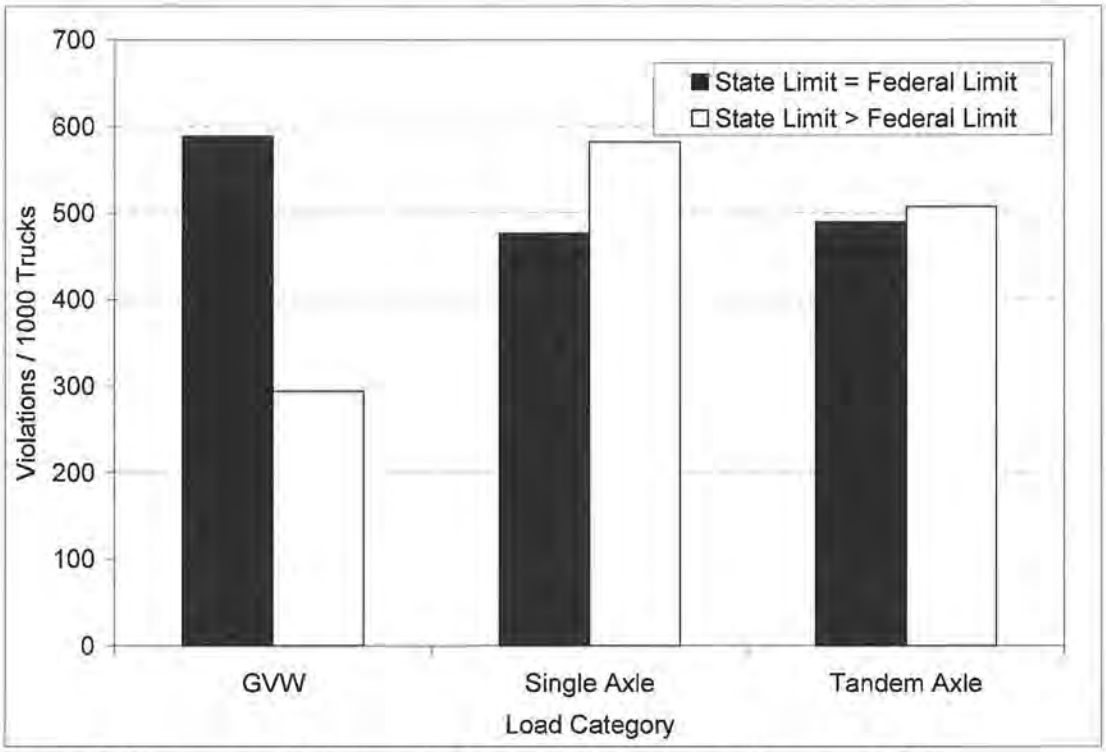


Figure 2.18 Average Violations by Federal Compliance (*Source: 5, 11*).

Straus and Semmens also surveyed states to estimate the frequency of GVW or axle overloads. Recall that previous estimates ranged from 15% (*19*) to 50% (*18*). Surveys issued to all 50 states and D.C. resulted in 25 responses with only 12 states estimating the frequency of overloading (*11*). Table 2.3 summarizes their data. This data set is in sharp contrast with the previously reported overload percentages with the majority below 10%. This highlights the acknowledged difficulty in accurately determining the frequency of vehicle overloads.

Table 2.5 Measured or Estimated Percentage of In-State Travel Comprised of Vehicles Exceeding Legal Limits (*11*).

STATE	PERCENTAGE
Arizona	30
Delaware*	~5 – 20
Indiana*	~2; 3-5
Louisiana	2
Montana	6.9
Nebraska	~0.5
Nebraska	<0.5
Oregon	10
South Dakota	0.5
Utah	~10
Washington	<5
Wisconsin	7
Alaska, Colorado, Georgia, Illinois, Maryland, Missouri, North Dakota, Ohio, Oklahoma, Tennessee, Vermont	Unknown
*varies by zone	

PAVEMENT DAMAGE ASSESSMENT DUE TO OVERLOADS

Accelerated pavement damage due to vehicle overloads has historically been quantified according to two general approaches. The first, based upon research conducted at the AASHO Road Test (*21*), quantifies damage in terms of equivalent single axle loads (ESALs) from the overloaded trucks. The second approach utilizes mechanistic-empirical (M-E) pavement analysis concepts to predict accelerated pavement damage due to the overloads. Though both approaches have their inherent advantages and disadvantages, the ESAL approach is currently the most popular among states. According to Chen et al. (*22*), "TxDOT and almost all other states are still basing predictions of overload damage on the 'fourth power rule' that was developed based on conditions that prevailed in the 1960s". The following subsections will detail both approaches, with examples provided from a variety of states.

Equivalent Single Axle Approach

The so-called "fourth-power rule" resulted from the AASHO Road Test that was conducted from 1958-1960 in Illinois (*21*). This full scale pavement test set the standard for flexible and rigid pavement design and is currently used by most U.S. states and has been adopted to other parts of the world. The fourth-power rule refers to the equivalent single axle load equations that were derived to convert axles of various configurations and load magnitudes into an equivalent number of passes of a standard axle. The standard axle was selected as an 18,000 lb single axle with dual tires. The equations for flexible and rigid pavement are (*23*):

Flexible Pavement Equivalent Axle Load Factor

$$\log\left[\frac{W_{tx}}{W_{t18}}\right] = 6.1252 - 4.79\log(L_x + L_2) + 4.33\log L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}} \tag{2.1}$$

Where:

W_{tx} = number of applications of an X-kip axle causing equivalent damage as W_{t18} passes at time, t

W_{t18} = number of applications of an 18-kip single axle at time, t

L_x = X-axle load magnitude, kips

L_2 = axle code (1 = single, 2 = tandem, 3 = tridem)

$$G_t = \log\left[\frac{4.2 - p_t}{4.2 - 1.5}\right]$$

p_t = terminal serviceability

$$\beta_x = 0.40 + \frac{0.081 \cdot (L_x + L_2)^{3.23}}{(SN+1)^{5.19} \cdot L_2^{\ 3.23}}$$

$\beta_{18} = B_x$ where x = 18

SN = structural number of pavement

Rigid Pavement Equivalent Axle Load Factor

$$\log\left[\frac{W_{tx}}{W_{t18}}\right] = 5.908 - 4.62\log(L_x + L_2) + 3.28\log L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}} \tag{2.2}$$

Where:

$$G_t = \log\left[\frac{4.5 - p_t}{4.5 - 1.5}\right]$$

$$\beta_x = 1.0 + \frac{3.63 \cdot (L_x + L_2)^{5.20}}{(D+1)^{8.46} \cdot L_2^{\ 3.52}}$$

D = slab thickness, in.

W_{tx}, W_{t18}, L_x, L_2, p_t = as defined above

Knowing the type of pavement, the load configuration, the load magnitude and the terminal serviceability, equations 2.1 and 2.2 can be used to define the relative damaging effect of any axle type compared to the standard 18,000 lb equivalent. This relationship is shown schematically in Figure 2.19. This concept can be applied to overloads by simply computing the equivalent number 18,000 lb standard axle passes caused by one pass of the overloaded axle or vehicle.

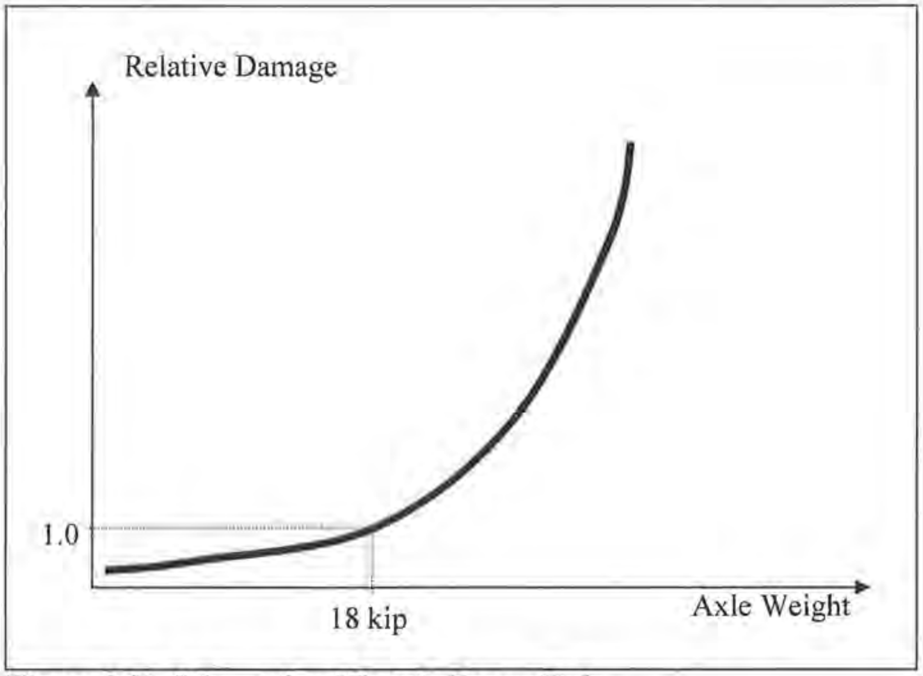


Figure 2.19 Schematic of Fourth-Power Rule.

The ESAL approach was used by Meyburg et al. (*18*) in their cost-benefit study to estimate infrastructure damage. They calculated ESALs/truck and then multiplied by the miles driven and a cost-coefficient ($/mile) to arrive at an infrastructure cost. Only loaded trips were counted since the empty trips resulted in very low ESALs (*18*). While Meyburg et al. (*18*) examined ESALs from both a per vehicle and per axle basis, they concluded that per axle better represented actual pavement damage.

A study conducted in New Jersey (*8*) also utilized the ESAL approach. Barros, in an effort to quantify the damaging effects of overloaded vehicles, developed a "model" truck fleet based upon violation data. Barros applied this truck fleet to "typical" New Jersey flexible and rigid pavements. It was assumed that the flexible pavements had a structural number of 5 and the rigid pavements had a 9 inch slab. Barros was able to estimate 38,146 ESALs of pavement damage per year based upon 9,060 overweight violations per year. This corresponded to 7.63% loss in pavement life due to the overloads when considering a design traffic level of 500,000 ESAL per year (*8*). This analysis assumed that the same number of trucks were carrying a larger amount of freight. Barros conducted a second analysis where he assumed that additional trucks were used such that overloads were not needed. The increased traffic volume also resulted in accelerated pavement damage resulting in a 6.17% loss of pavement life per year (*8*) which was slightly less than the case where overloads were used. Barros highlights an important concept in pavement damage analysis. That is, given a total weight of cargo to carry within a local economy it can either be carried by fewer, more heavily loaded trucks, or more, legally loaded trucks. Either case may result in reduced pavement life.

Roberts and Djakfar's study of special, commodity-specific, permitting in Louisiana utilized the ESAL concept (*14*) and computed the shortened time to overlay of flexible pavements due overloads. Similar to Barros' (*8*) analysis, they computed reduced life as a function of increasing the vehicle weights, increasing the number of trips and a combination of the two to carry the total freight. A number of scenarios were evaluated from which the following conclusions were made (*14*):

1. Increasing GVW on vehicles transporting commodities on a system of roads that were designed for vehicles operating at a lower GVW decreases the service life of the road in a manner proportional to the ratio of ESALs produced by vehicles under the new GVW divided by the number of ESALs remaining in the design period.
2. The larger the original design ESALs were for the roadway in question, the smaller the impact of increased GVW on pavement life. This implies that the impact is less severe for interstate highways, than for U.S. or state routes.

3. The cost to road users of increasing the GVW limit for a few select commodities can be significant.

A study from Ontario in 1985 (*3*) examined the increased pavement damage due to increasing the single axle load limits from 18,000 lb to 20,000 lb. Using actual load distributions measured in 1967 (18,000 lb limit) and 1981 (20,000 lb limit), Abdel Halim and Saccomanno (*3)* applied load equivalency factors to determine the relative damaging effect of the increased weight limit with an assumed structural number equal to 6. They stated the importance of a long time horizon in their analysis so that changes in trucking technology due to the increased limit would be reflected in the loading data. Using too short a time interval after the increase would not accurately capture the change in axle weights (*3*).

As a result of their investigation, Abdel Halim and Saccomanno describe four interesting trends in the interval from 1967 to 1981 (*3*):

1. There was an increase in the proportion of single axles during the time period. In 1967, single axles were 50% of all axles which increased to 62.6% in 1981. One could surmise that more single axles were being used since they could be loaded more heavily and potentially reduce the operating costs to the motor carrier.
2. Though more single axles were being used in 1981, more of the GVW of a vehicle was being carried by tandem and tridem axles. The authors determined that though the legal limit had gone up, vehicle capacity was being used more efficiently with respect to pavement deterioration.
3. GVWs were less in 1981 than 1967. The authors attributed this observation to better trucking technology and more efficient use of truck fleet capacity.

4. The increased single axle limit reduced the incidence of single axle overloads by about 15%.

Overall, Abdel Halim and Saccomanno found a net reduction in pavement damage due to increasing the legal single axle limit. They cited, as discussed above, more efficient trucking technology that allowed the total freight to be carried more efficiently, as the primary factor in their analysis (*3*).

Similar research was conducted prior to the Ontario study by Carmichael et al. (*24*) in 1979 to evaluate the potentially damaging effects of increased load limits in the U.S. Their methodology, embodied in the computer program NULOAD, was meant to assess the life cycle cost of flexible and rigid pavements due to changes in legal limits. The methodology consisted of four parts (*24*):

1. Determine ESALs for current and projected legal limits. This includes a procedure for shifting axle weight distributions to reflect the higher legal limits.
2. Predict the expected life cycle for pavements that includes rate of deterioration, time of overlay and overlay requirements.
3. Estimate the minor maintenance and overlay costs associated with the representative pavement structures.
4. Print a summary of the cost analysis.

Though the NULOAD program is undoubtedly outdated by now, there were some notable features that are important for analysis at any time. First, the program could accommodate up to ten different truck types with various axle combinations and new truck sizes could be considered (*24*). This is an important feature as trucking technology continues to evolve. Second, analyses could be conducted on the assumption either that the same total payload per year is carried under present and proposed limits or that the same number of truck trips per year is made under both limits (*24*).

Mechanistic-Empirical Approach

In recent years, there has been a national movement in the U.S. toward mechanistic-empirical (M-E) pavement design and analysis. This approach utilizes mechanistic pavement models to simulate pavement responses under applied traffic loads. The responses can then be empirically correlated to expected pavement performance. While there are existing M-E approaches, such as those developed by the Asphalt Institute and the Portland Cement Association, recent efforts under NCHRP 1-37A (*25*) to develop the M-E Pavement Design Guide (MEPDG) have helped to unify design standards for both flexible and rigid pavement structures.

Prior to the development of the MEPDG, there were a number of studies that applied M-E concepts in assessing pavement damage due to vehicle or axle overloads. In 1989, Kilareski (*26*) reported on a study for Pennsylvania that evaluated both flexible and rigid pavements subjected to overloaded axles. Using the computer programs BISAR and JSLAB to evaluate flexible and rigid pavements, respectively, he examined critical pavement responses on two flexible pavements and one rigid pavement. The flexible pavements represented thin and thick structures, while the rigid analysis considered a representative slab commonly used in Pennsylvania (*26*).

With respect to traffic loadings, Kilareski considered single, tandem, quad and five axle configurations typically found in Pennsylvania. Loadings per axle ranged from 18,0000 lb to 34,000 lb. Thus, a five axle configuration with 34,000 lb per axle resulted in 170,000 lb for the axle group. It was assumed that the tire/pavement interface was circular with 100 psi contact pressure (*26*).

The above-mentioned pavement and axle loadings were simulated to determine pavement response as a function of axle loadings. Figure 2.20 illustrates the relationship between tensile strain and axle load. While Kilareski computed pavement responses for both flexible and rigid pavements, his final analysis in terms of pavement life predictions only utilized the tensile strain data from Figure 2.20 to predict flexible pavement fatigue cracking. The prediction was made with a fatigue transfer function developed at Penn State University, as shown in Figure 2.21. It is interesting to note the good comparison between the Penn State function and the other available fatigue transfer functions. It is also important to note that the function utilizes ESALs on the x-axis. This signifies how strongly the pavement engineering community is tied to the ESAL concept; even when conducting an M-E analysis, traffic is still commonly expressed as ESALs.

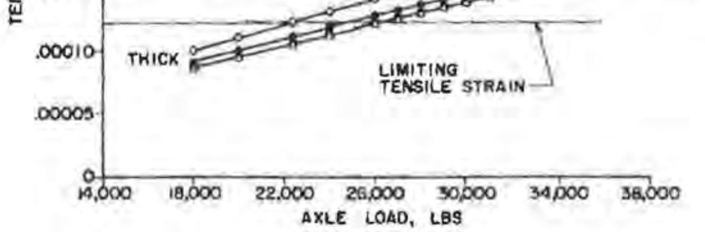


Figure 2.20 Strain vs. Axle Load (*26*).

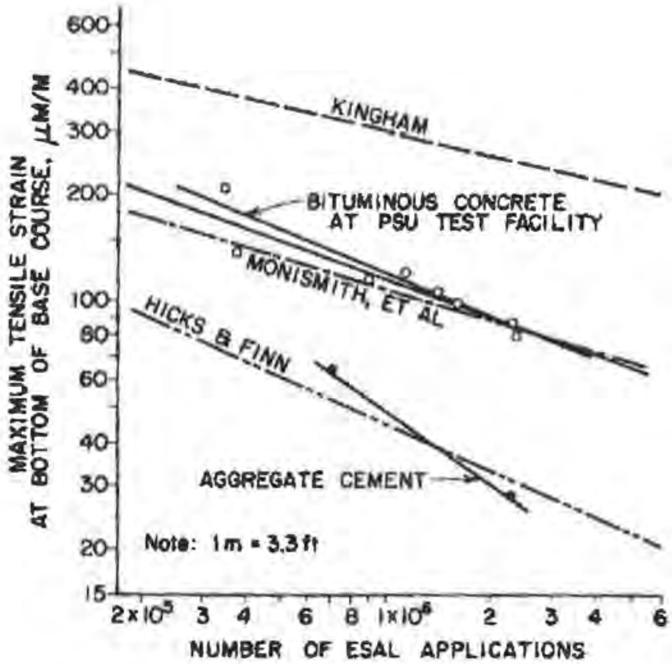


Figure 2.21 Fatigue Algorithm (*26*).

The final step taken by Kilareski was to combine the data from Figures 2.20 and 2.21 to establish pavement life curves for the various scenarios and weights considered, as shown in Figure 2.22. Kilareski noted in his analysis that no significant differences existed between the axle configurations in the thin pavement category since all the curves overlapped (*26*). Overall, Figure 2.22 can be thought of as a design chart whereby engineers could consider the relative fatigue damaging effects of various axle configurations and load magnitudes.

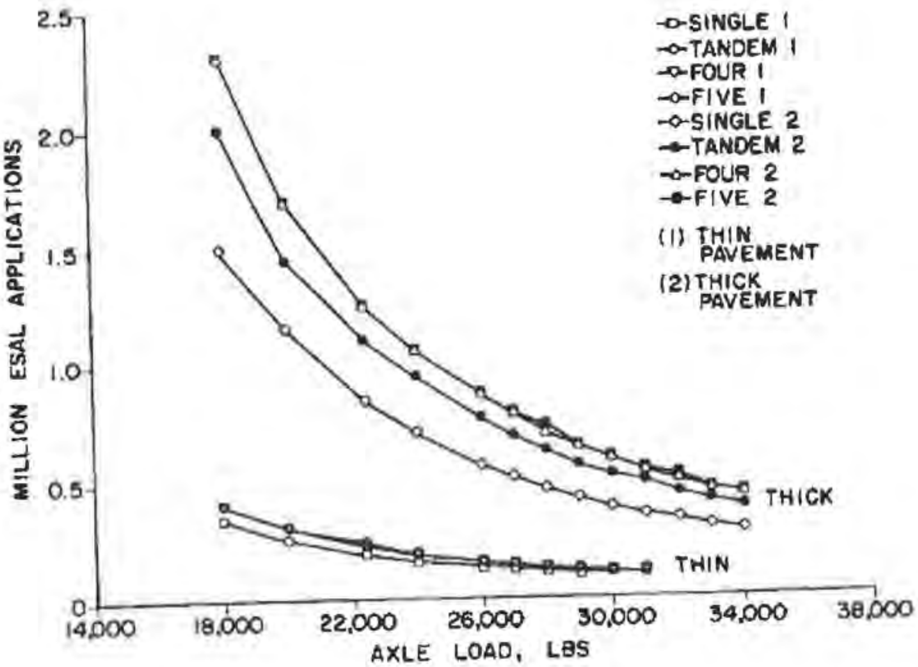


Figure 2.22 Predicted Fatigue Life (*26*).

More recent studies conducted in Texas have documented the M-E analysis of the super heavy load permitting program (*12, 13*). As discussed previously in this literature review, TxDOT operates a super heavy load program which mandates pavement analysis for GVWs exceeding 500,000 lb (*12*), though loads exceeding 250,000 lb are classified as super heavy. Jooste and Fernando (*13*) described a framework for assessing pavements on super heavy routes that included:

1. The use of ground penetrating radar (GPR) to estimate pavement layer thicknesses along a proposed route.
2. Non-destructive testing using a falling weight deflectometer (FWD) to assess structural integrity of the pavement.
3. The use of an automated road analyzer to measure pavement roughness, and document pavement condition before and after a super heavy move.

4. The use of Texas Triaxial Class data to evaluate the potential for subgrade pavement failure under the super heavy load.

The primary objective of the program described by Jooste and Fernando was to identify cases where sudden subgrade failure would occur due to the super heavy move (*13*). To determine the layer properties of the pavement structures, the GPR data are used with the FWD data in the backcalculation program MODULUS. The GPR data are also used to identify potentially weak points along the route (*13*). The material properties can then be used to simulate the pavement loads on route segments to estimate if subgrade failure would occur. Ultimately, their analysis rested upon a Mohr-Coulomb stress analysis using Texas Triaxial data and the computed stress state to determine if failure would occur (*13*).

More recently, Chen et al. (*12*) reevaluated the TxDOT super heavy permitting program and focused more on the potential for damage due to repeated loads rather than damage due to a single pass. This shift was based primarily on Jooste and Fernando (*13*) finding that subgrade stress was not typically near the critical point for many super heavy moves. Like the previous study (*13*), Chen et al. (*12*) also simulated the pavement sections under the super heavy loads to determine tensile and compressive strains. These responses were then transformed through the Asphalt Institute's fatigue and rutting transfer functions to estimate the number of cycles until failure (*12*):

$$N_f = 0.0796\left(\frac{1}{\varepsilon_t}\right)^{3.291}\left(\frac{1}{E_{ac}}\right)^{0.854} \tag{2.3}$$

$$N_c = 1.365\times10^{-9}\left(\frac{1}{\varepsilon_c}\right)^{4.477} \tag{2.4}$$

where:

N_f = allowable number of load repetitions to control fatigue cracking

N_c = allowable number of load repetitions to control rutting

ε_t = tensile strain at bottom of asphalt layer

E_{ac} = asphalt modulus, psi

ε_c = vertical compressive strain on top of subgrade

It was noted by Chen et al. (*12*) that rutting controlled for most super heavy cases they considered. Also, as of 2005, Chen et al. had provisionally recommended a lower limit of 588 cycles until failure, as calculated by equation 2.4. Above this value, it is believed that the structure is capable of supporting the super heavy load. They also recommended revising the 588 cycle lower limit as further field results are gathered (*12*).

As noted previously in this literature review, TxDOT has many load-zoned pavements that are limited to 58,240 lb (*10*). As TxDOT has worked toward rehabilitating their road network, there is a need to reevaluate segments and determine if higher limits can be used (*27*). To that end, the Program for Load-Zoning Analysis (PLZA) was recently developed to assist TxDOT engineers in the rational determination of the appropriate load limits (*27*). Based upon M-E concepts, the procedure is shown schematically in Figure 2.23. Similar to Chen et al. (*12*), this procedure relies upon mechanistic pavement modeling to determine the critical pavement responses. The responses are then transformed through equations 2.3 and 2.4 so that predictions of pavement performance can be made. As shown in Figure 2.23, the pavement distress increases as the load increases at a given time. The final outcome of the load-zoning analysis is a plot indicating the time until pavement resurfacing as a function of the posted limit.

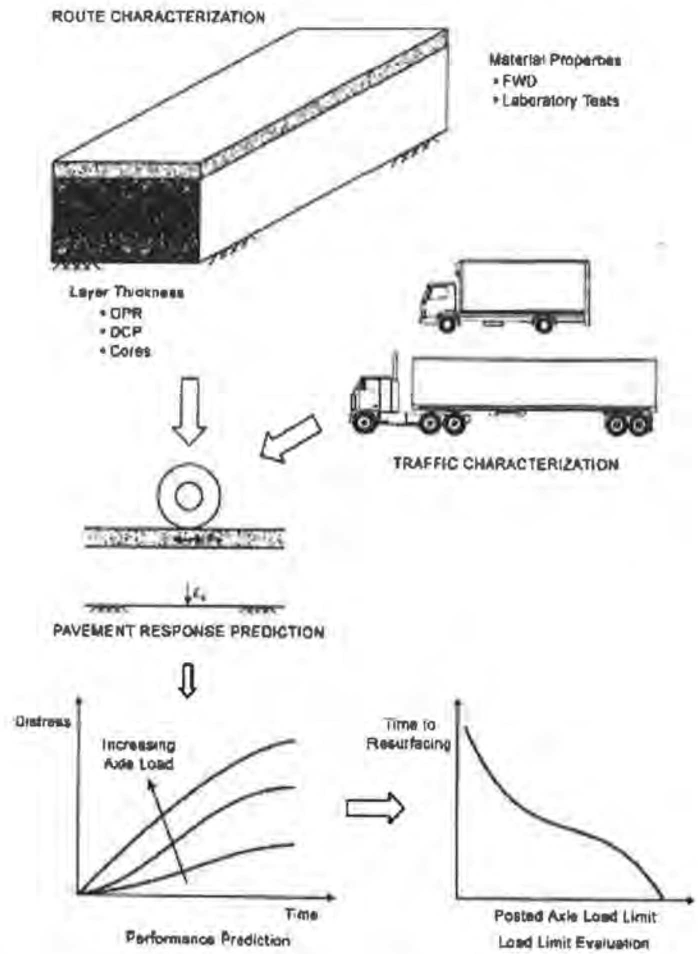


Figure 2.23 Framework Used to Develop Program for Load-Zoning Analysis (*27*).

The pavement damage analysis conducted as part of the Comprehensive Truck Size and Weight Study utilized the computer program NAPCOM (*28*). NAPCOM relies upon 11 individual distress models, rather than the "fourth-power" rule to estimate pavement damage. Using mechanistic principles and performance data for calibration, NAPCOM predicts the distresses listed in Table 2.6.

Table 2.6 NAPCOM Distresses (*28*).

Flexible Pavements	Rigid Pavements
Pavement serviceability rating loss	Traffic-related pavement serviceability rating loss
Expansive clay present serviceability loss	Faulting
Fatigue cracking	Loss of skid resistance
Thermal cracking	Fatigue cracking
Rutting	Spalling
Loss of skid resistance	Soil induced swelling and depression

In comparison to the "fourth-power" relationship developed by AASHTO, the relationships in NAPCOM are typically less than the fourth-power. As reported by Merriss (*29*), the relationship between pavement damage and axle load is typically on the order of 2.5 instead of 4.0.

OVERLOAD ECONOMIC AND COST-BENEFIT ANALYSES

Since the inception of size and weight limits in 1913, states have been faced with balancing infrastructure health with the desire of the trucking industry to move toward more efficient, cost-effective, and typically heavier freight hauling options. As reported in the 1990 Truck Weight Limits Report (*1*), increasing truck weights can significantly reduce the cost of goods movement due to more efficient trucking. In general, these benefits generally outweigh the additional pavement and bridge costs to highway agencies. However, the report does go on to caution that the infrastructure is likely to deteriorate more rapidly and there must be a funding mechanism to account for increased maintenance and rehabilitation (*1*).

A study by Meyburg et al. (*18*) estimated the relative costs and benefits of increasing the GVW limits. In their study, 125%, 135% and 145% of legal were considered. Using the aforementioned "fourth-power" rule to estimate ESALs for each load level, Meyburg et al. (*18*) assigned cost rates to interstate, state and local highways. Their cost rates, listed in Table 2.7, were based upon consultation with highway officials from several states and review of pertinent literature. Clearly, the costs to thinner pavements (i.e. local highways) are much greater since they are less capable of handling the increased loadings and have the potential to deteriorate more rapidly.

Table 2.7 Cost Rates (*after 18*)

Route Type	\$/mile/ESAL
Interstate Highway	0.02
State Highway	0.06
Local Highway	0.40

Benefits were calculated by assuming freight carriers could make fewer, more heavily loaded trips, to carry the same total freight (*18*). Though not explicitly mentioned in their paper, it appears that the trucking company benefits were derived solely from reduced labor costs and fewer trips. It did not appear that increases in fuel consumption as the GVW increased was considered in their analysis.

Barros conducted an economic analysis pertaining to overloaded vehicles on pavement structures (*8*). He applied classic life cycle cost analysis in his approach to determine the effects of overweight trucks. Figure 2.24 illustrates the method schematically where two scenarios are considered. The first scenario is the expected rehabilitation and maintenance schedule when overloads are not considered or allowed. The second scenario has shorter time spans between recurring maintenance and rehabilitation activities due to faster pavement deterioration rates from the overloads. Barros cites some important factors when conducting this type of analysis that include:

1. Interest and inflation rates to account for future costs of construction and the time value of money.

2. Since a statewide analysis was conducted, the average trip length of each overloaded truck was important.
3. Engineering, traffic control, patching, resetting manholes and other associated maintenance and rehabilitation costs.

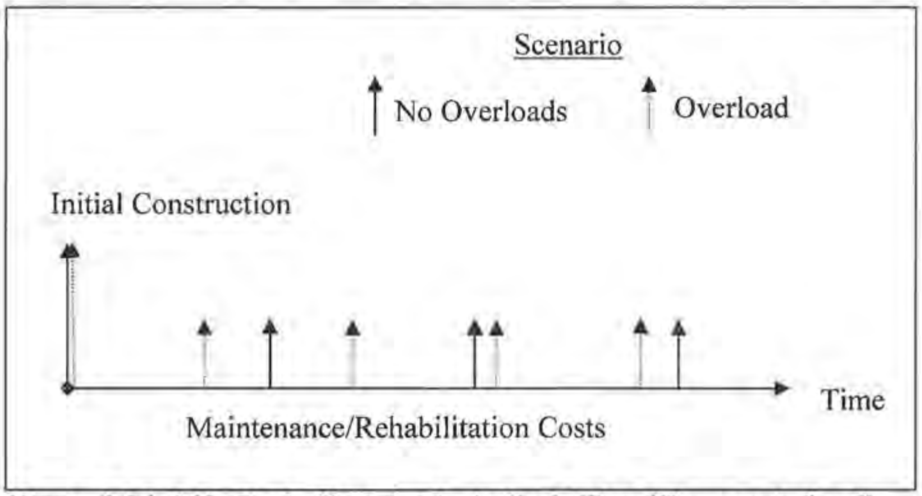


Figure 2.24 Life Cycle Cost Analysis Cash Flow Diagram (*after 8*).

As part of Barros' (*8*) analysis, the net present value (NPV) was calculated for a number of conditions in New Jersey. The NPV formula is:

$$NPV = I.C. + \sum_{k=1}^{N} R.C._{k} \left[\frac{1}{(1+i)^{n_k}} \right] \tag{2.5}$$

where:

I.C. = initial construction cost

R.C. = recurring rehabilitation/maintenance costs

k = rehabilitation/maintenance activity

i = interest rate

N = year in which rehabilitation/maintenance occurs

Barros' analysis, based upon 1983 dollars, estimated increased pavement costs ranging from \$7 million to \$43 million per year, depending upon the assumptions made regarding the interest rate, inflation, the average remaining life of the pavement and the length of the overweight trip (*8*).

SUMMARY

This literature review began by examining current weight regulations from both a state and federal perspective. It was found that weight regulations are highly state specific and that many states have legal limits exceeding federal limits for GVW, single and tandem axles, respectively. Most states also issue so-called "Routine Permits" that require no special investigation, but only payment of a permitting fee to operate at a heavier weight. It was also found that GVW and axle weights are considered separately in permitting.

Regarding permitting fees, it was found that the fee structure has not historically been damage-based. Despite general knowledge of significant costs to the transportation infrastructure due to heavy loads, permitting fees are usually established to recover administrative costs only (*5*).

An investigation of the number of overloads indicated that they are significant on a nationwide basis. Current estimates of overloads, from government agencies, range from 15-30% of all trucks. While this is significant, one study in New York surveyed trucking companies and found a 50% overload percentage, which was deemed an unconservative estimate since trucking companies may not wish to reveal operations in excess of legal limits. Clearly, more well-defined information regarding the occurrence of overloads is warranted.

Pavement damage assessment due to overloads has traditionally been based upon an empirical, equivalent single axle load (ESAL), approach or a mechanistic-empirical (M-E) approach. Using the ESAL approach, agencies have accounted for increased pavement damage and faster deterioration rates by computing the relative damaging effect of the heavier loads. The M-E approach has relied upon mechanistic modeling of typical or specific pavement sections with heavier loads to predict the increased rate of deterioration. Both approaches have been used with reasonable success. Given that pavement engineering is progressing toward mechanistic-empirical pavement design, using an M-E approach is considered state-of-the-art.

The last portion of the literature review focused upon cost analysis. It was found that standard economic models, whereby the net present value of various scenarios can be evaluated, were common. As part any economic analysis, it is important to also weight the potential economic benefits of increased weight limits due to more efficient trucking operations.

CHAPTER 3

SURVEY OF STATES' PRACTICE

INTRODUCTION

A wide variety of factors, such as overweight permit regulations set by transportation agencies and state legislatures, types of products commonly hauled in a particular state, enforcement resources, as well as traffic patterns and climatological factors, affect the loading and routing of overweight vehicles. For these reasons, an equally wide range of procedures for issuing and managing these permits has developed among the states.

The purpose of this task is to document the state of current practice. There is an interest in documenting, at a high level, the range of practices on items such as permit regulations and damage analysis methods. Additionally, such an effort could potentially highlight novel, innovative, or otherwise useful practices that could be of use to other agencies.

DATA COLLECTION

Development of the Survey Instrument

To determine the state of the practice regarding permit processes and regulations for overweight vehicles, a questionnaire was developed and sent to the responsible agency in each state. Objectives of this effort included documenting regulations, identifying previous/ongoing research, practices to assess damage and associated cost, and factors affecting weight limits and enforcement. The questionnaire was developed to obtain information to meet the objectives listed above. The draft of the survey originally included questions pertaining to these objectives. This draft was reviewed by several members of the FHWA project advisory panel; revisions were made and several additional questions were added to address topics of general limits on vehicle width, height, length, and weights by vehicle and axle. The questionnaire is attached as Appendix I; the list of states responding is shown in Table 3.1.

Table 3.1 States Responding to the Questionnaire

Alabama	Maine	Oklahoma
California	Michigan	Oregon
Colorado	Minnesota	Tennessee
Connecticut	Missouri	Texas
Florida	Nebraska	Utah
Idaho	New Hampshire	Virginia
Illinois	New Jersey	Washington
Kansas	North Carolina	West Virginia
Louisiana	Ohio	Wyoming

Survey Distribution

This survey instrument was distributed during April and May of 2006. Distribution of the survey entailed contacting each state DOT, and then many state highway patrols or similar agencies, in order to target a specific person in each state. Several attempts to identify the most appropriate person were made; this person could be in a state DOT or a law enforcement agency (such as a state highway patrol). After follow-up contacts were attempted, a total of 27 states were represented in the responses, for a response rate of 54%. Two key factors that may have affected the response rate include challenges in identifying the most likely and knowledgeable person to respond within each state (and that typically the expertise of several offices/agencies would be required to answer all questions) and the length of the survey, as evidenced by the fact that most states either left the tables on general limits (the last four of ten pages) blank or sent copies of regulations or handbooks that included this information instead. Two survey respondents noted, in telephone conversations with the researchers, that the survey was "long" and that much of the information pertaining to limits could be found on their websites.

DATA ANALYSIS

Permit Processes and Coordination

The first section of the questionnaire pertained to general information on the development and coordination of permit application processes. All states indicated that both single-trip and either annual or repeat-trip permits were available. 21 of the 27 states (78%) that responded to the survey have online application processes available (although in one state this is for annual permits only). One additional state reported that development of an online application process is underway. Regarding the basis of permit fees, gross vehicle weight (GVW) and a combination of mileage and GVW are most common, comprising a majority of states. Table 3.2 and Figure 3.1 show the range of responses among the 27 states that responded to the survey.

Table 3.2 Permit Fee Criteria

Fee Criteria	Number of States (Percentage)
Gross Vehicle Weight + Mileage	8 (30%)
Gross Vehicle Weight	7 (26%)
Gross Vehicle Weight + Number of Axles	5 (19%)
Commodity Type	3 (11%)
GVW + Axle Weight + Axle Configuration	2 (7%)
Mileage	1 (4%)
Duration	1 (4%)

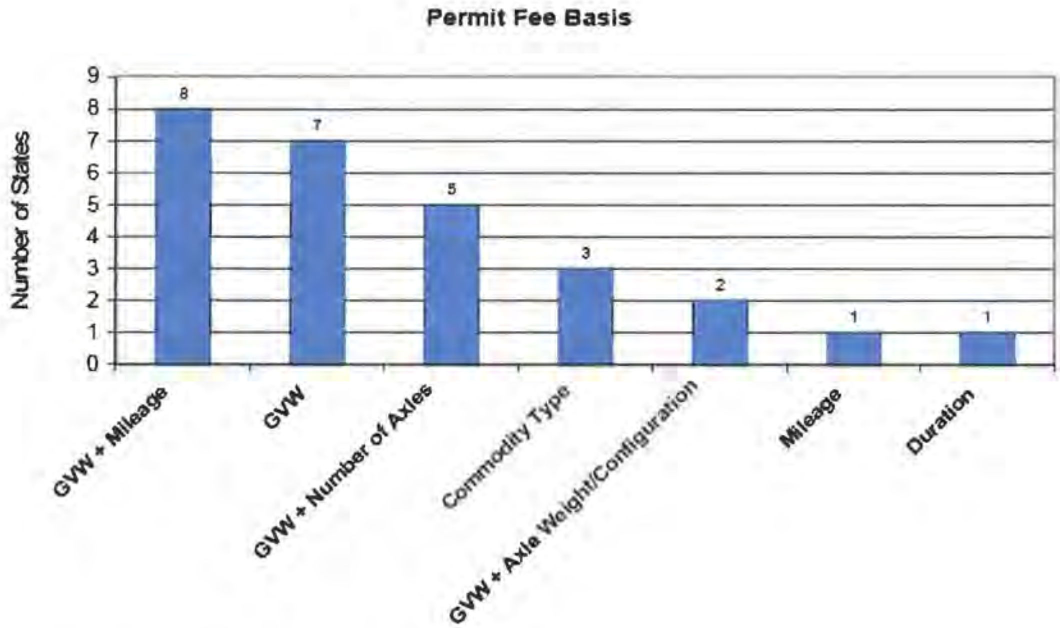


Figure 3.1 Permit Fee Criteria.

Even though a wide variety of bases for permit fees exist, in many states, fees are set by the state legislature and therefore the transportation agency is not in control of the fee basis. While this was not the focus of a particular question in the survey, 12 (44%) of survey respondents referenced sections of their state codes in their responses. Additionally, three of the respondents, in telephone conversations with the research team, expressed the notion that their "hands are tied" when it comes to setting fees and that a study that would relate permit fees to infrastructure damage would not be worthwhile. At least one respondent noted that the fees do not come close to reflecting the cost associated with damage to highways and bridges caused by overweight vehicles.

Four of the 27 states (15%) responding to the survey reported that some effort had been made to relate the permit fee structure to the extent or cost or infrastructure damage caused by the permitted vehicle, and one other state indicated plans to do so. In two of the four states (Idaho and Minnesota), such studies were conducted in the 1980s and characterized as "in-house" studies. One state (Florida) supplied a table showing the results of a study conducted in 2001 relating fees to ranges of GVW, and one state (Louisiana) had conducted two studies, each focused on a particular commodity (forest products and sugar cane).

Regarding coordination with other states, most states reported participating in periodic discussions with nearby states through regional groups such as SASHTO and WASHTO (Southern and Western Associations of State Highway and Transportation Officials, respectively), the New England Transportation Consortium, or other communications. This group includes 19 (70%) of survey respondents; among the other eight states, two reported participating in such working groups in the past but not currently. However, it appears that the New England states group (consisting of Maine, Massachusetts, New Hampshire, Rhode Island, and Vermont) and the WASHTO states are the only cases where a reasonably high degree of uniformity in regulations exists among the participating states and in which carriers commonly obtain one permit good in several states. In fact, a participant in one of the other regional groups noted that coordination efforts have "proved fruitless".

Infrastructure Damage Analysis and Assessment

An objective of this effort was to document the breadth and extent of processes used to analyze infrastructure damage and associated costs. A wide variety of methods is used to quantify the damage of the highway infrastructure caused by vehicles requiring overweight permits; 20 of the 27 states (74%) responding to the survey indicated that some approach was used to do this either for damage to pavements, bridges, or both. Table 3.3 provides the responses on this topic.

Table 3.3 Infrastructure Damage Analysis Techniques

Infrastructure Damage Analysis Techniques	Number of States (Percentage)
Agency-specific Approaches (Unspecified)[1]	7 (26%)
Bridge Analysis Rating System (BARS)	5 (19%)
AASHTO 1993 Pavement Design Guide	3 (11%)
AASHTO 1993 Pavement Design Guide + Mechanistic-Empirical Approaches	3 (11%)
Mechanistic-Empirical Approaches only	2 (7%)
None Reported	7 (26%)

[1] One state in this group reported use of a mechanistic-empirical approach

It can be seen from Table 3.3 that a wide variety of practices exist. Additionally, some states focus solely on bridges, some only on pavements, and some address impacts on both. It is also noteworthy that at least three states reported using M-E approaches in conjunction with the procedures in the 1993 AASHTO Pavement Design Guide, and that three other states reported using only M-E approaches to analyze impacts of overweight vehicles on pavements, for a total of six (22%) of states using M-E approaches. These methods included the Mechanistic-Empirical Pavement Design Guide under development through the National Cooperative Highway Research Program, and the Everstress Program developed by the Washington State DOT.

18 of 27 states (67%) did not report instances in which pavement distress was caused by a permitted overweight vehicle. The other states provided a wide range of responses to this question, in which some states stated that examples existed but that they could not give specifics and others noted that manual or falling weight deflectometer

surveys were done after passage of the vehicle. While it appears that little effort has been made to quantify such damage, a wide variety of opinions do appear to exist on the topic. One respondent noted "...our theory is that most of the pavement damage is a result of overweight vehicles", while another noted that "...most roadway damage is an accumulation of traffic loadings and weather." And not specifically attributing damage to overweight vehicles. It is important to note that these comments do not reflect the results of scientific, quantified study but are untested "theories".

About half (13 of 27) of the responding states take preventive measures to minimize damage impacts. Six states (California, Colorado, Florida, Louisiana, Tennessee, and Wyoming) may require more axles or changes in proposed axle spacing, one state (North Carolina) may require a change in routes, and four (Alabama, Minnesota, Missouri, and Ohio) may require any combination of these. One state (Illinois) reported doing before and after manual surveys, while one state (Texas) reported requiring nighttime travel to avoid the hottest part of the day and minimize impacts on flexible pavements in hot conditions.

Sharing of data collected in traffic monitoring and weight enforcement programs may be of assistance in the bridge and pavement design processes. 16 states (59%) reported doing so; however, three of these states reported that this is typically done only when requested or on a case-specific basis. Involvement of pavement and bridge design engineers in the permitting process is even more common. In general, nine states (33%) involve pavement and bridge offices, eight states (30%) involve only bridge offices, and one (4%) involves only pavement designers. Among the remaining nine states that responded to the survey, seven (26%) do so in specific cases; all of but one of these states does so specifically for superloads. Among these states, the definition of superloads ranges from in excess of 150,000 pounds to in excess of 254,300 pounds (Texas).

Most states attempt to analyze the impact of overweight vehicles prior to their occurrence – as noted above, 70% estimate impacts on pavements, bridges, or both, and 59% share such data with their pavement and/or bridge design office, far fewer go to the effort of measuring, analyzing, and assessing the actual impact of these loads. Such an effort would likely require greater resources associated with field visits and instrumentation. 21 of 27 states (78%) did not report doing such evaluations. Among the other six states that responded to the survey, two noted that specific evaluations are only done "in special cases", one noted that the fee-structure reflects such impacts (but did not note specific evaluations efforts), one refers to prior research studies, and two use pavement design approaches. One of these states (Washington) uses the program Everstress, based on mechanistic considerations, to develop incremental damage models and estimate change in time to next maintenance treatment. Although it appears that the survey question on this topic elicited some responses that did not directly address the question, it does appear that site-specific, field-level evaluations of infrastructure damage are very rare.

Weight Limits and Enforcement

The next section of the survey instrument was intended to gather information on factors that may allow for exceptions or changes to the general policy on weight limits in each state and to determine the extent of field devices used for traffic monitoring and enforcement purposes. Factors addressed in the questionnaire include seasonal climate effects on the pavement structure, commodities hauled, restrictions on particular routes or route systems, vehicle types, load type, and pavement condition. Regarding seasonal considerations, seven states (26%) reduce legal weight limits during the "spring thaw" period, during which pavement structures are more susceptible to damage due to the increased moisture contents and lower soil stiffness after an extended period of subfreezing temperatures. One state noted that limits are reduced for the period of September 15 – March 15, which is substantially longer than a spring thaw period that in some states lasts eight to ten weeks. 17 of 27 states (63%) have exceptions for certain commodities. In some states the list of excepted commodities is short (only coal in one state, only logs in another), while in some other states, the list of commodities that may exceed general limits is a page long, with different limits for different items. 15 of 27 states (56%) have reduced limits for entire systems (e.g., county roads) or for a select set of routes based on site-specific conditions. 14 of 27 states (52%) exempt certain types of vehicles from weight limits; commonly cited vehicles include fire trucks, ready-mix concrete trucks, construction equipment, and log hauling trucks. 9 of 27 states (33%) do not issue permits for divisible loads. 7 of 27 states (26%) consider pavement condition (rather than design) when issuing permits.

Survey respondents were asked to provide information on their states' equipment used to collect data for traffic monitoring purposes and for enforcement. 26 states responded to this set of questions. The results pertaining to enforcement purposes are shown in Table 3.4, and for traffic monitoring the results are presented in Table 3.5. Some states noted whether they had equipment but did not note the number of installations. The number in parentheses in the second column in each table indicated the number of states that supplied these numbers; this is the basis for the average and range data provided in the third and fourth columns.

Table 3.4 Weight Limit Enforcement Equipment

Purpose/Type	Number of States[1]	Average	Range
Static Weigh	26 (16)	19	1 to 102
Portable or Roadside WIM	19 (9)	10	1 to 14
Mainline WIM	9 (3)	-[1]	-[1]

[1] Insufficient data

Table 3.5 Traffic Monitoring Equipment

Purpose/Type	Number of States[1]	Average	Range
Static Weigh	21 (11)	14	2 to 51
Portable or Roadside WIM	16 (7)	6	3 to 14
Mainline WIM	18 (8)	11	1 to 18

Most states have an apparently small number of locations at which weights are regularly monitored, both for traffic monitoring purposes as well as for enforcement. Several states reported having no weigh-in-motion stations in traffic ("mainline" as opposed to "roadside"); however, it is suspected by the researchers that in most of these cases, the person or persons responding to the survey are not involved in traffic monitoring (more likely the responsibility of transportation planning unit rather than a permitting unit of a transportation agency). Therefore, the extent of traffic monitoring equipment deployed in the field, based solely on the survey results, is probably underestimated.

SUMMARY

Survey Administration

To document current practices regarding overweight vehicle permit fees, a questionnaire was sent to a targeted individual in each state, either with the state transportation department or highway patrol. 27 responses were received, for a response rate of 54%. The length of the survey, as well as the wide variety of expertise required to thoroughly complete it are assumed to have affected the response rate – in many states, the effort required to complete the questionnaire required the participation of at least three individuals, representing the state's permit, pavement design, and bridge offices.

Permit Processes and Coordination

In most states, fees for overweight permits are based on some combination of gross vehicle weight, axle weight and configuration, and mileage. A fee structure based on groups of GVW values and mileage is most common (30% of responding states), while a fee structure based solely on mileage is next most common (26%). Other factors that occasionally affect permit fees are commodity type and trip duration. 44% of states noted that their fee structure is set by legislative act; however, since this information appeared incidentally or as supplements to completed questionnaires, and was not a specific question in the survey, it is suspected that the true figure is much higher. In their conversations with the researchers, some survey respondents noted that they have little or no impact permit fees and that the fees themselves do not accurately reflect the cost of damage to the infrastructure associated with overweight vehicles. Most states (78%) now have online application processes. Regarding coordination among states, other than the

New England group and the WASHTO states, which appears to be the only working example with substantial uniformity of regulations among states and permits recognized in multiple states, efforts to increase uniformity and simplify processes for permit applicants appear to consist mainly of discussions.

Infrastructure Damage Analysis and Assessment

As part of the permit application process, most states (74%) conduct analyses of the impact of the subject vehicle on pavements, bridge, or both. Analysis approaches vary widely, but the Bridge Analysis Rating System (BARS) and a variety of agency-internal approaches are most common. Six states (22%) reported using or experimenting with mechanistic-empirical approaches, and most states report typically involving their bridge design units, and many the pavement design units. To reduce impact of overweight vehicles on the infrastructure, 48% of states noted that they may require alternate routing, axle configuration, or travel periods. Although sharing of data obtained through monitoring and enforcement programs with offices responsible for pavement and/or bridge design is common (59% of states), site visits to evaluate damage are rare. While 78% of states responded that they do *not* conduct field investigations of damage, interpretation of the comments provided by the other 22% of states responding to this question suggest that site-specific, field-level evaluations of infrastructure damage are very rare.

Weight Limits and Enforcement

There is an incredible diversity among states with respect to factors affecting weight limits associated with climate and commodity type. For example, changes in legal weight limits for the spring thaw period are very common in the northern states. The variety of commodities specifically addressed in regulations (typically statutes) may reflect the importance of various industries to their respective state's economies.

Most states have an apparently small number of locations at which weights are regularly monitored, both for traffic monitoring purposes as well as for enforcement. Several states reported having no weigh-in-motion stations in traffic ("mainline" as opposed to "roadside"); however, it is suspected by the researchers that in most of these cases, the person or persons responding to the survey are not involved in traffic monitoring (more likely the responsibility of transportation planning unit rather than a permitting unit of a transportation agency). Therefore, the extent of traffic monitoring equipment deployed in the field, based solely on the survey results, is probably underestimated.

CHAPTER 4

FRAMEWORK AND ANALYSIS

INTRODUCTION AND BACKGROUND

Although truck size and weight legislation in the U.S. originated in 1913 (*1*), it was only in the early 1960's, at the American Association of State Highway Officials (AASHO) Road Test, that pavement engineers and researchers began to precisely quantify the effects of axle loads on pavement damage (*21*). The so-called fourth-power relationship between pavement damage and axle load has been well-documented since then and has been characterized by a change in pavement serviceability resulting from the application of a known number of equivalent single axle loads (ESALs). More recently, efforts have been made toward characterizing pavement damage in terms of actual distresses through mechanistic-empirical (M-E) pavement analysis techniques (e.g., *25*). This approach more robustly predicts pavement distress as a function of stress or strain levels within the pavement cross-section.

As discussed in the literature review, M-E analysis is well suited to evaluate the damaging effects of overloads and has been used by a number of agencies (*27, 12*). A schematic presented by Fernando et al. (*27*), shown in Figure 4.1, accurately depicts the M-E approach to overload evaluation.

According to M-E analysis, the pavement cross section must first be characterized by its component materials, material properties and layer thicknesses. Loads can then be simulated in a mechanistic model to predict pavement response under specific loadings. For illustrative purposes, Figure 4.1 depicts tensile strain at the bottom of the asphalt layer (ε_t) to predict fatigue cracking while vertical strain at the top of the subgrade (ε_v) is used to predict structural rutting. Other pavement types or cross sections may have different critical response locations and parameters. These critical responses are then related to pavement performance through so-called transfer functions, as discussed in the literature review. The transfer functions are responsible for generating the distress vs. time plot shown in Figure 4.1. Assuming a critical level of distress, the time to resurfacing or major rehabilitative work can be predicted as a function of increasing load levels.

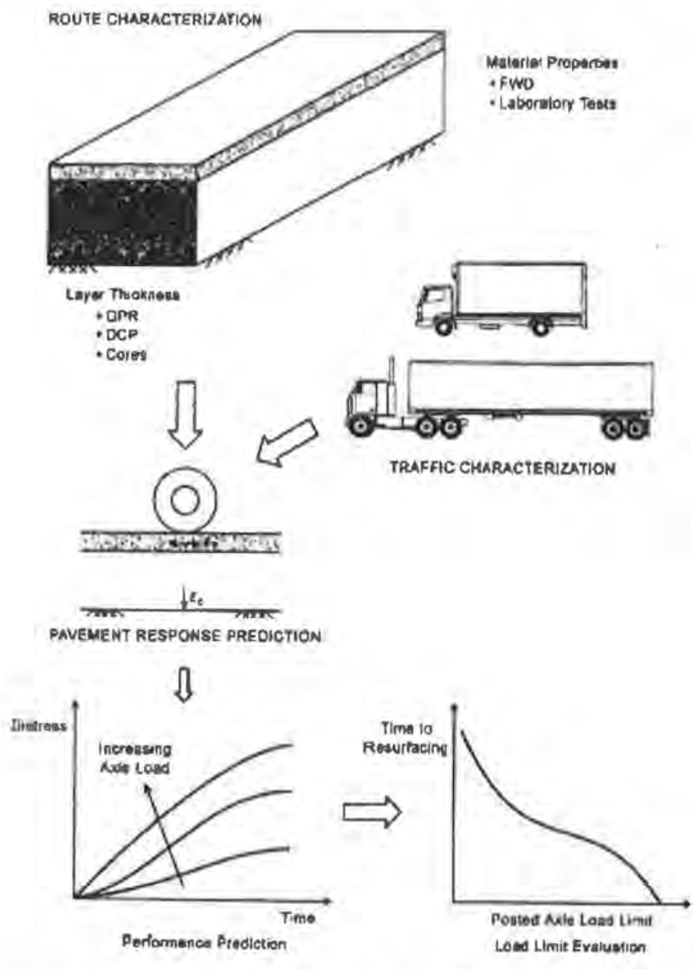


Figure 4.1 Framework Used to Develop Program for Load-Zoning Analysis (*4*).

The final step in analysis, not presented in Figure 4.1, is to translate the reduced time between rehabilitation efforts into an increased cost to the agency. As discussed by Barros (*8*), this can be accomplished by well-established net present value computations derived from a cash-flow diagram shown schematically in Figure 4.2.

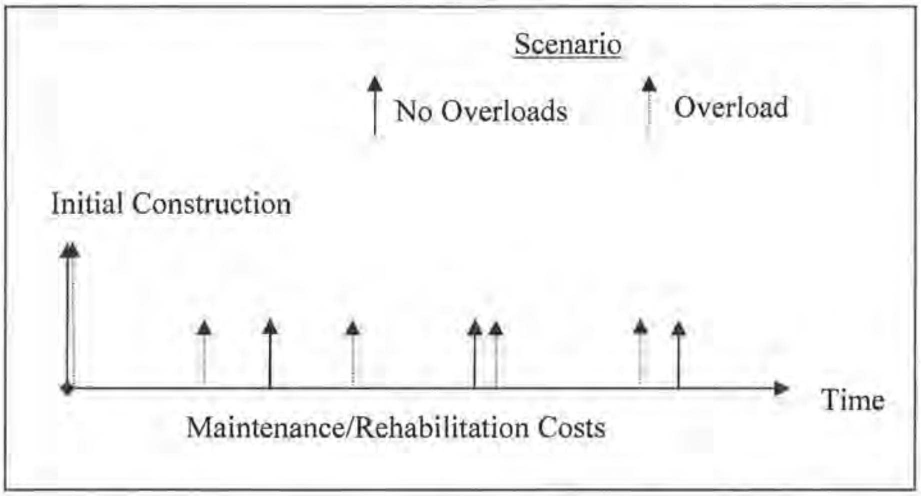


Figure 4.2 Life Cycle Cost Analysis Cash Flow Diagram (*after 6*).

M-E AND LCCA FRAMEWORK

The concepts described above are combined in Figure 4.3 illustrating the entire M-E and life cycle cost analysis framework for evaluating overloads (Scenario B) against a baseline load distribution (Scenario A). This framework can be expanded to accommodate numerous alternative scenarios, but only two are shown here for clarity.

Regardless of the number of scenarios, they are all applied to the same pavement section (i.e., cross-section, material properties and current condition) that is entered into the mechanistic model for the computation of pavement response. This is to ensure that various loading scenarios are evaluated equally.

After establishing the pavement cross section, the traffic must be characterized. In Figure 4.3, Scenario A can be thought of as the "baseline" load distribution. This is the traffic that is currently using the facility under normal or expected conditions. Scenario B represents some arbitrary overloaded condition. This could be permitted overloads, changes due to increased load limits or any number of alternative situations. Regardless, according to the M-E approach, the load spectra for each scenario must be defined by the constituent axle types, loadings and frequencies. These are then simulated on the pavement section in the mechanistic model. More discussion of traffic characterization is provided under "Loading Scenarios" below.

The M-E model in Figure 4.3 predicts the pavement distress over time for the various loading scenarios. Presumably, Scenario B, having heavier loads, will result in more frequent maintenance and rehabilitation efforts to maintain the structure at the same level of performance.

The next step in the process is to use life cycle cost analysis (LCCA) to determine the increase in cost resulting from the more frequent maintenance and rehabilitation efforts. To maximize accuracy in determining actual costs, a detailed study of interest rates, material and labor cost rates would be required. In some cases, this level of detail may be warranted and agency records can be consulted to determine these costs. In other cases, however, simply knowing the percent increase in cost would be very useful from a planning perspective. In this situation, actual costs would not be needed as long as the cost per work activity was held constant between the scenarios.

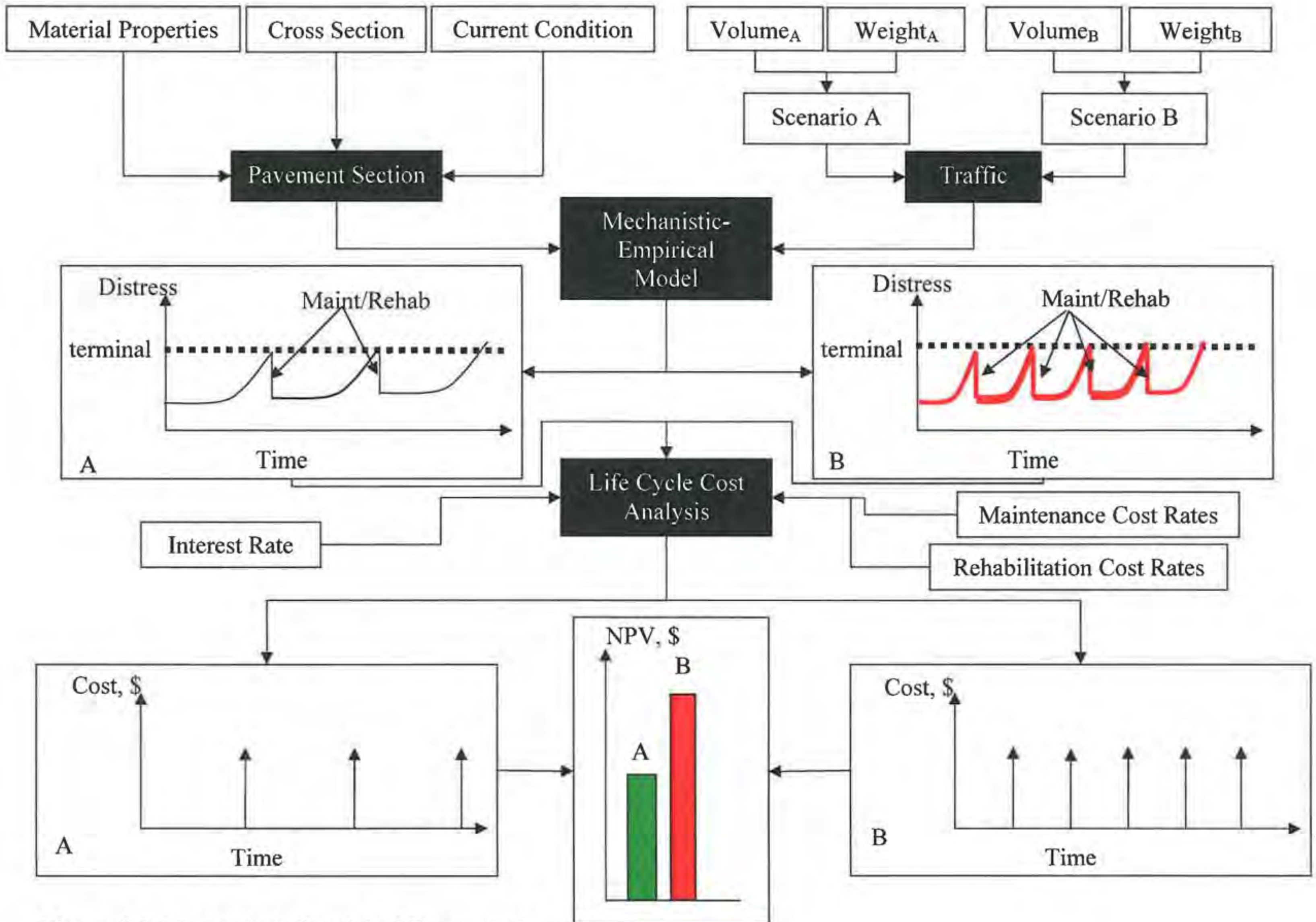


Figure 4.3 Mechanistic-Empirical Framework.

MEPDG

Putting the above concepts into practice can be accomplished, in part, by using the Mechanistic-Empirical Pavement Design Guide (MEPDG). This guide, developed under NCHRP Project 1-37A (*25*), and currently under review by AASHTO to replace the existing 1993 Pavement Design Guide (*23*), is a comprehensive M-E design and analysis package. Though it requires local calibration for maximum accuracy, it does contain representative default inputs in terms of material properties, load spectra and performance predictions that facilitate pavement performance analysis. Though not specifically designed to handle overload analysis, the software can be manipulated to accomplish this task. A full discussion of the MEPDG, its development and use is beyond the scope of this report, but has been documented elsewhere (*25*).

LOADING SCENARIOS

With respect to overloaded or permitted vehicles, there are literally thousands of potential scenarios to consider. Therefore, a framework, as described above is needed to accommodate the many scenarios that could be encountered. In the context of this report, three general categories are considered. First is the situation where entire load spectra are shifted toward heavier loads. This scenario would potentially result from state or federal agencies increasing load limits across a network. The second situation considers when particular axles are overloaded. This scenario would result from permitting of individual vehicles or when there are known illegal overloads. The third situation includes altering the axle configuration on trucks with specific axle weights. This scenario would arise from state or federal agencies allowing the gross vehicle weight limit to be increased;

therefore, the rear tandem axles of some trucks would need to be replaced by tridem axles in order to carry the additional weight. Each of the categories is discussed below.

Shift Entire Load Spectra

When legal limits are increased, it is expected that the corresponding load spectra will also increase. While a methodology to predict changes in load spectra resulting from changes in legal limits is beyond the scope of this work, it is necessary to develop a knowledge base of how pavement damage may increase due to heavier load spectra. Figure 4.4 illustrates three load spectra. The baseline spectrum is the default single axle load distribution contained within the MEPDG. The other two spectra used in the framework developed herein, represent shifting the loads by 3,000 and 7,000 lb, respectively. Similar shifts can be applied to tandem, tridem and quad axles. This may be a simplistic view of how load spectra may change, but it does serve as a starting point short of a comprehensive investigation to predict changes due to new regulations. In reality, increasing load limits may result in stretching of the load spectra over a wider range of loads rather than simply shifting into higher weight ranges. Further study is recommended in this area.

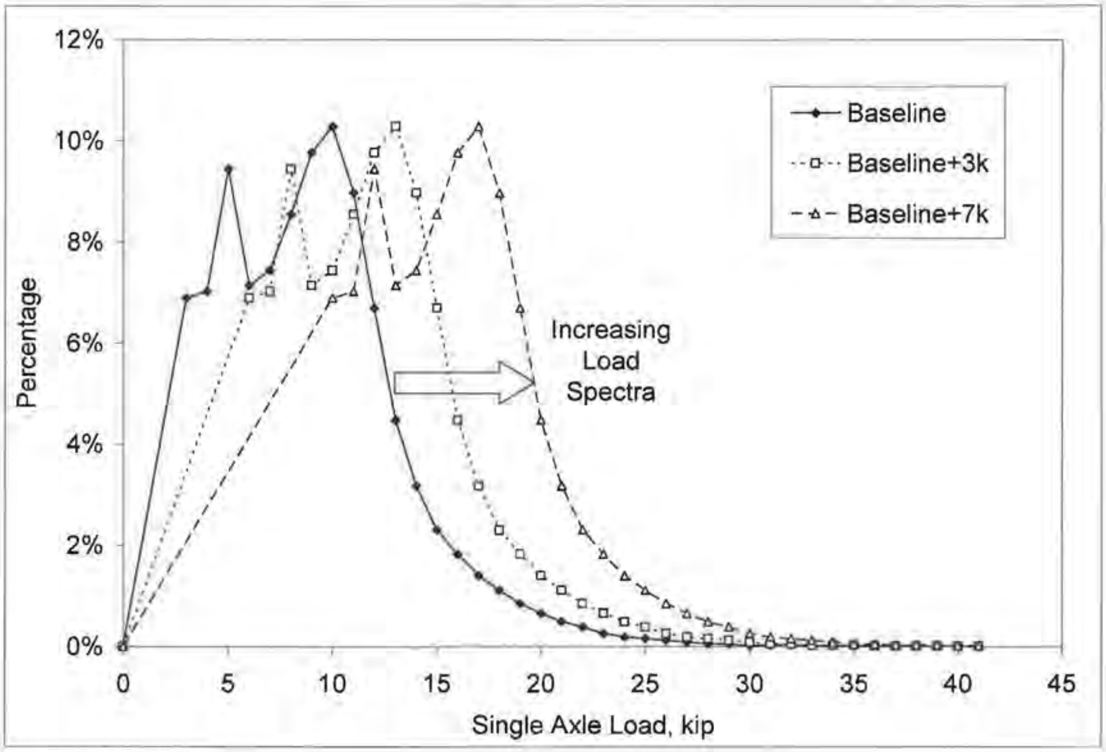


Figure 4.4 Single Axle Load Spectra Shift.

Permitting Specific Axles

A more common situation arises when specific axles are permitted to operate above legal limits. So-called routine permitting requires no special investigation at the state level, but these permitting weights vary widely amongst the states as discussed in the literature review. In the context of permitting, two sub-scenarios will be considered and discussed below:

1. Constant Volume – Increased Weight
2. Decreased Volume – Constant Weight

Constant Volume – Increased Weight

As the name implies, this scenario fixes the total axle volume, in terms of axles/day, while reassigning a certain percentage of the axle repetitions to the permitted (or overloaded) axle weight. By way of further explanation, consider the hypothetical single axle load distributions shown in Figure 4.5. The baseline load distribution is identical to that shown in Figure 4.4 (MEPDG default load distribution), except expressed in axles/day rather than a percentage. Adding up the number of axles at each load level results in 1,535 single axles/day. Multiplying each load magnitude by the corresponding volume results in $7,034*10^3$ tons/day. The permitted distribution represents the condition where 5% of the axle volume below the permitted weight (24,000 lb in this case) has been permitted to operate at 24,000 lb. 5% of the volume at each load level below 24,000 lb has been reassigned to the 24,000 lb increment; thus the dramatic change from less than 5 axles/day at 24,000 lb to nearly 80 axles/day. Slightly less dramatic is the 5% change in the volume for each load level less than 24,000 lb. In this scenario, it was assumed that loads heavier than the permitted weight were left unaffected. The net result for the permitted scenario is the same number of axles per day (1,535) but an increase of $570*10^3$ tons/day due to the increased number of 24,000 lb axles.

It should again be noted that many other scenarios where the traffic volume remains constant are possible. For example, it is possible that only the heavier axles, all ready close to the permitted weight, would begin operating at the permitted weight, leaving the more lightly loaded axles unaffected. A more detailed investigation of how permitting activities affect load spectra would allow for a more informed and realistic representation of changes in load spectra but is beyond the scope of this investigation.

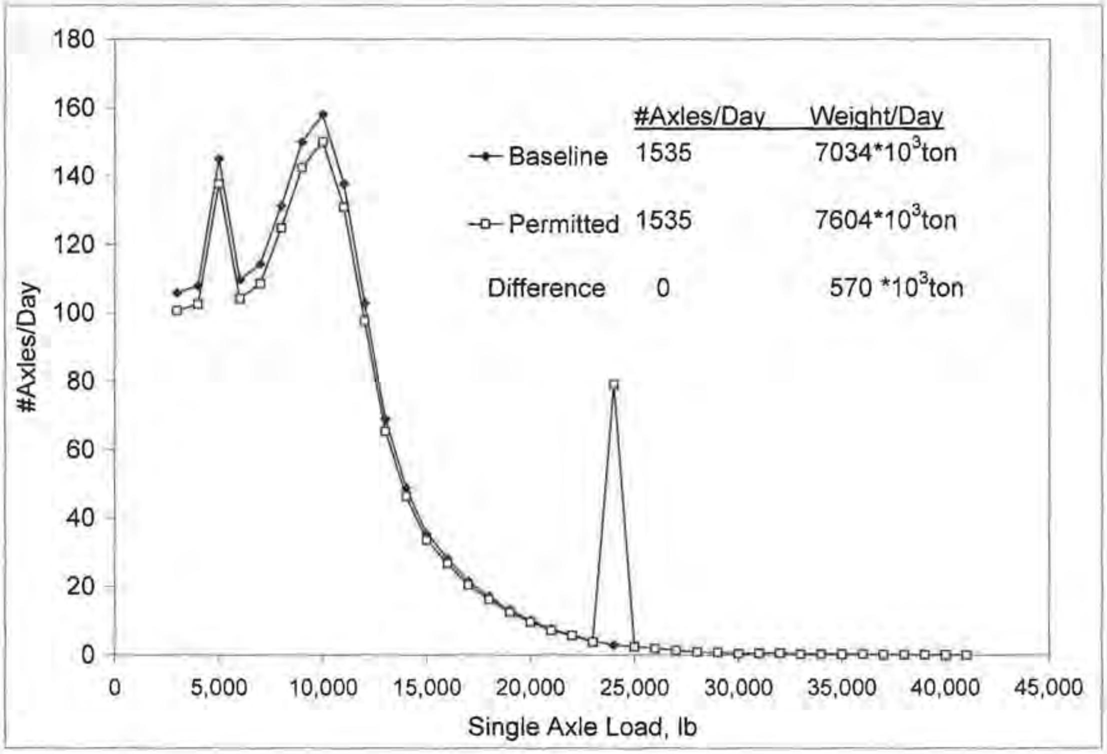


Figure 4.5 Constant Volume – Increased Weight Example.

Decreased Volume – Constant Weight

The second, and perhaps more realistic, scenario fixes the total weight which results in fewer total trips due to more efficient use of the trucking fleet. Figure 4.6 illustrates this situation with the same hypothetical baseline single axle spectrum and targeted single axle load of 24,000 lb. The baseline volume and weight were again 1,535 axles/day and 7,034*10^3 tons/day, respectively. However, rather than determining 5% of the volume up to the permitted axle, 5% of the weight up to the permitted axle was calculated by:

$$W_{T<P} = \sum_{i=1}^{p-1} W_i * N_i \tag{4.1}$$

where:

$W_{T<P}$ = total weight of axles up to the permitted axle

W_i = weight of i^{th} axle

N_i = frequency of i^{th} axle (axles/day)

The number of additional 24,000 lb axles needed to carry this load was then found by:

$$N_{P+} = \frac{W_{T<P}}{\frac{W_p}{Axle}} \tag{4.2}$$

where:

N_{P+} = number of additional axles at P load level

W_p = weight of axle P

The net result of applying equations 4.1 and 4.2 is the same total load level (7,034 $*10^3$ tons/day) but with 48 fewer axle passes.

Again, there are a large number of potential scenarios where the weight would remain constant while the volume changes due to permitting activities. Such analyses could include different approaches to load spectra shifts and ranges of values for percentages of axles affected. Further investigation is warranted.

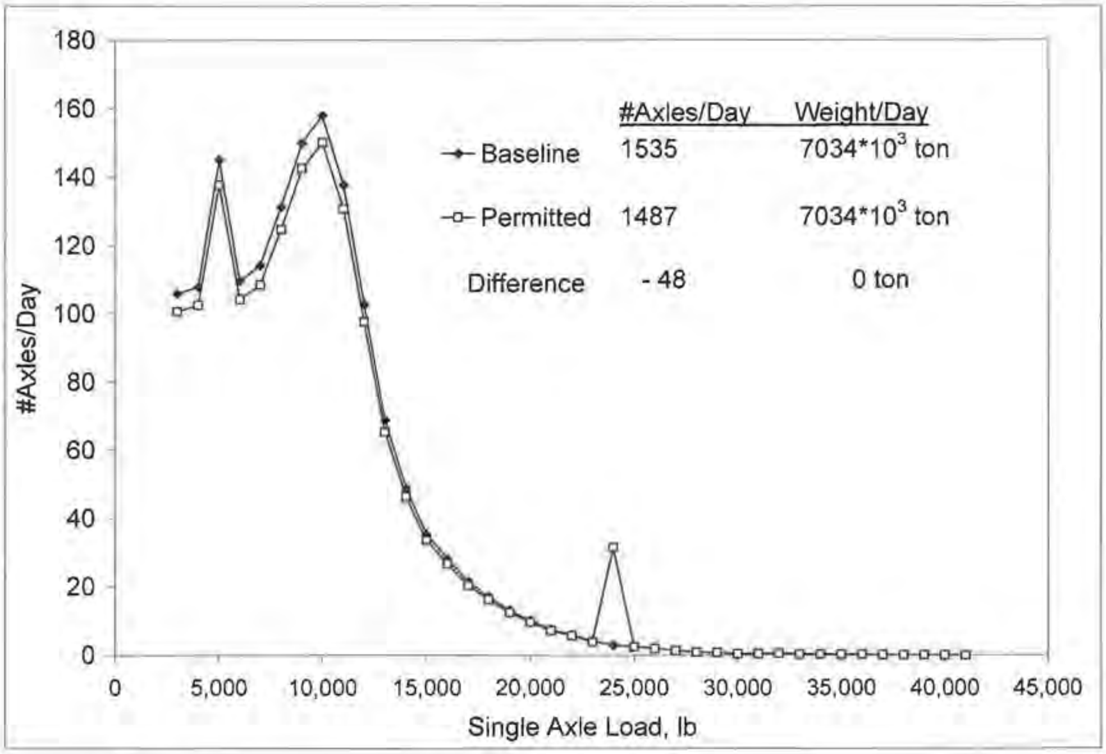


Figure 4.6 Decreased Volume – Constant Weight Example.

Changing Axle Configuration

The last scenario involves changing the axle configuration on certain trucks so that they will be able to carry more weight. This was achieved by maintaining a constant weight in the traffic stream while decreasing the volume. For this case study, a rear tandem axle weighing 34,000 lb was replaced by a 51,000 lb tridem axle. This was done at varying percentages of tandem axle volume. Figures 4.7 and 4.8 illustrate this concept. In Figure 4.7, a baseline tandem (MEPDG default load distribution) is shown along with the various percentages of tandem axles that are changed to tridem axles. As expected, as the percentages increase, the amount of 34,000 lb tandem axles decreases. Figure 4.8 is the complement to this graph, as it shows the baseline tridem distribution, and the 51,000 lb tridems added for each of the corresponding respective percentages. To determine the amount of tridem axles required to replace tandem axles while maintaining a constant weight, first the percent of volume chosen was subtracted from one, and then multiplied by the number of 34,000 lb tandem axle passes/day. For example, if the percentage selected was 5%, and there were 200 tandem axles/day, then the number of tandems would decrease to 190 axles/day. Then, the number of fewer tandems (in this case 10), were multiplied by 2/3 to find the number of added 51,000 lb tridems, which for this example would be 6.7. Therefore, if there were an average of 100 tridem axle passes/day, that number would increase to 106.7.

Although shown in Figures 4.7 and 4.8, replacing 100% of 34,000 lb tandem axle sets with tridem axle sets is not realistic. This percentage was used in the study for data analysis and illustrative purposes only.

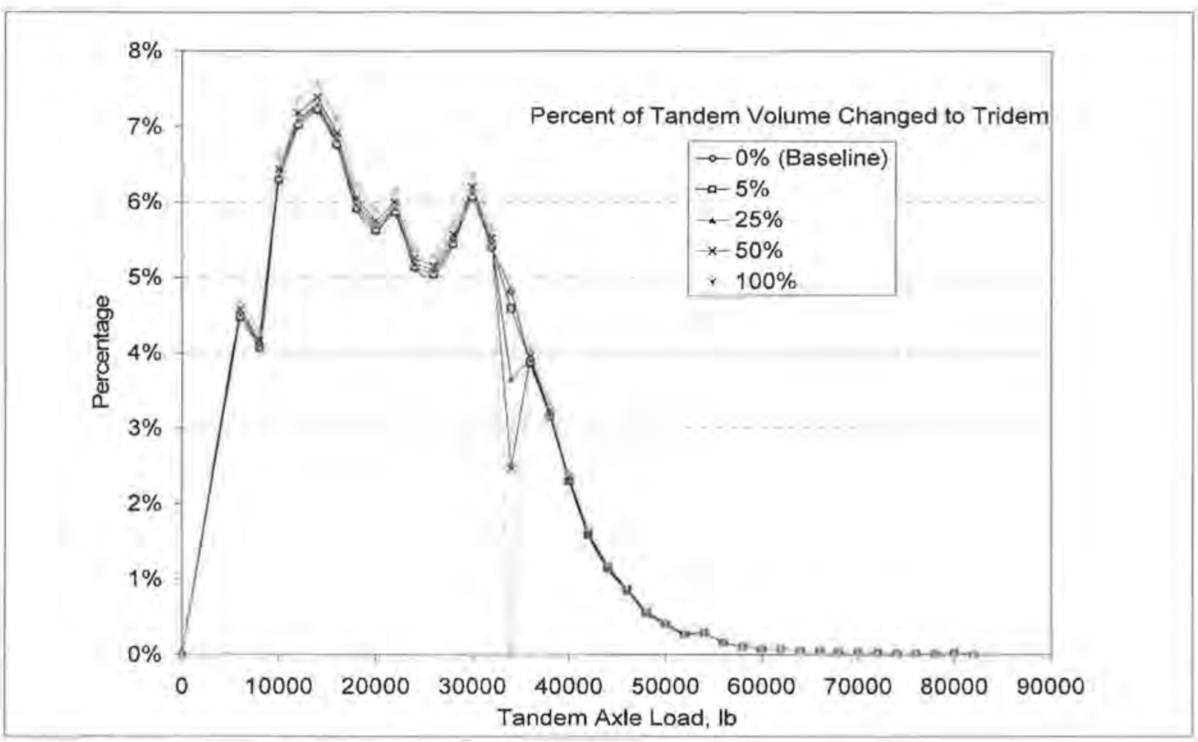


Figure 4.7 Changing Axle Configuration – Decreased Tandems

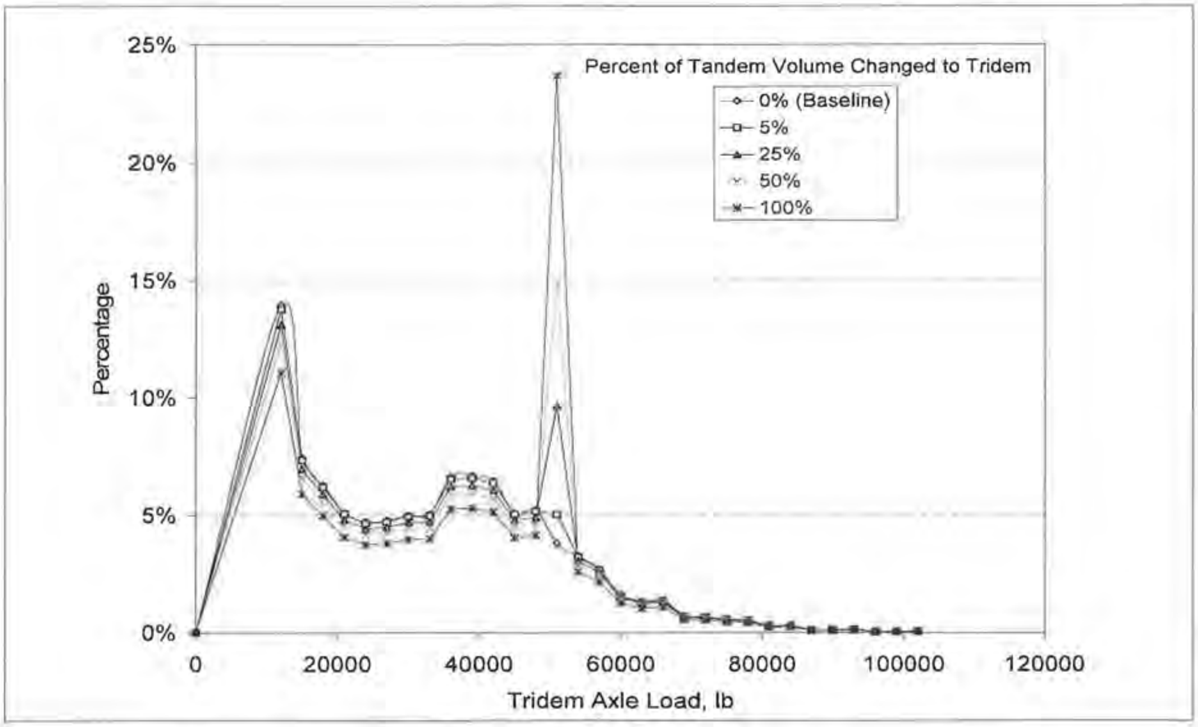


Figure 4.8 Changing Axle Configuration – Increased Tridems

Overloads in the MEPDG

Like most M-E methods of pavement analysis and design, the MEPDG uses load spectra to characterize the traffic. Within the software, there are a number of modules that facilitate data entry and modification of default load spectra. These modules, listed in the screen capture shown in Figure 4.9, include tables for seasonal adjustments, vehicle types, hourly distributions, etc. The most important of these is the "Axle Load Distribution Factors" that allows the designer to enter the probability distribution for each axle type by vehicle type as illustrated in Figure 4.10. As the figure shows, single, tandem, tridem and quad axles can be considered on a monthly basis.

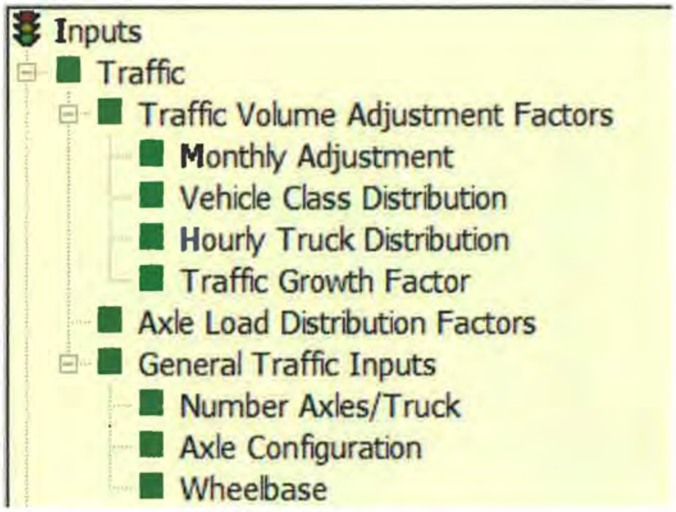


Figure 4.9 MEPDG Traffic Data.

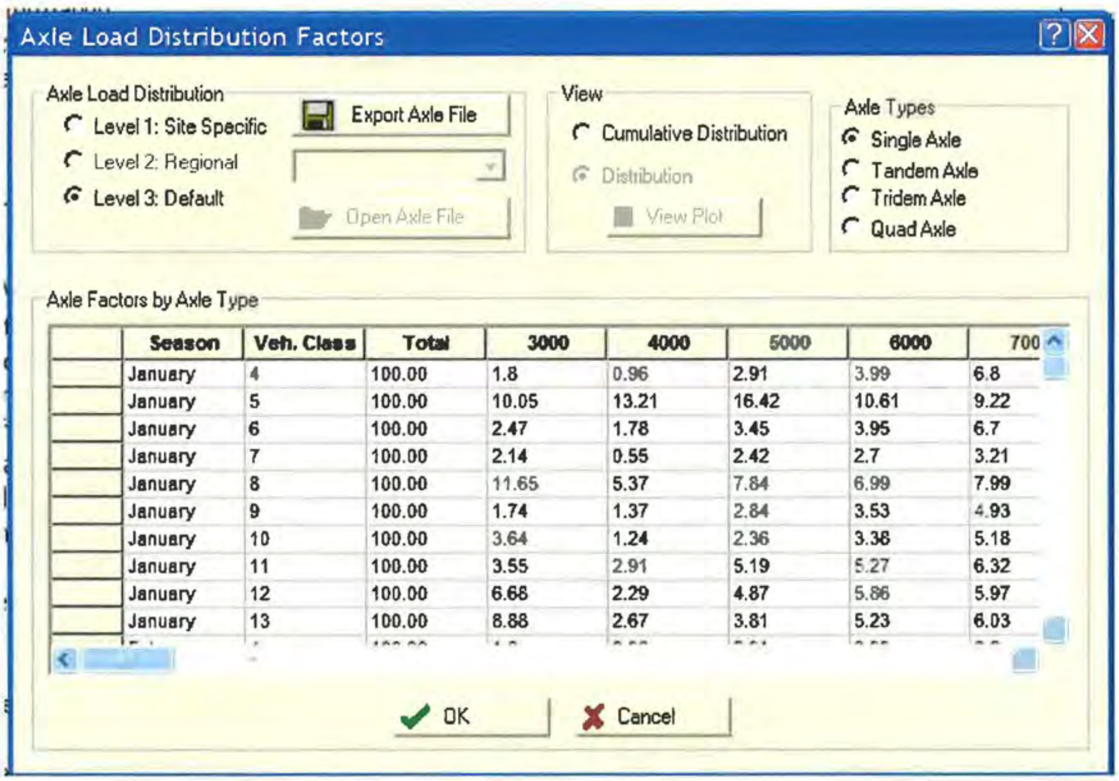


Figure 4.10 Axle Load Distribution Factors.

While the tables shown in Figure 4.10 could be modified to accommodate the types of loading scenarios described above, the user interface is a bit cumbersome for this type of activity. However, after searching through the MEPDG software for methods of modifying the load spectra, and consulting with the developers of the MEPDG, it was concluded that the most efficient way of handling the loading scenarios was to manually modify the temporary traffic files the MEPDG creates during simulation. These temporary files are created from the load spectra shown in Figure 4.10, but simply contain the average axles/day in each axle type and load magnitude for each month of analysis in a given design scenario.

From a practical standpoint, the procedure for modifying the files is as follows:

1. Execute a simulation with the baseline traffic.

2. Locate the temporary load spectra files created by the MEPDG. These are located in the project folder generated by the MEPDG and have the following designations:

    _SingleAxleOutput.csv

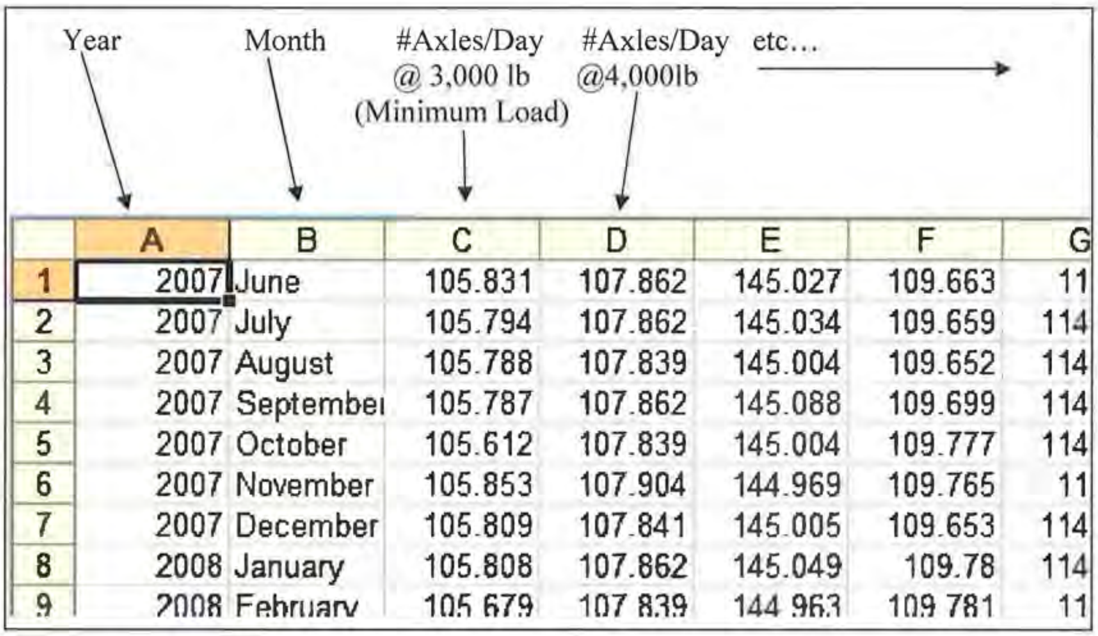
    _TandemAxleOutput.csv

    _TridemAxleOutput.csv

    _QuadAxleOutput.csv

3. Modify each file as necessary to simulate the permitted or overloaded axles. It is important to understand the file format since there is no published information regarding these files and how they are organized. Figure 4.11 illustrates the format of _SingleAxleOut.csv. The first two columns indicate the year and month of analysis.

The next column represents the average number of axles per day at the first load magnitude which depends on the axle type. For example, as shown in Figure 4.11, there are 105.831 single axles weighing 3,000 lb in June, 2007. Each column after that contains the average number of axles/day at the next increment of load. Each axle type has an identical file format, except that the minimum load and load increment depend upon the axle. Table 4.1 lists the pertinent load spectra parameters for each of the axle types.



	A	B	C	D	E	F	G
1	2007	June	105.831	107.862	145.027	109.663	11
2	2007	July	105.794	107.862	145.034	109.659	114
3	2007	August	105.788	107.839	145.004	109.652	114
4	2007	Septembei	105.787	107.862	145.088	109.699	114
5	2007	October	105.612	107.839	145.004	109.777	114
6	2007	November	105.853	107.904	144.969	109.765	11
7	2007	December	105.809	107.841	145.005	109.653	114
8	2008	January	105.808	107.862	145.049	109.78	114
9	2008	February	105.679	107.839	144.963	109.781	11

Figure 4.11 _SingleAxleOutput.csv File Format.

Table 4.1 MEPDG Axle Group Weights

Axle Type	Minimum Load, lb	Load Increment, lb	Maximum Load, lb
Single	3,000	1,000	41,000
Tandem	6,000	2,000	82,000
Tridem	12,000	3,000	102,000
Quad	12,000	3,000	102,000

To modify the files according to the loading scenarios provided above, several different approaches are needed. First, and perhaps simplest, is to shift the entire load spectra a certain amount above the baseline. In this case, data can be simply copied and pasted the required number of columns over corresponding to the desired weight shift, then adding the leftover percentages to the last column. For example, a 1,000 lb weight shift in single axles would require copying column C to D, D to E and so forth, and then adding the values of the last column to the one before it, which is easily done in Excel.

Secondly, targeting specific axle weights for permitting or overload analysis required the creation of two conversion templates corresponding to the constant volume-increase weight and decreased volume-constant weight scenarios, respectively. These templates require the designer to copy the raw baseline load spectra into the Excel template. Then, a target load is selected for permitting or overload analysis. The designer also enters the percent of volume (constant volume-increase weight) or the percent of weight (decreased volume-constant weight) to be considered. The template makes the necessary adjustments to the original load spectra, following the equations and methodologies presented in the Loading Scenarios section above. The modified load spectra can then be copied back into the original load spectra file for use by the MEPDG in simulation.

Finally, changing a percentage of tandem axles to tridem axles required the creation of an additional conversion template. This template requires the designer to copy and paste the raw baseline tandem and tridem load spectra into the Excel template. Next, a percent of volume to change is entered. The template calculates the amount of tandem axles to subtract, and the amount of tridem axles to add. The modified data can then be copied and pasted back into the original load spectra files and used for the next MEPDG simulation.

4. Save the modified files and mark them as "Read Only." This is a critical step in that the MEPDG, when executing a simulation will try to overwrite the modified files, containing the new load spectra, unless they are marked as read only.

5. Re-run the analysis using the new, overloaded, load spectra.

It is important to note that the MEPDG contains a special module to facilitate analysis of specific, user-defined loading conditions. Pictured in Figure 4.12, this module allows the designer to select specific tire spacing, axle spacing and load magnitude for up to twelve simultaneous tires. While this module is useful for evaluating a particular load configuration, it does not allow the designer to integrate specific loadings with existing load spectra. Therefore, the so-called "Special Axle" module does not exactly meet the needs of overload analysis in the context presented above. However, it is important to be aware of this module as it would be useful for other types of studies such as evaluating unique facilities with special loading configurations.

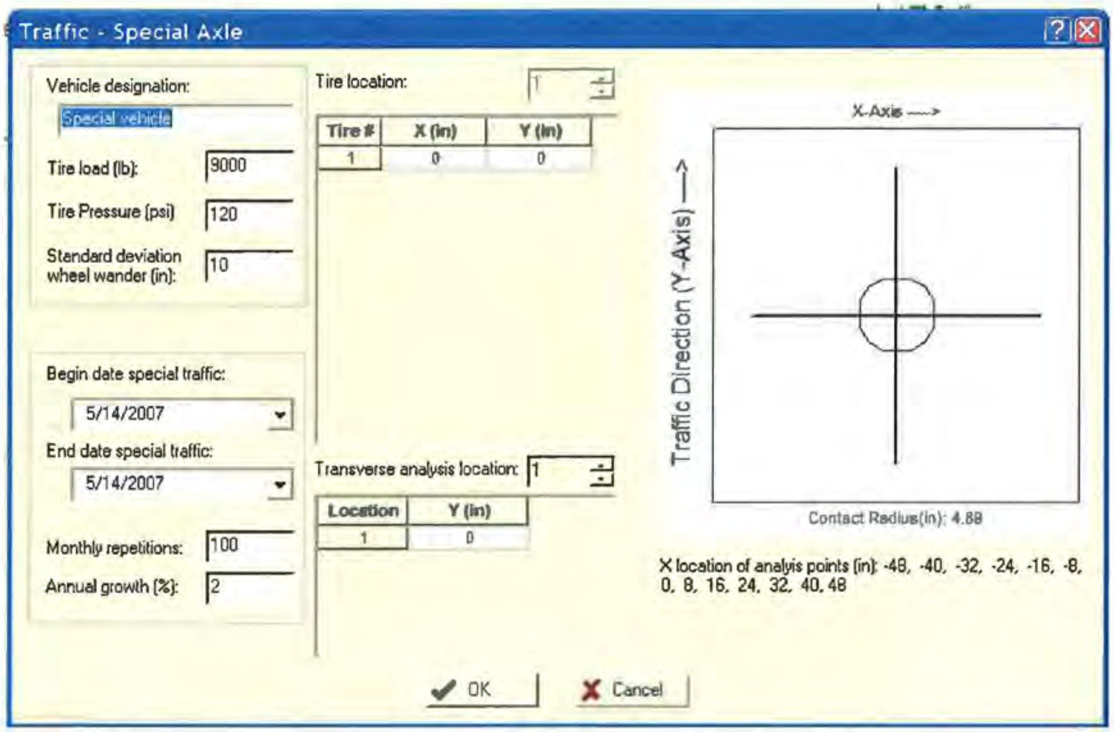


Figure 4.12 MEPDG Special Axle Module.

LIFE CYCLE COST ANALYSIS

Translating the reduction in pavement life resulting from increased loads into cost is an important consideration for agencies responsible for load regulation, permitting and enforcement. To achieve that end, LCCA is a powerful and much needed tool. FHWA has published a technical guide (*30*) on LCCA best practices that contains both simple analysis tools and complex theory on accounting for variability of the input data and work zone user costs. For pavement type selection, these considerations are important since they may vary between design alternatives and may be the driving factor between selecting one pavement over another. However, in the context of overload analysis, where a single pavement structure is to be evaluated with various loading scenarios, factors such as input variability and work zones activities would not vary and therefore could be ignored for simplicity.

As presented by Barros (*8*), and discussed in the FHWA LCCA guide (*30*), the straightforward net present value (NPV) calculation can be used as a fair comparison between various scenarios:

$$NPV = I.C. + \sum_{k=1}^{N} R.C._k \left[\frac{1}{(1+i)^{n_k}} \right] \tag{4.3}$$

where:

I.C. = initial construction cost

R.C. = recurring rehabilitation/maintenance costs

k = rehabilitation/maintenance activity

i = interest rate

n = year in which rehabilitation/maintenance occurs

Because the overload analysis considers the same pavement structure under various loading conditions, the initial construction cost can be ignored. This leaves only the recurring rehabilitation/maintenance costs to vary as a function of loading scenario. Assuming for the purposes of overload analysis that the same activity would be required regardless of the loading scenario, just at different frequencies, then a ratio of the NPV of the costs associated with the overload scenario to the NPV of the costs associated with the baseline case can be developed. This ratio, or "cost factor" can be defined by:

$$CF = \frac{NPV_{overload}}{NPV_{baseline}} \tag{4.4}$$

Where:

CF = cost factor

$NPV_{overload}$ = net present value of life-cycle rehabilitation and maintenance costs in overloaded condition

$NPV_{baseline}$ = net present value of life-cycle rehabilitation and maintenance costs in baseline condition

To illustrate this concept, consider a simple example where a baseline case has a 20-year design life while an overloaded case results in a two-year life reduction to 18 years, depicted in Table 4.2. Over a 60-year analysis period, with an interest rate of 4%, the cost factor is 1.13 (13% increase in NPV from the baseline to the overload scenario). The dollar amounts can change, but the cost factor will remain constant so that this result can be applied to a wide range of pavements.

Table 4.2 Example LCCA Analysis

				Baseline
Year		Cost		NPV
20	$	1,000,000	$	456,387
40	$	1,000,000	$	208,289
60	$	1,000,000	$	95,060
		Total	$	759,736
				Overload
Year		Cost		NPV
18	$	1,000,000	$	493,628
36	$	1,000,000	$	243,669
54	$	1,000,000	$	120,282
		Total	$	857,579
		CF =	1.13	

REPRESENTATIVE CASES - INPUTS

To demonstrate the framework detailed above, a number of hypothetical scenarios were analyzed with the MEPDG. The following sub-sections detail the performance parameters, climate, pavement cross sections, material property and traffic inputs used in the simulations.

Performance Parameters

Unlike the current AASHTO Design Guide (*23*) that determines required pavement thickness as a function of loss of serviceability; the MEPDG predicts and designs for specific modes of distress. Tables 4.3 and 4.4 list the performance limits for flexible and rigid pavement designs, respectively. Three criteria, rutting, fatigue cracking and pavement roughness (IRI), were used for flexible pavement design. Slab cracking, joint faulting and roughness were used for rigid pavement design. All the parameters were analyzed at 90% reliability and a pavement was considered failed when any of predicted distresses exceeded the limit. These criteria represent the default conditions and limits within the MEPDG. A 20-year design period was targeted for the baseline case. Once the pavement cross section was established to meet this time frame, the various loading scenarios were applied to measure the reduction in pavement life.

Table 4.3 Flexible Pavement Design Performance Parameters

Performance Criteria	Limit
Initial IRI (in/mi)	63
Terminal IRI (in/mi)	172
AC Bottom Up Cracking (Alligator Cracking) (%):	25
Permanent Deformation (AC Only) (in):	0.25
Permanent Deformation (Total Pavement) (in):	0.5

Table 4.4 Rigid Pavement Design Performance Parameters

Performance Criteria	Limit
Initial IRI (in/mi)	63
Terminal IRI (in/mi)	172
Transverse Cracking (% slabs cracked)	15
Mean Joint Faulting (in)	0.12

Climate

For this investigation, the Alabama climate file available as part of the MEPDG was used. This climate file is used by the MEPDG, in conjunction with the enhanced integrated climate model (EICM) to predict states of moisture and temperature from which pavement properties such as modulus can be derived. It must be noted that using a different climate file could result in different results; therefore the solutions provided herein are somewhat climate specific. Future investigations could examine climatological effects.

Pavement Cross Sections

As shown in Figure 4.13, the scenarios all considered a pavement built upon 15 inches of good quality crushed stone base over a relatively poor quality soil (A-6). The moduli shown in Figure 4.13 were automatically estimated by the MEPDG based upon the material selection. The full complement of material properties for each sub-structure pavement layer are listed in Appendix A.

Figure 4.13 Pavement Sub-Structure.

Both asphalt and concrete pavement surfaces were considered in this analysis. The asphalt was a PG 76-22, selected to meet the requirements of the warm Alabama climate. The other properties were selected to be representative of typical Superpave mixtures and are listed in Appendix B. Given the pertinent input properties, the MEPDG divides the total asphalt thickness into sublayers and computes the modulus on a monthly basis. Figure 4.14 illustrates the change in modulus over the first year of analysis for the thickest pavement design considered in this analysis. The HMA modulus was similar for the other pavement cross-sections. Notice that the modulus tends to increase during the colder months and decreases when the temperatures increase. Also, the modulus tends to decrease with depth due to more moderate temperature changes within the pavement.

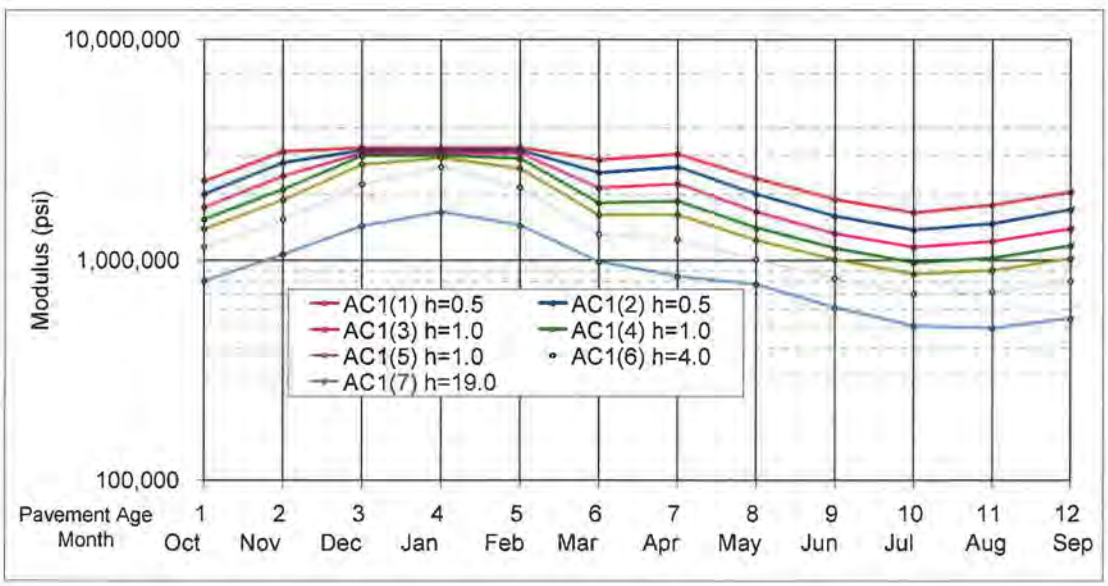


Figure 4.14 HMA Modulus for Thickest HMA Cross Section.

For the rigid pavement designs, the concrete properties were selected to be representative of typical paving-grade mixtures. A modulus of rupture equaling 690 psi was entered from which the MEPDG calculated a Young's modulus of 4,403,280 psi. Joint spacings of 15 ft with dowels providing load transfer were also selected. It should be noted that future analyses could examine continuously reinforced concrete pavement (CRCP). Complete details regarding the concrete input properties may be found in Appendix C.

TRAFFIC

The overload analysis, as described previously, examined shifting the entire load spectra, permitting specific axles, and altering the axle configuration. In all three cases, in addition to both flexible and rigid pavement designs, the same baseline traffic data were used. Another factor was the volume of traffic. Four levels were selected so that a range of pavement thicknesses would be generated. The following subsections detail the traffic volume, baseline load spectra and overloaded scenarios, respectively.

Traffic Volume

The MEPDG requires traffic volume to be entered as the average annual daily truck traffic (AADTT). The two-way volumes were selected to represent typical pavements in Alabama ranging from lower-volume routes to higher-volume interstates. A 50% directional split and 95% lane distribution factor were assumed, in addition to a 4% compound annual growth factor. Table 4.5 lists the four traffic volumes, as AADTT, in addition to trucks in the design lane (after applying the 95% and 50% factors). The other pertinent loading information is contained within Appendix D.

Table 4.5 Two-Way Traffic Volumes

Traffic Level	AADTT (Trucks/Day)	Trucks/Day in Design Lane
Low	250	119
Low-Medium	1,000	475
Medium-High	4,500	2,138
High	8,000	3,800

Baseline Axle Load Spectra

Figure 4.15 contains the baseline axle load spectra for the single, tandem and tridem axles used in the analysis, respectively. These load spectra were generated automatically by the MEPDG based upon the default vehicle distribution built into the software. Figure 4.16 illustrates the frequency of each axle type. It should be noted that the default axle load spectra within the MEPDG do not contain quad axles.

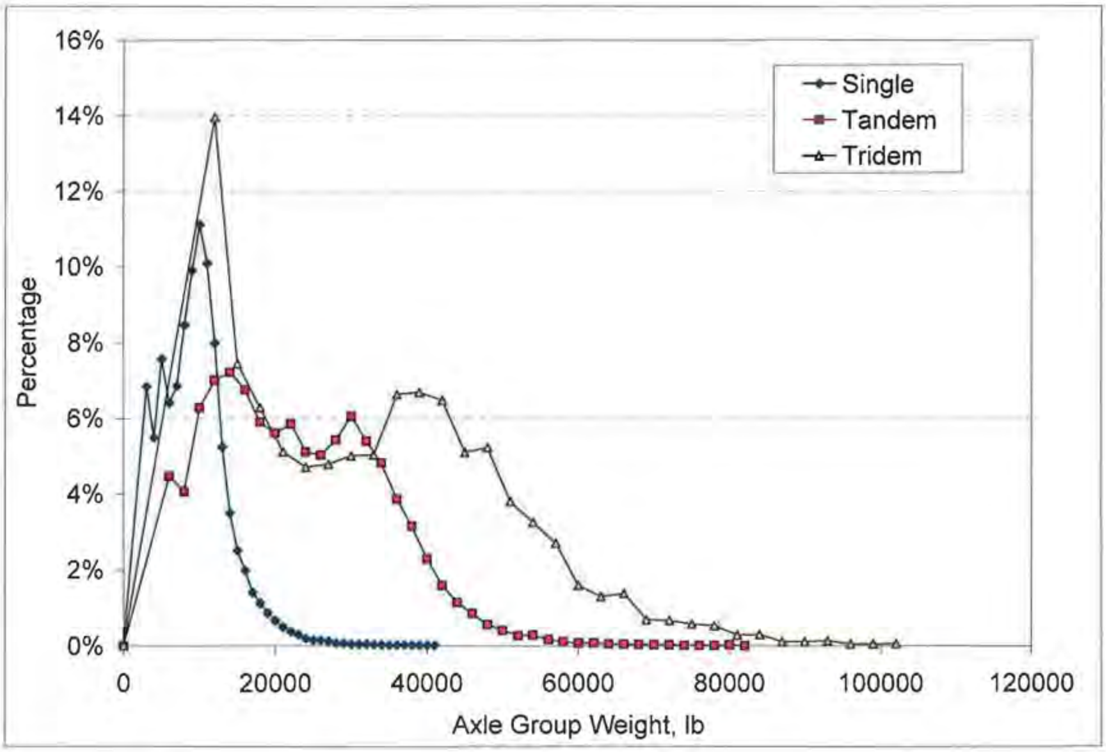


Figure 4.15 Baseline Axle Load Spectra.

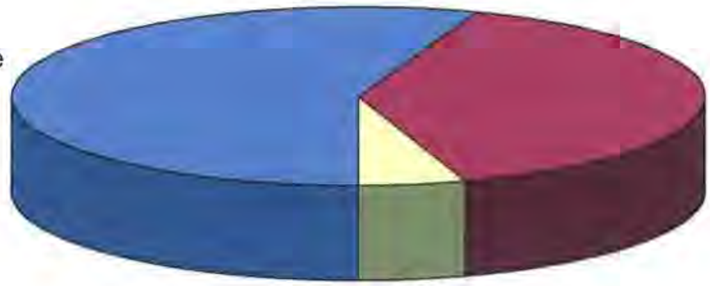


Figure 4.16 Baseline Axle Type Distribution.

Shift Entire Load Spectra

To evaluate the effect of increasing loads throughout the load distribution, the load spectra were shifted to successively higher weight ranges. Table 4.6 shows the three cases considered (A, B and C) to represent a range of load shift scenarios. The increase in load per axle was held constant across the three axle types, but the total weight axle group depended upon the number of axles in the group. Figures 4.17, 4.18 and 4.19 illustrate the various load spectra considered in this part of the analysis, respectively. It should be noted that the load shift case (A, B or C) was applied to all three axle types simultaneously. For example, when simulating case A, a 3,000 lb/axle shift was applied to the single, tandem and tridem axle groups. Further investigations could be done to examine other scenarios such as shifting only one axle type, or shifting the axle types by differing amounts.

Table 4.6 Load Shifting Cases

		Load Shift per Axle Group (lb/Axle Group)		
Load Shift Case	Load Shift per Axle (lb/Axle)	Single Axle	Tandem Axle	Tridem Axle
A	3,000	3,000	6,000	9,000
B	5,000	5,000	10,000	15,000
C	7,000	7,000	14,000	21,000

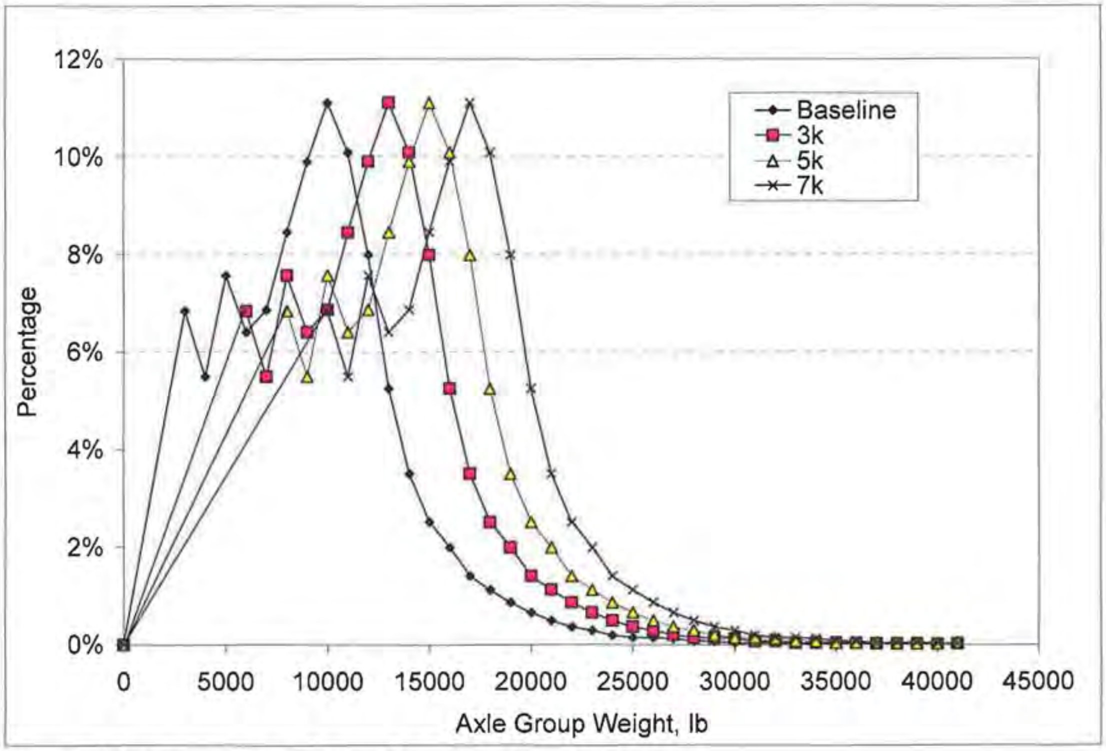


Figure 4.17 Single Axle Load Spectra Comparison.

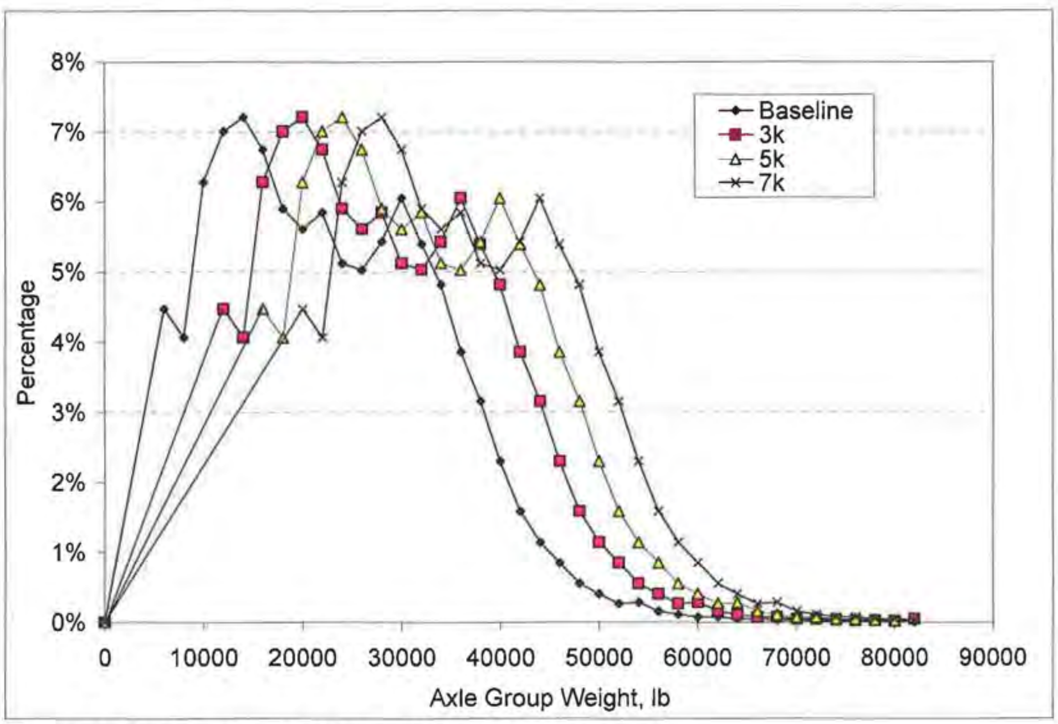


Figure 4.18 Tandem Axle Load Spectra Comparison.

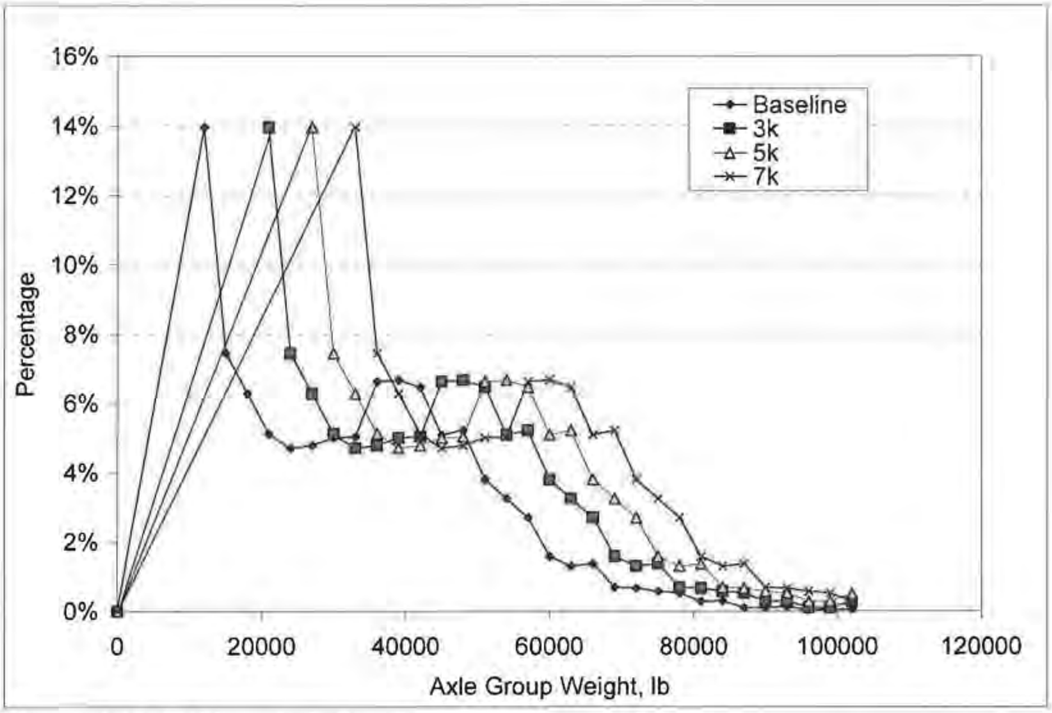


Figure 4.19 Tridem Axle Load Spectra Comparison.

Permitting Specific Axles

The permitting of specific axles considered both "Constant Volume – Increased Weight" and "Decreased Volume – Constant Weight" scenarios as stipulated previously. For each of these cases, average (Case D) and maximum (Case E) routine permitted axles were simulated as listed in Table 4.7. The average permitted weights were computed from the routine permitting weights published by the Comprehensive Truck Size and Weight Study (*28*). The maximum permitted weights were taken from the same data set (*28*) and represent the routine permitting in Washington, D.C. For this study, each case considered both the single and tandem axles simultaneously. Also, three percentages of permitted axles (1%, 10% and 20%) were used to cover a wide range of conditions. As discussed previously, these values are the percentage of lower weighted axles reassigned to the permitted or targeted axle weight. Other scenarios could be investigated in the future examining singles and tandems separately or examine a range of permits resulting in very complex load spectra.

Table 4.7 Permitted Axles

Permitted Axle Case	Axle Type	Weight, lb
D (Average)	Single	24,000
	Tandem	46,000
E (Maximum)	Single	31,000
	Tandem	62,000

Figures 4.20 and 4.21 illustrate the load spectra, for cases D and E, for the constant volume-increased weight (CV-IW) and the decreased volume-constant weight (DV-CW) scenarios, respectively. Similarly, Figures 4.22 and 4.23 illustrate the corresponding tandem axle load spectra for the CV-IW and DV-CW scenarios, respectively. It should be noted that these figures are specific to the 4,500 AADTT cases while the other volumes (250, 1,000 and 8,000) are linearly related to these plots. In any case, the net reduction in trips, resulting from fewer heavier axles required to carry the same total weight, is apparent in both sets of figures. For example, in comparing Figures 4.20 and 4.21 the 24,000 lb single axle at 10% yields a reduction of approximately 200 axle passes per day when switching from CV-IW (≈350 axles/day) to DV-CW (≈150 axles/day).

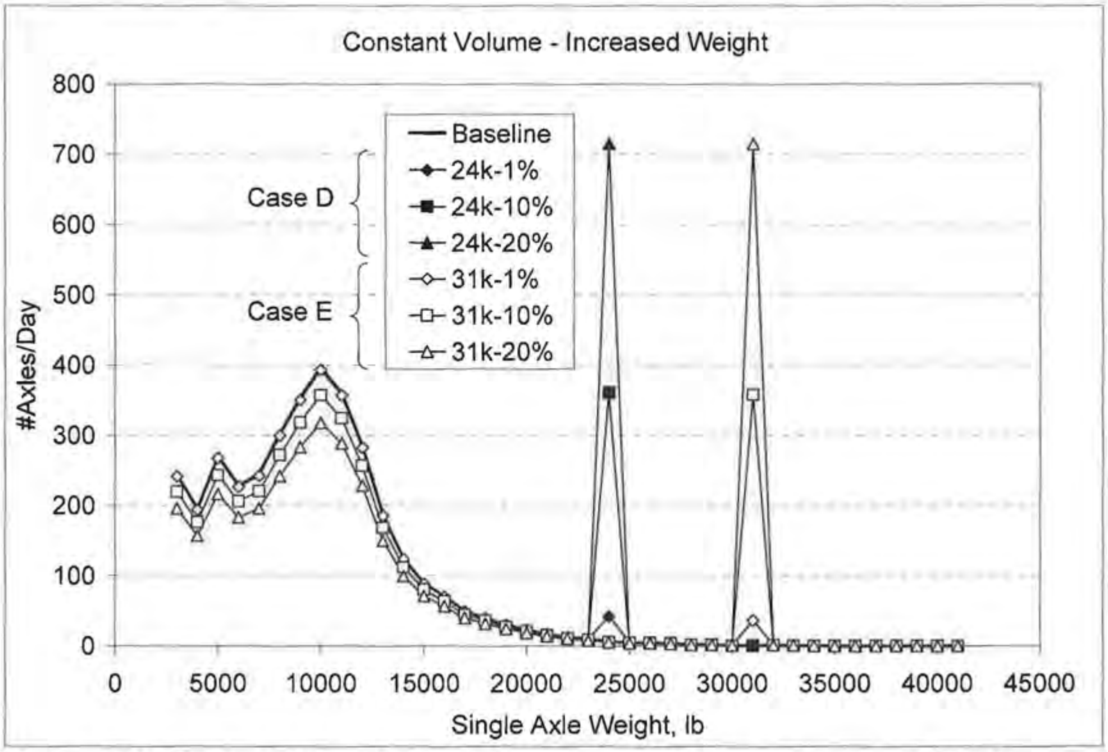


Figure 4.20 Single Axle Load Spectra – Constant Volume – Increased Weight.

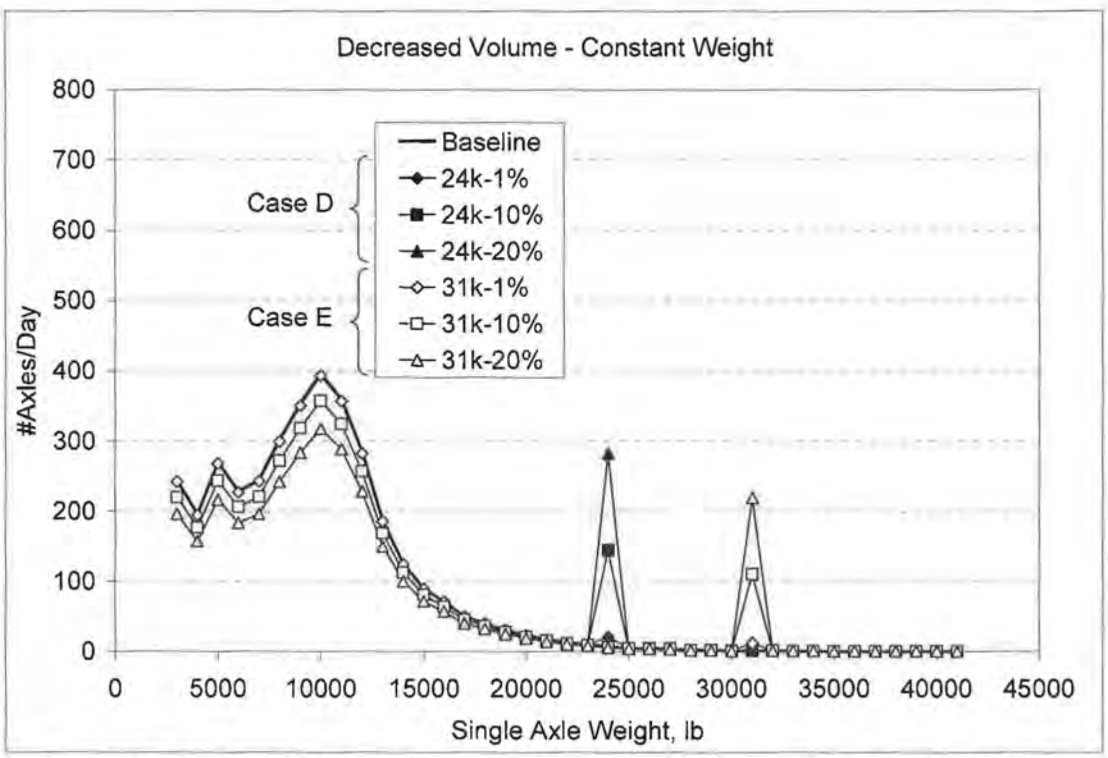


Figure 4.21 Single Axle Load Spectra – Decreased Volume – Constant Weight.

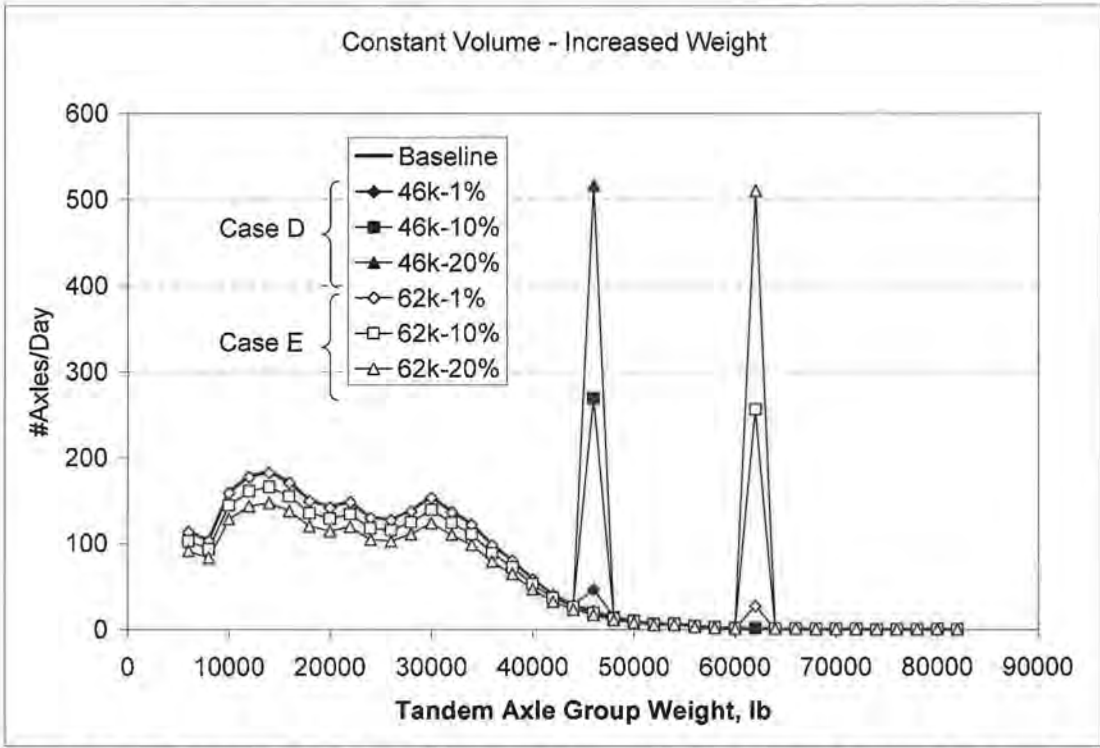


Figure 4.22 Tandem Axle Load Spectra – Constant Volume – Increased Weight.

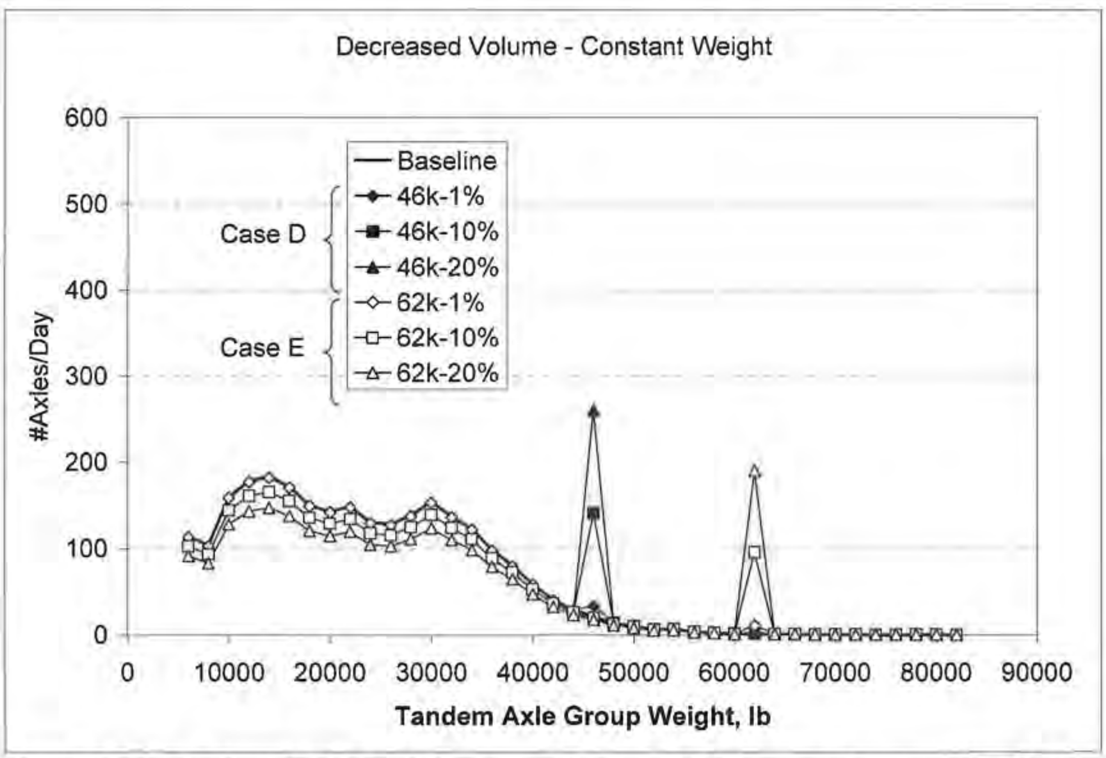


Figure 4.23 Tandem Axle Load Spectra – Decreased Volume – Constant Weight.

Changing Axle Configuration

In order to evaluate the effect on pavement performance, load spectra files were modified so that rear tandem axles weighing 34,000 lb were replaced by 51,000 lb tridem axles at various volume percentages, while maintaining a constant weight in the truck fleet. This was done for all traffic levels previously listed in Table 4.5. The volume percentages selected were 0% (baseline), 5%, 25%, 50% and 100%. Again, using a reduction of 100% tandem axle volume is not realistic, it was used in this study as an extreme case.

RESULTS AND DISCUSSION

Shift Entire Load Spectra

Damage Analysis

The first set of simulations examined shifting the entire load spectra at various increments. The surface design thicknesses determined through the MEPDG are summarized in Figure 4.24. As expected, there is a general trend of increasing the required thickness with increasing truck traffic volume. It is important to note, as will be explained later, that the minimum allowable concrete thickness in the MEPDG is 6 in. Therefore, as seen in the Figure 4.20, it can be surmised that the pavement designed for 250 AADTT was overly conservative since the 1,000 AADTT case also only required 6 in. It is also important to note that the HMA at 8,000 AADTT is extremely thick and may not be a practical pavement design. However, as it was a result of using the MEPDG, it was included in this analysis.

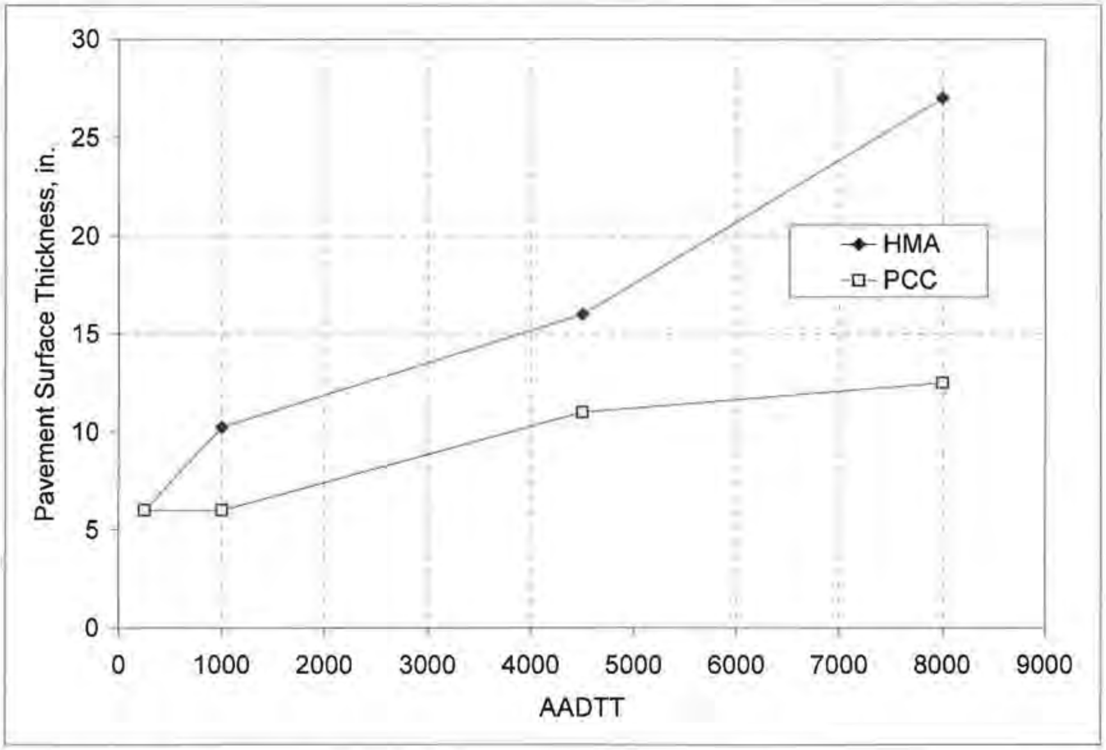


Figure 4.24 Pavement Thickness Summary.

After the required thicknesses were determined for the baseline traffic (no load shift), the pavement cross-sections were re-evaluated using the increased load spectra (3000, 5000 and 7000 lb). Figures 4.25 and 4.26 summarize the data for the HMA and PCC designs, respectively. The figures show, within a given truck traffic volume, the decline in pavement life resulting from the shifting load spectra. For example, Figure 4.25 shows that shifting the load spectra 3000 lb at an AADTT of 8000 decreases the HMA pavement life 6 years; from 20 years to 14 years.

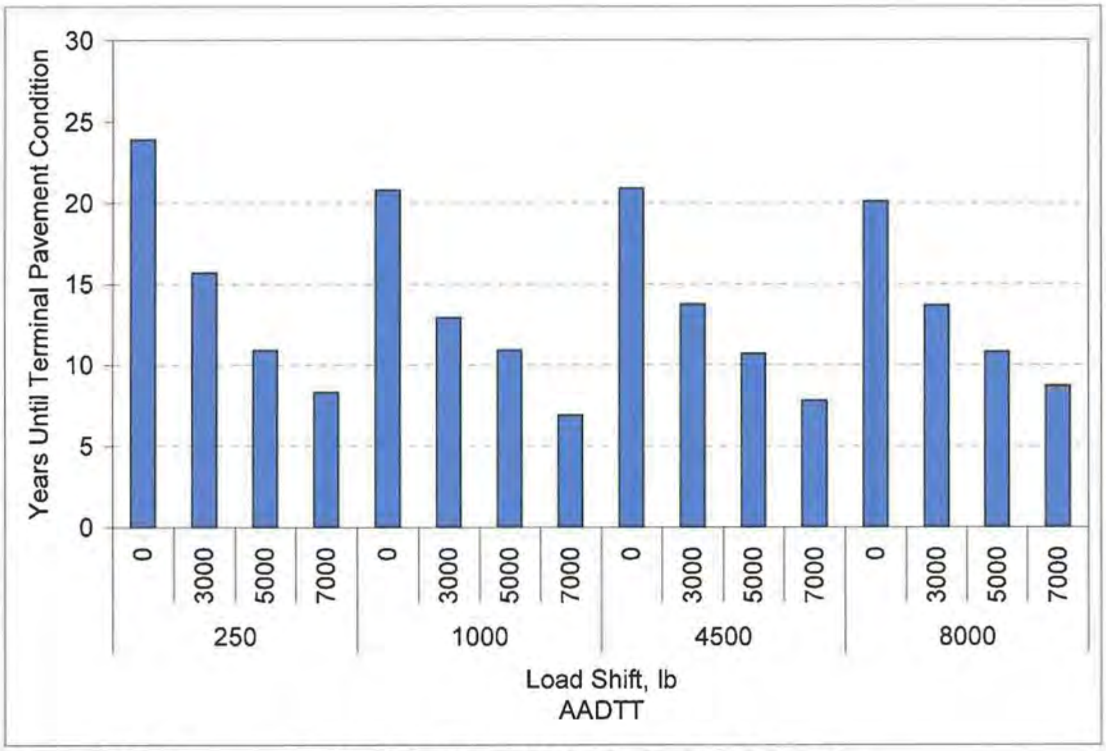


Figure 4.25 Pavement Life – Flexible – Shift Entire Load Spectra.

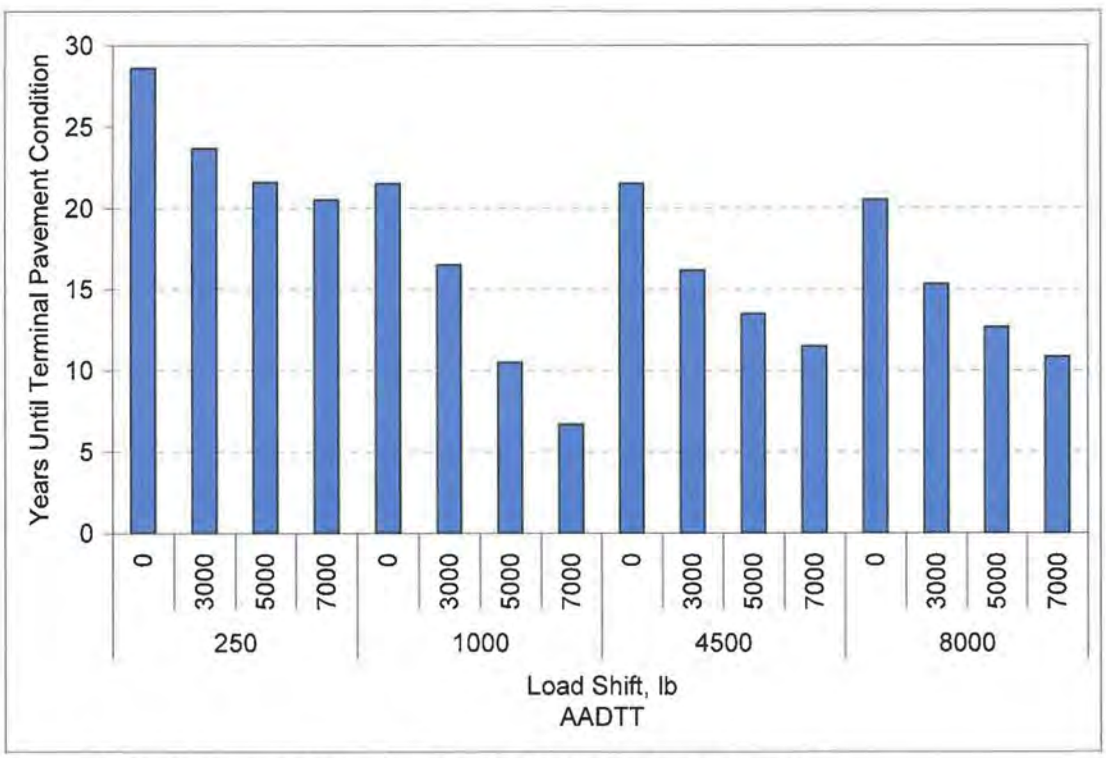


Figure 4.26 Pavement Life – Rigid – Shift Entire Load Spectra.

In general, the decrease in pavement life appears to follow a negative exponential trend with respect to the amount of load shift. Figure 4.27 illustrates this trend for the 250 AADTT flexible pavement case, which are the same data points as the first four bars from Figure 4.25 plotted on an arithmetic scale. A negative exponential function was fit to the data with the form:

$$Life = \alpha_1 e^{-\alpha_2 \cdot Shift} \tag{4.5}$$

Where:

Life = years until terminal pavement condition

Shift = load spectra shift, lb

α_1, α_2 = regression coefficients

α_1 can be thought of as the pavement life with no load shift (baseline case) while α_2 represents the reduction in pavement life per pound of load spectra shift. The high R^2 indicates that interpolation between data points will provide reasonable estimates of pavement life without having to run the actual cases. For example, shifting the load spectra 2,000 lb would result in a pavement life reduction of approximately 6.3 years.

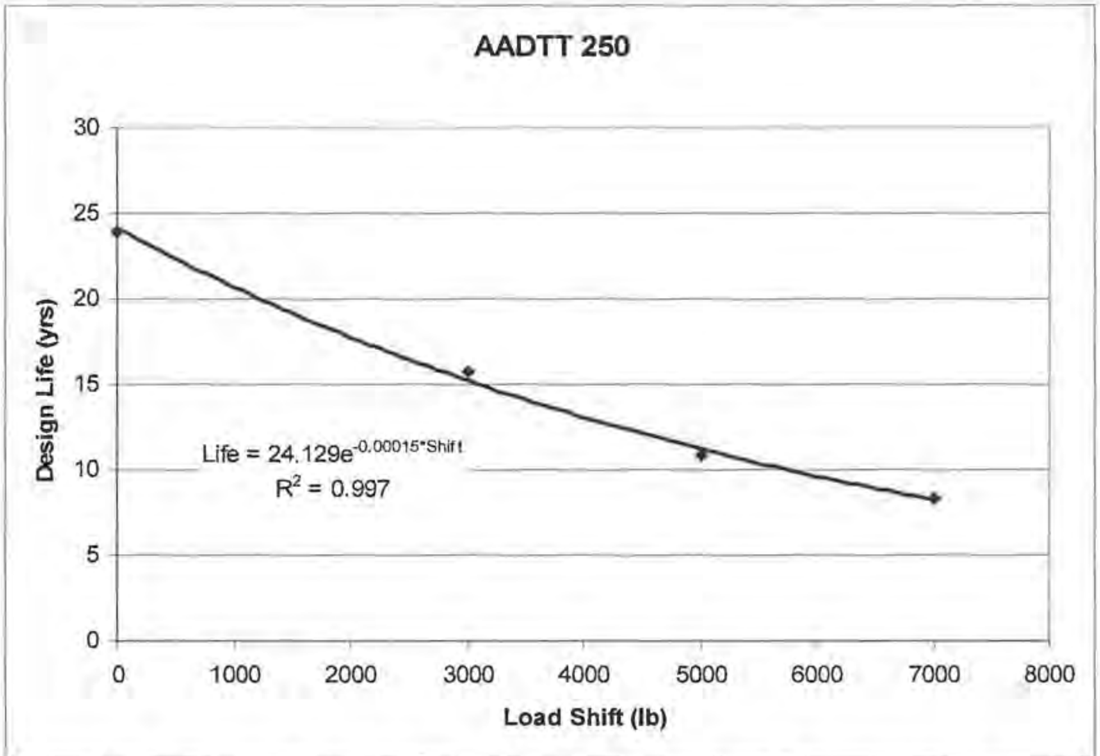


Figure 4.27 Effect of Shifting Entire Load Spectra on HMA Pavement–250 AADTT.

Regression was conducted for all the other load shift cases, for both asphalt and concrete designs, the results of which are summarized in Tables 4.8 and 4.9. The first observation is that in every case the model has a very high R^2 value, signifying a very good fit for the negative exponential model. Therefore, it can be stated for the cases investigated here that shifting load spectra in increments has an approximately negative exponential effect on the resulting pavement life. One could argue that a higher-order polynomial would fit the data better; however with only four data points per graph, this would make the curve fit artificially high by matching the degrees of freedom to the order of the polynomial. Also, with a minimum R^2 of 0.95 (among the scenarios evaluated), the negative exponential model provides an excellent fit and therefore a higher order polynomial is unwarranted.

The second observation, based on Tables 4.8 and 4.9, is that there does not appear to be a general trend in α_2 with increasing truck volume. This is especially true for the rigid pavement designs where between the 250 and 1,000 AADTT cases, α_2 more than triples then drops again between the 1,000 and 4,500 AADTT cases. It was previously stated that the MEPDG required a minimum PCC thickness of 6 in. which resulted in an overdesigned pavement for the 250 AADTT case. Because of this overdesign, shifting the load spectra had less of an effect (resulting in a smaller slope) than for the other cases where the designs were optimized.

The slight and erratic effect that traffic volume has upon the slope is very similar to the effect that pavement thickness has on axle load equivalency factors in the current 1993 AASHTO Pavement Design Guide (*23*). To make this connection it is important to first understand that traffic volume and pavement thickness are highly correlated; all else being equal, higher volume pavements require thicker cross-sections. Secondly, equivalent axle load factors are a relative measure of pavement damage that are analogous to the reduction in pavement life presented in this study. Bearing these two thoughts in mind, Huang (*31*) cited the erratic effect of pavement thickness (analogous to traffic volume) on equivalent axle load factors (analogous to pavement life reduction) in the equivalent axle load tables published by AASHTO (*23*). This seems to indicate the unique nature of a given pavement design and that each design should be treated individually. Huang (*31*) also goes on to state that the equivalent axle load factors are only slightly affected by the pavement thickness, which is consistent with observations in this study with regard to pavement damage.

Table 4.8 Non-Linear Regression Results – Flexible

AADTT	α_1	α_2	R^2
250	24.13	$-15*10^{-5}$	0.997
1,000	21.00	$-15*10^{-5}$	0.977
4,500	20.98	$-14*10^{-5}$	0.999
8,000	19.88	$-12*10^{-5}$	0.998

Table 4.9 Non-Linear Regression Results – Rigid

AADTT	α_1	α_2	R^2
250	28.02	$-4.8*10^{-5}$	0.969
1,000	23.60	$-17*10^{-5}$	0.951
4,500	21.34	$-9.0*10^{-5}$	0.999
8,000	20.34	$-9.2*10^{-5}$	0.998

To further investigate the result of minor effects of truck volume, a multiple linear regression was conducted on the data presented in Figures 4.25 and 4.26, respectively. For both the flexible and rigid cases, the truck volume was not found to be statistically significant at a significance level of $\alpha = 0.05$. In other words, from a statistical standpoint the truck volume is not significant. The full results of this regression analysis are presented in Appendix E.

Cost Analysis

The reductions in pavement life, due to shifting the axle load spectra, were converted into cost factors following the methodology described in the Life Cycle Cost Analysis section above. For this analysis, a 60-year period was considered so that at least two rehabilitation cycles could be considered for every baseline case. Also, an interest rate of 4% was assumed. Other analyses could be considered altering either, or both, of these two parameters. Figures 4.28 and 4.29 summarize the cost factors for the flexible and rigid pavement designs, respectively.

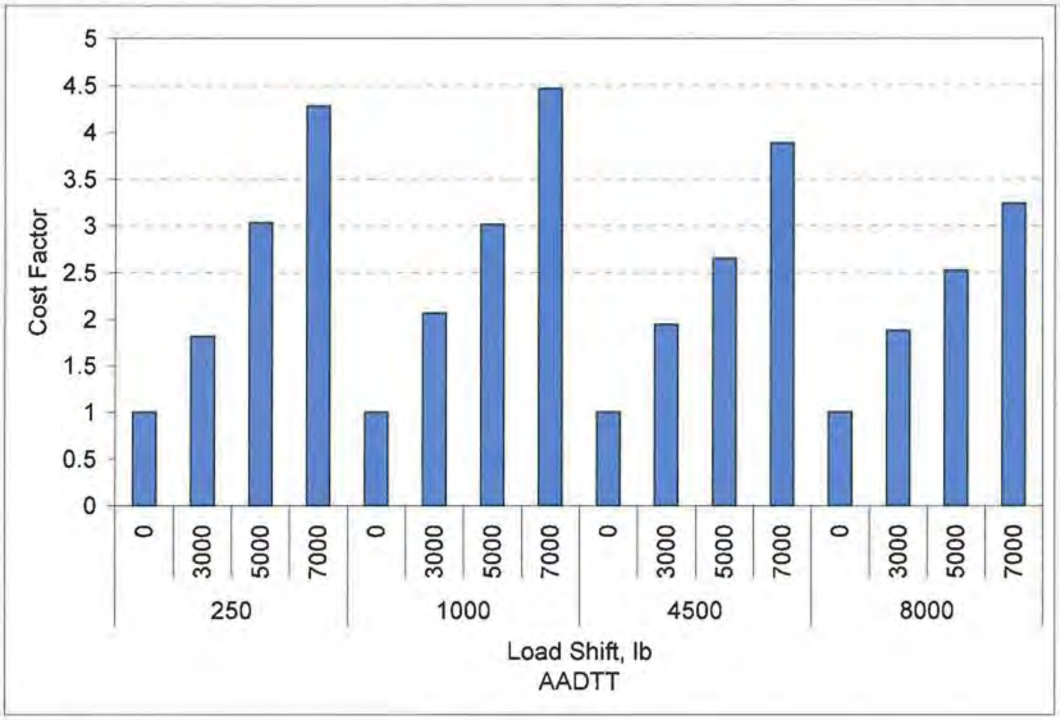


Figure 4.28 Cost Factors – Flexible – Shift Entire Load Spectra.

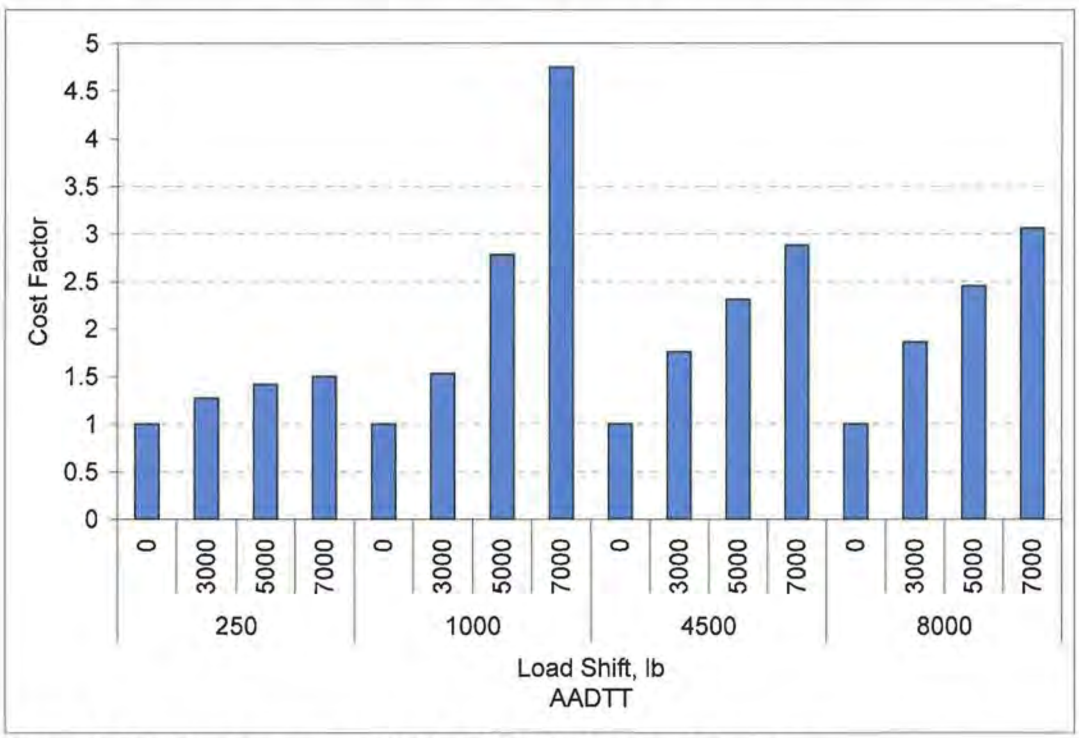


Figure 4.29 Cost Factors – Rigid – Shift Entire Load Spectra.

Both Figures 4.28 and 4.29 indicate rather significant cost factors due to the load shifting. For example, increasing the load spectra by 3,000 lb for the 250 AADTT flexible pavement (first grouping in Figure 4.28) almost doubled the pavement costs. An increase of 5,000 lb tripled the costs and an increase of 7,000 lb resulted in more than four times the cost of the baseline case. Similar increases were generally observed for all the other cases. The exception, as noted previously with respect to reduced pavement life, was the rigid pavement at 250 AADTT (first grouping in Figure 4.29). Since this pavement was overdesigned, increasing the load spectra did not have as dramatic of an effect on either the pavement damage or resulting cost increases.

Each of the data groupings in Figures 4.28 and 4.29 were plotted on an arithmetic scale to determine a best fit exponential function:

$$CF = \beta_1 e^{\beta_2 \cdot Shift} \tag{4.6}$$

Where:

CF = Cost Factor

Shift = load spectra shift, lb

β_1, β_2 = regression coefficients

Figure 4.30 illustrates a representative case using equation 4.6 while Tables 4.10 and 4.11 summarize the curve-fitting parameters and corresponding R^2 values. It can be seen from the tables that each curve had an extremely good fit; the minimum R^2 was 0.957. Also, similar to the observations regarding reduced pavement life above, there did not appear to be a clear trend with respect to the effects of truck volume (AADTT). This is logical since the cost factors were derived directly from the reduced life computations and the same arguments and reasoning presented above hold here as well.

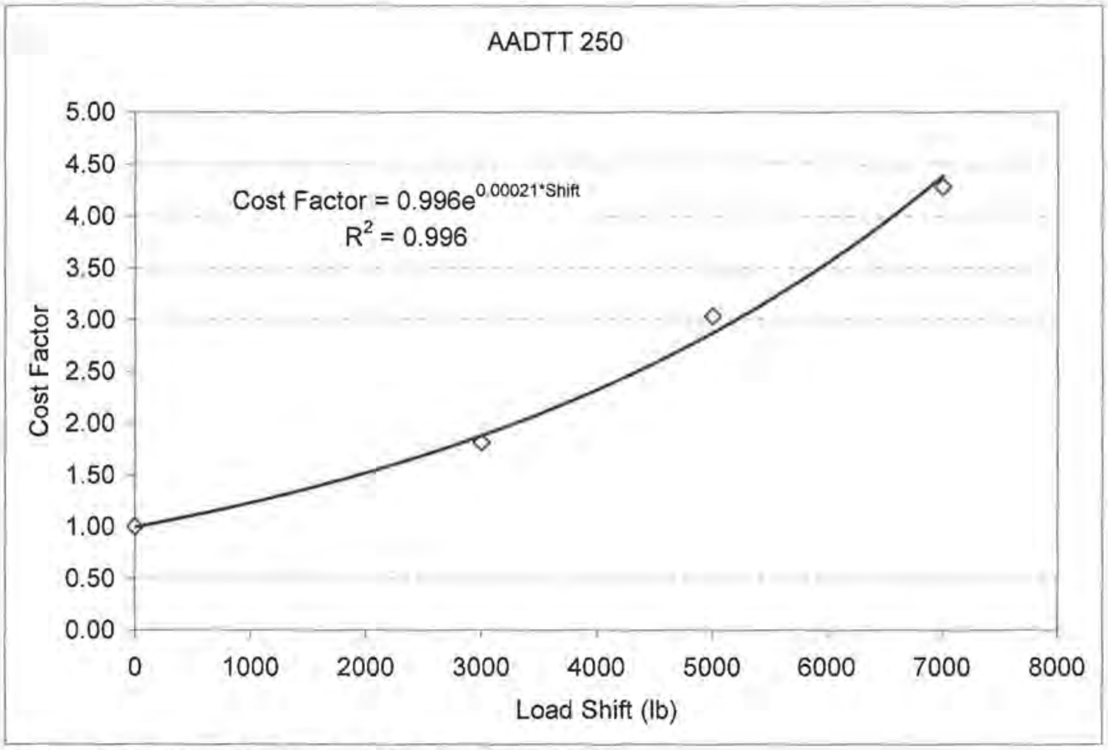


Figure 4.30 Cost Factors – HMA – 250 AADTT.

Table 4.10 Cost Factor Regression Results – Flexible

AADTT	β_1	β_2	R^2
250	0.996	$21*10^{-5}$	0.996
1,000	1.033	$21*10^{-5}$	0.996
4,500	1.030	$19*10^{-5}$	0.995
8,000	1.053	$17*10^{-5}$	0.984

Table 4.11 Cost Factor Regression Results – Rigid

AADTT	β_1	β_2	R^2
250	1.030	$6*10^{-5}$	0.957
1,000	0.911	$23*10^{-5}$	0.972
4,500	1.047	$15*10^{-5}$	0.984
8,000	1.060	$16*10^{-5}$	0.978

The results presented above certainly indicate that shifting load spectra can have significant impacts on both pavement life and life cycle cost. However, the shifts included in this analysis could be considered "extreme" and not likely to occur without widespread changes in truck size and weight legislation in addition to adaptations made by the trucking industry. The results of more commonly occurring scenarios, where specific axles are permitted, are presented below.

Permitting Specific Axles

The following subsections detail the damage and life cycle cost analysis results, respectively, from the analyses of the impact of permitting specific overloaded axles. This portion included 128 pavement simulations as shown by the test matrix in Table 4.12.

Table 4.12. Permitting Specific Axles – Test Matrix

Parameter	# of Permutations	Permutations
Pavement Type	2	HMA (flexible) PCC (rigid)
Loading Scenario	2	Constant Volume – Increased Weight (CV-IW) Decreased Volume – Constant Weight (DV-CW)
AADTT	4	250 1,000 4,500 8,000
Permitted Weight	2	Case D (Average) Case E (Maximum)
% Permitted	4	0% (Baseline) 1% 10% 20%
Total	2x2x4x2x4 = 128	

Damage Analysis

Figures 4.31 to 4.34 summarize the change in pavement life according to the parameters in Table 4.12. As expected, each graph shows a decrease in pavement life as the proportion of permitted axles increases from 0 – 20%. The effect of permitted weight (Case D vs. E) also was as expected; the heavier weight (Case E) resulted in shorter pavement lives compared to Case D.

The effect of truck volume on reduced pavement life was similar to that observed for the shifted load spectra cases presented above. Namely, there did not appear to be a strong correlation between reduced life and volume. A good example of this is presented in Figure 4.33 where the differences between 4,500 AADTT and 8,000 AADTT are quite small.

An interesting case was seen in Figure 4.31 (4500 AADTT with Case E). A sharp decrease in pavement life was observed between the baseline and 1% case which was not observed for the other scenarios in Figure 4.31. These simulations were rechecked and found to be correct. Therefore, it evidently shows that small numbers of permitted axles in certain cases can have a dramatic impact on pavement life. This also stresses the need to consider pavements on a case-by-case basis.

The effect of loading scenario (CV-IW vs. DV-CW) on reduced pavement life was expected. For example, a comparison of Figure 4.31 vs. Figure 4.32 shows that carrying the same total tonnage on fewer axles (DV-CW) causes less pavement life reduction than the alternative (CV-IW). A comparison of the two loading scenarios, shown in Figure 4.35, indicates an average life increase of approximately 3.5 years when a reduction in volume is included in the analysis. The graph was generated by plotting loading scenario paired data from Figures 4.31 – 4.34 (by pairing a CV-IW value with its corresponding DV-IW value). The baseline cases were excluded since these were equal amongst the pairs.

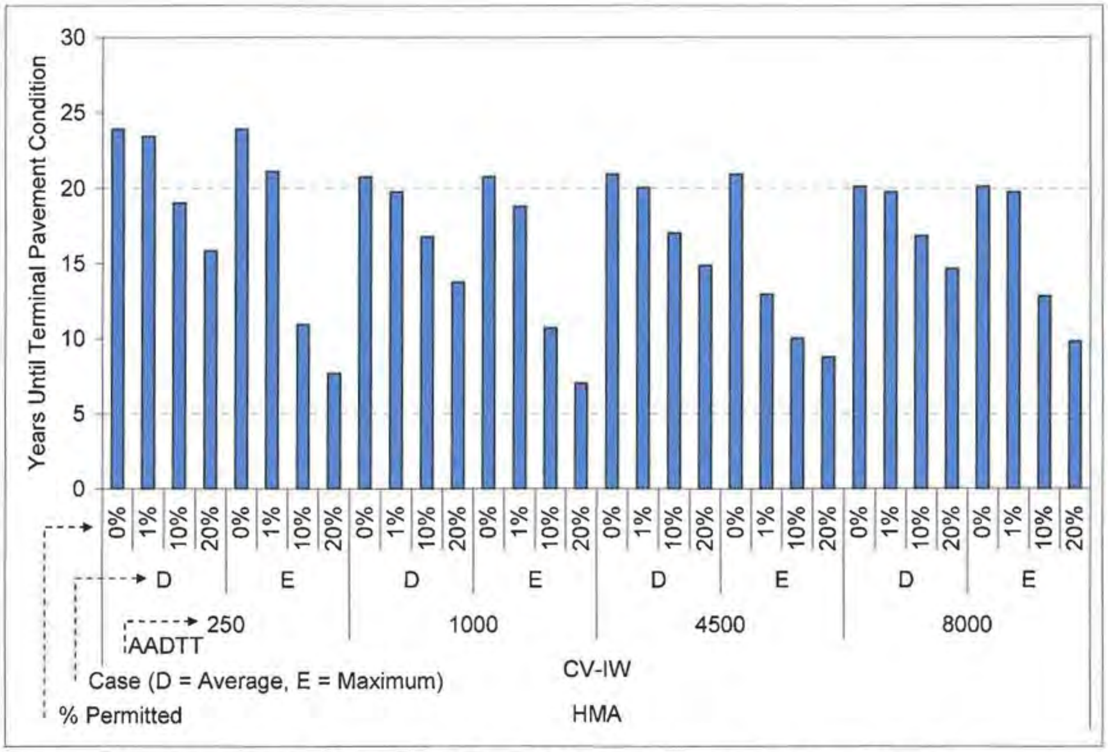


Figure 4.31 Pavement Life – Flexible – CV-IW – Permitting Specific Axles.

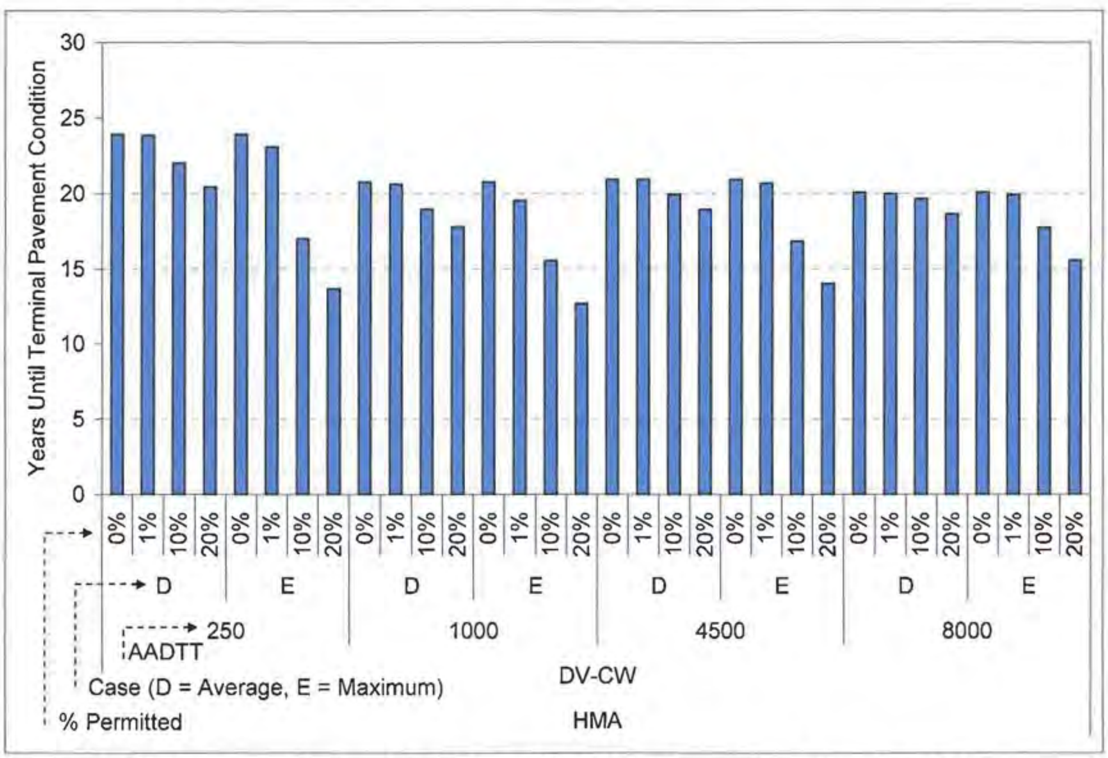


Figure 4.32 Pavement Life – Flexible – DV-CW – Permitting Specific Axles.

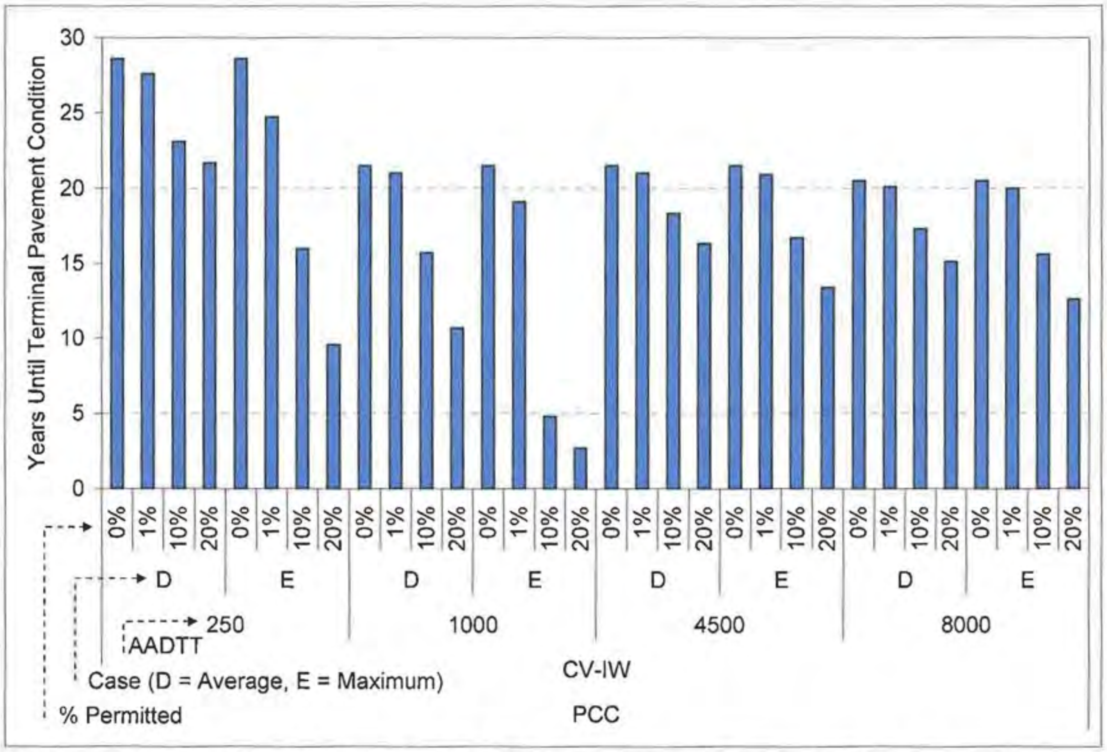


Figure 4.33 Pavement Life – Rigid – CV-IW – Permitting Specific Axles.

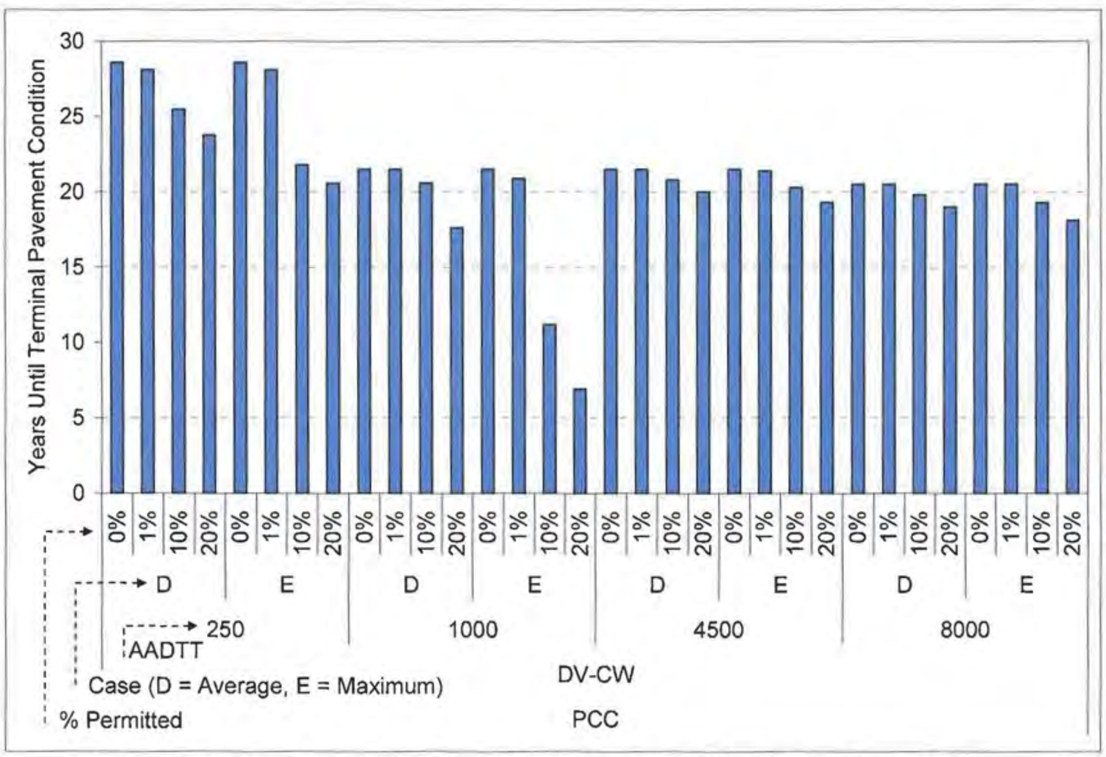


Figure 4.34 Pavement Life – Rigid – DV-CW – Permitting Specific Axles.

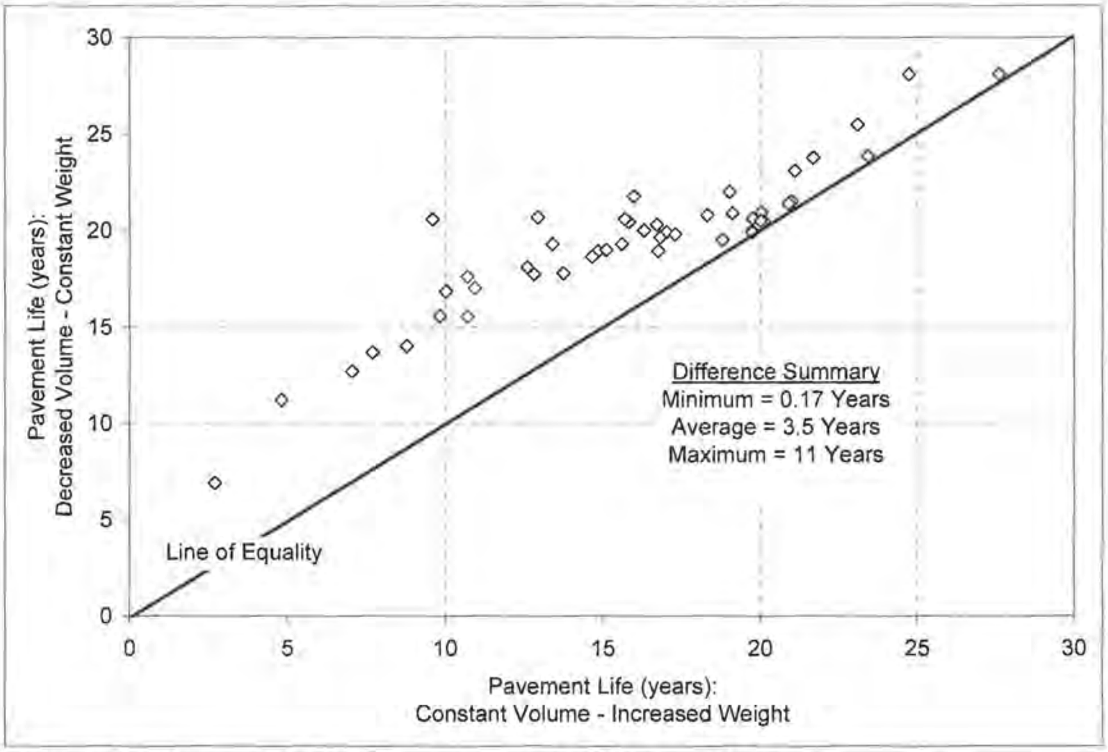


Figure 4.35 Pavement Life Comparison: CV-IW vs. DV-CW.

The relationship between pavement life and percent of permitted axles was found to have a negative exponential relationship, similar in form to that observed between pavement life and load spectra shift. Figure 4.36 provides an example of this relationship for the flexible pavement, CV-IW, 250 AADTT, average (Case D) permitted axle scenario and had the form:

$$Life = \lambda_1 e^{\lambda_2 \cdot \% Permitted} \tag{4.7}$$

Where:

Life = years until terminal pavement condition

% Permitted = percent of axles that are at the permitted weight

λ_1, λ_2 = regression coefficients

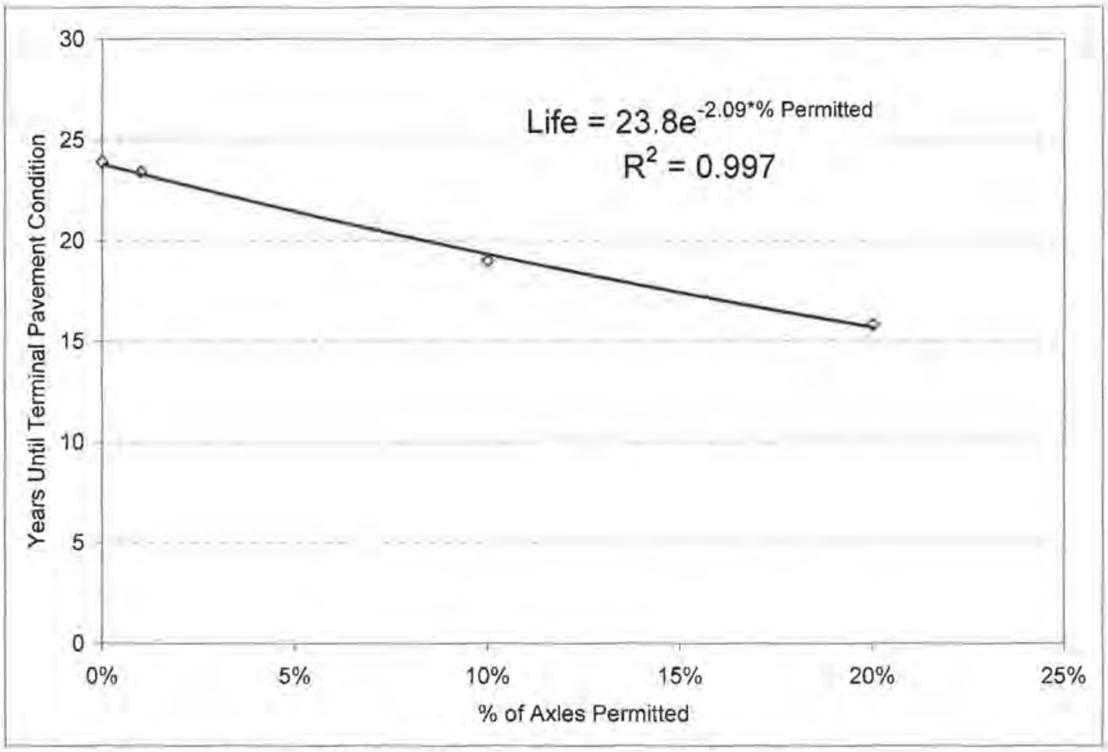


Figure 4.36 Flexible – CV-IW – 250 AADT – Case D Scenario.

Regression analyses were performed for all the cases investigated and the results are summarized in Tables 4.13 and 4.14 for the flexible and rigid pavements, respectively. The high R^2 in almost all the cases indicates a strong relationship between pavement life and the percentage of permitted axles. The one exception was observed in the flexible scenarios (Case E, CV-IW, 4500 AADTT). As noted above, there was a sharp decrease in life between baseline and 1% permitted axles, resulting in a relatively poor fit of the exponential model. This fact carried through to the cost factor analysis as well.

The λ_2 columns in the two tables can be interpreted as the impact increasing the percentage of permitted axles has on the pavement life. Since these values were not strongly influenced by the truck traffic volume, they were averaged by case and scenario to bring more meaning to the data. It is interesting to note that λ_2 is consistently two to four times greater for the CV-IW cases than the comparable DV-CW cases. This is consistent with the observations made regarding Figure 4.35. Similar ratios were observed when comparing the averages between the Case D and E scenarios. It must be finally noted that these relationships are specific to the cases investigated in this study and though they may represent general trends, would not necessarily hold true for all cases. For example, if the Case E loadings were much greater, the resulting pavement life (in terms of λ_2) would be much shorter.

Table 4.13 Flexible Pavement Designs – Life Regression Summary

Case	Scenario	AADTT	λ_1	λ_2	R^2
D	CV-IW	250	23.8	-2.09	0.997
		1000	20.5	-1.99	0.996
		4500	20.5	-1.67	0.987
		8000	20.0	-1.59	0.996
			Average	-1.83	
	DV-CW	250	24.0	-0.80	0.998
		1000	20.7	-0.79	0.991
		4500	21.0	-0.51	0.997
		8000	20.1	-0.35	0.948
			Average	-0.61	
E	CV-IW	250	22.3	-5.69	0.965
		1000	19.9	-5.40	0.987
		4500	16.3	-3.48	0.715
		8000	19.9	-3.70	0.981
			Average	-4.57	
	DV-CW	250	23.6	-2.83	0.988
		1000	20.3	-2.40	0.990
		4500	20.9	-2.04	0.997
		8000	20.1	-1.28	1.000
			Average	-2.14	

Table 4.14 Rigid Pavement Designs – Life Regression Summary

Case	Scenario	AADTT	λ_1	λ_2	R^2
D	CV-IW	250	27.9	-1.39	0.930
		1000	21.7	-3.49	0.998
		4500	21.3	-1.38	0.993
		8000	20.4	-1.53	0.997
			Average	-1.95	
	DV-CW	250	28.4	-0.92	0.984
		1000	21.8	-0.98	0.918
		4500	21.5	-0.37	0.997
		8000	20.5	-0.39	0.997
			Average	-0.66	
E	CV-IW	250	27.3	-5.26	0.994
		1000	19.6	-10.73	0.951
		4500	21.4	-2.37	0.999
		8000	20.4	-2.46	0.996
			Average	-5.21	
	DV-CW	250	28.0	-1.73	0.903
		1000	21.4	-5.83	0.993
		4500	21.5	-0.54	0.999
		8000	20.6	-0.64	0.998
			Average	-2.19	

Cost Analysis

Following the procedures described above, a life cycle cost analysis was conducted for the permitted axle scenarios. Again, a 60-year time frame with an interest rate of 4% were used. Figures 4.37 through 4.40 summarize the data for all the cases. In general, the cost factors were lower than those obtained from shifting entire load spectra. Also, as expected, the cost factors were lower for the DV-CW cases compared to the CV-IW cases. Figure 4.41 clearly indicates the trend of lower cost factors for the DV-CW cases. As explained above, the reduction in costs is derived from carrying the same total weight more efficiently on fewer total axle passes.

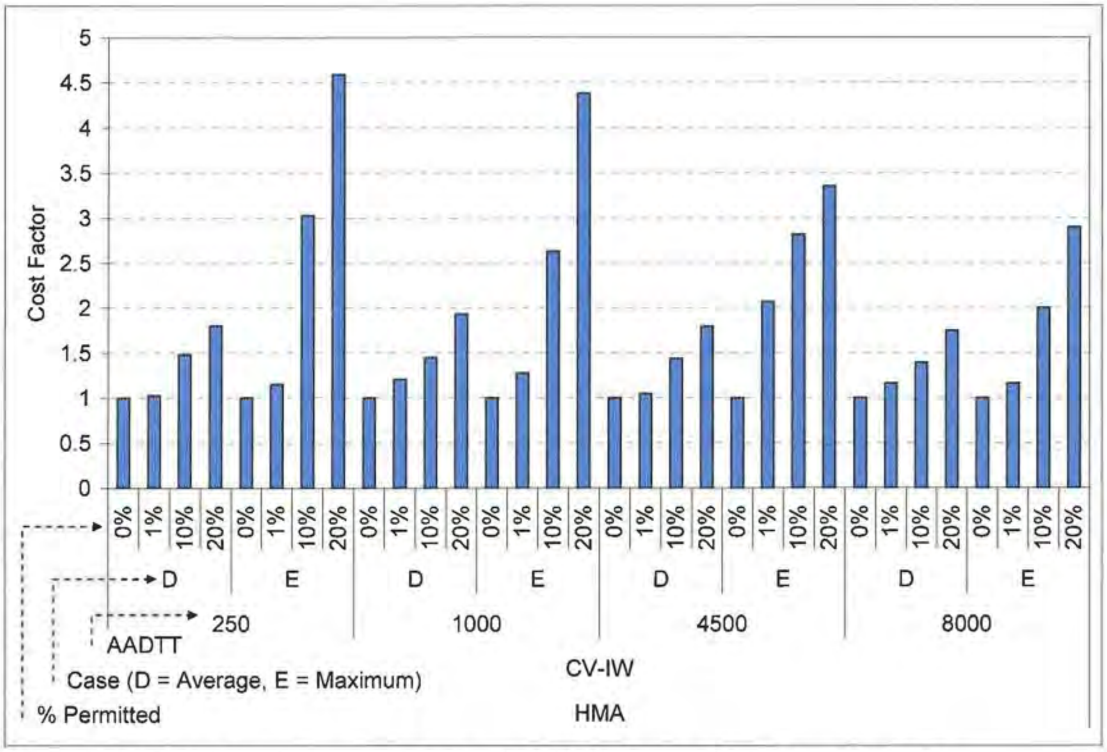


Figure 4.37 Cost Factors – Flexible – CV-IW – Permitting Specific Axles.

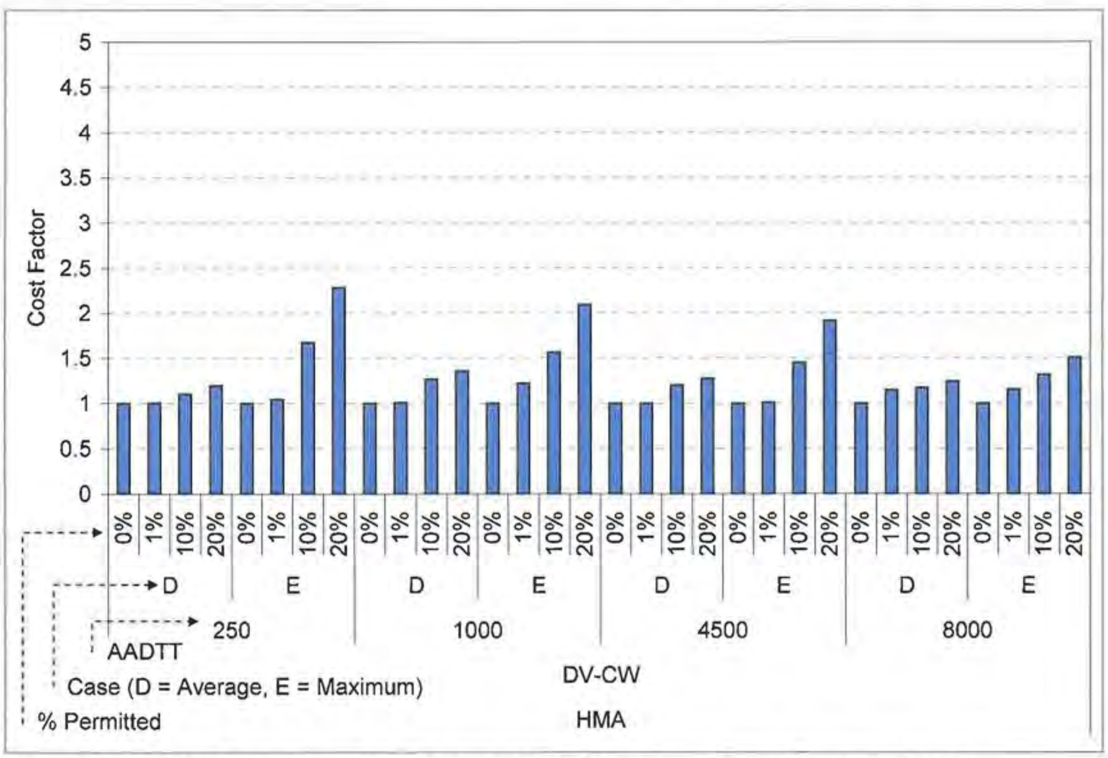


Figure 4.38 Cost Factors – Flexible – DV-CW – Permitting Specific Axles.

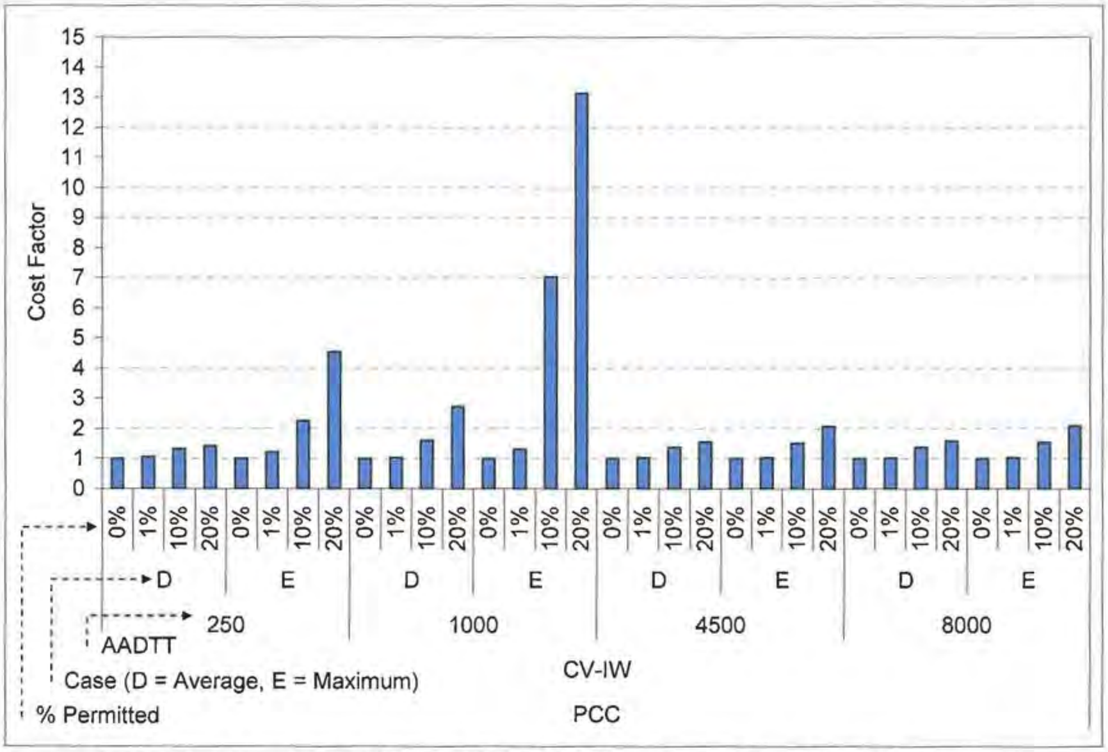


Figure 4.39 Cost Factors – Rigid – CV-IW – Permitting Specific Axles.

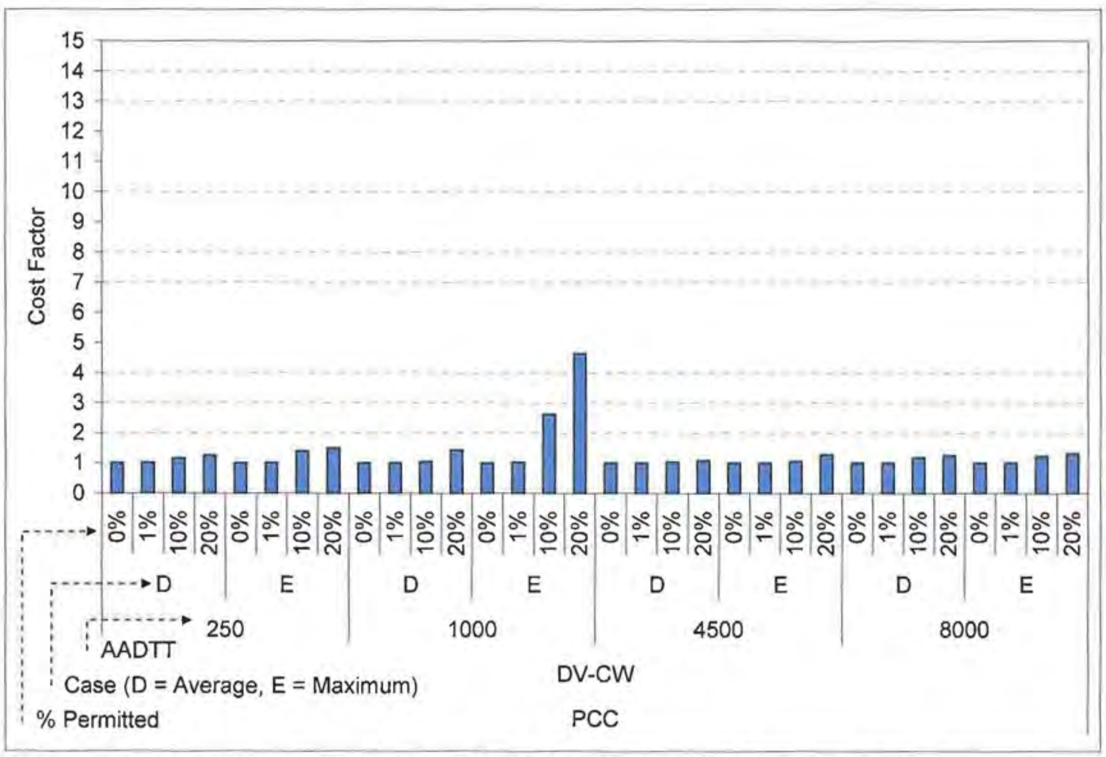


Figure 4.40 Cost Factors – Rigid – DV-CW – Permitting Specific Axles.

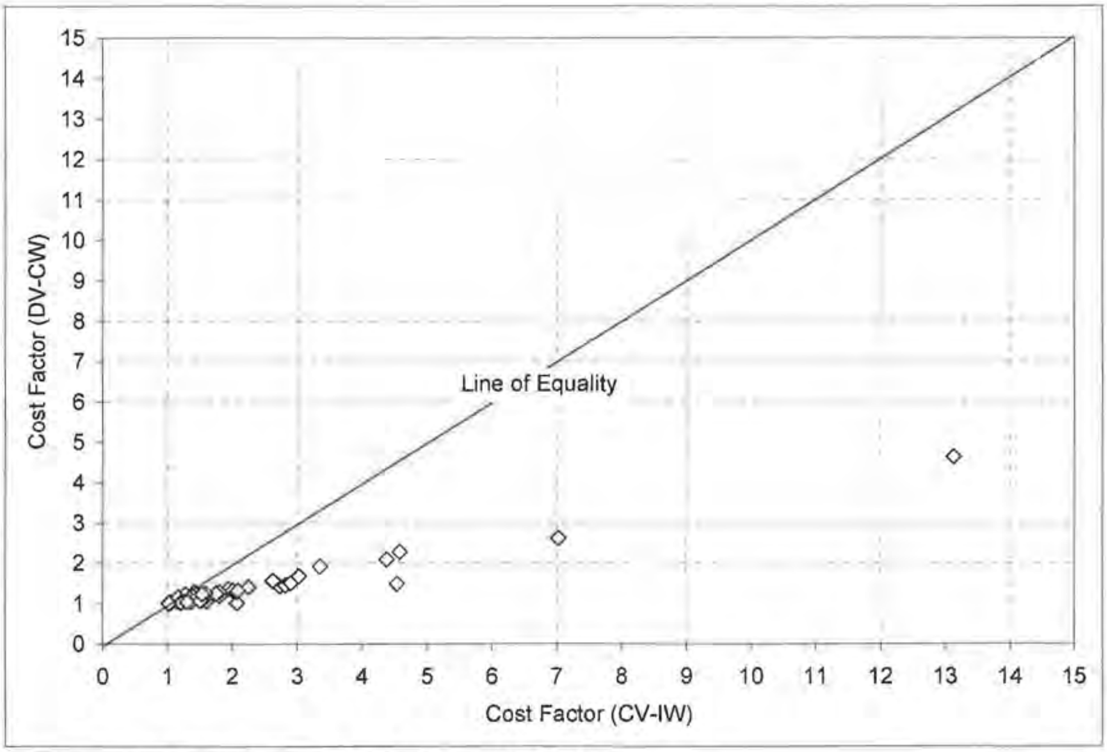


Figure 4.41 Cost Factor: CV-IW vs. DV-CW.

Similar to the previous analyses, cost factor was found to have an exponential relationship with percentage of permitted axles. Figure 4.42 provides an example of this relationship and has the following the form:

$$CF = \phi_1 e^{\phi_2 \cdot \% Permitted} \tag{4.8}$$

Where:

CF = Cost Factor

% Permitted = percent of axles that are at the permitted weight

ϕ_1, ϕ_2 = regression coefficients

Tables 4.15 and 4.16 summarize the cost factor regression results for all the cases considered. It is again interesting to note the trends within the average ϕ_2 results. As expected, the costs tended to increase more rapidly (increased ϕ_2) for the CV-IW cases compared to the DV-CW cases. Furthermore, the heavier loaded axle cases (E) were greater than their counterparts (D).

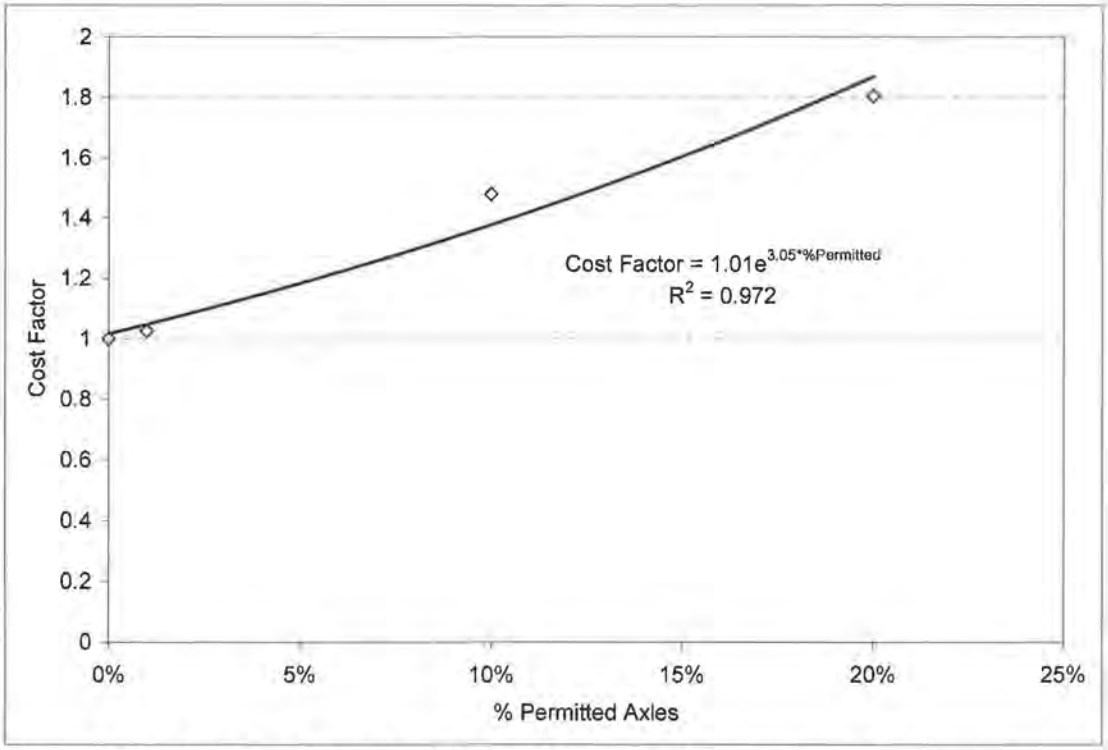


Figure 4.42 Flexible – CV-IW – 250 AADT – Case D Scenario – Cost Factors.

Table 4.15 Flexible Pavement Designs – Cost Factor Regression Summary

Case	Scenario	AADTT	ϕ_1	ϕ_2	R^2
D	CV-IW	250	1.02	3.045	0.972
		1000	1.08	2.935	0.947
		4500	1.02	2.957	0.986
		8000	1.07	2.519	0.951
			Average	2.864	
	DV-CW	250	1.00	0.906	0.996
		1000	1.01	1.626	0.929
		4500	1.01	1.312	0.933
		8000	1.07	0.808	0.660
			Average	1.163	
E	CV-IW	250	1.10	7.743	0.951
		1000	1.13	7.139	0.971
		4500	1.46	4.693	0.671
		8000	1.08	5.200	0.974
			Average	6.194	
	DV-CW	250	1.02	4.216	0.984
		1000	1.09	3.335	0.951
		4500	1.00	3.339	0.992
		8000	1.07	1.795	0.903
			Average	3.171	

Table 4.16 Rigid Pavement Designs – Cost Factor Regression Summary

Case	Scenario	AADTT	ϕ_1	ϕ_2	R^2
D	CV-IW	250	1.03	1.729	0.920
		1000	0.98	5.068	0.999
		4500	1.02	2.258	0.954
		8000	1.02	2.396	0.959
			Average	2.863	
	DV-CW	250	1.01	1.186	0.980
		1000	0.97	1.714	0.864
		4500	1.00	0.392	0.998
		8000	1.01	1.179	0.921
			Average	1.118	
E	CV-IW	250	1.06	7.315	0.995
		1000	1.20	13.093	0.938
		4500	1.01	3.678	0.994
		8000	1.01	3.747	0.991
			Average	6.958	
	DV-CW	250	1.03	2.111	0.893
		1000	1.02	7.949	0.980
		4500	0.98	1.227	0.931
		8000	1.01	1.463	0.942
			Average	3.187	

Changing Axle Configuration

Damage Analysis

Simulations were performed using each of the four traffic levels (250, 1000, 4500 and 8000 AADTT), and for each of the percentages of volume to change tandem axles to tridems (0%, 5%, 25%, 50%, and 100%). All of these simulations displayed no change in design life; the 100% volume change results were equal to the 0% volume change. Therefore, it was decided that further investigation was warranted. Though the MEPDG calculates the critical pavement responses during program execution, the data cannot be viewed in any of the files generated. Therefore, another tool was used for this part of the investigation.

Using a flexible pavement model with equivalent pavement cross-section, material properties and axle configurations, the same loading scenario was simulated in WESLEA, a layered elastic pavement analysis software. The HMA modulus used was a weighted average from the MEPDG output data. The HMA thickness used was from the 1000 AADTT level, which was 10.25 inches. The results were analyzed to find the maximum horizontal tensile strain at the bottom of the HMA layer for each loading case. Detailed results can be found in Appendix F. The locations of these maximum values are shown in Figure 4.43, where half of each axle is shown. For axle dimensions, see Appendix G. The results showed that the difference in the maximum strains varied by less than 3 microstrain ($\mu\varepsilon$); the tandem axle and tridem axle loads created strains of 59 $\mu\varepsilon$ and 57 $\mu\varepsilon$, respectively.

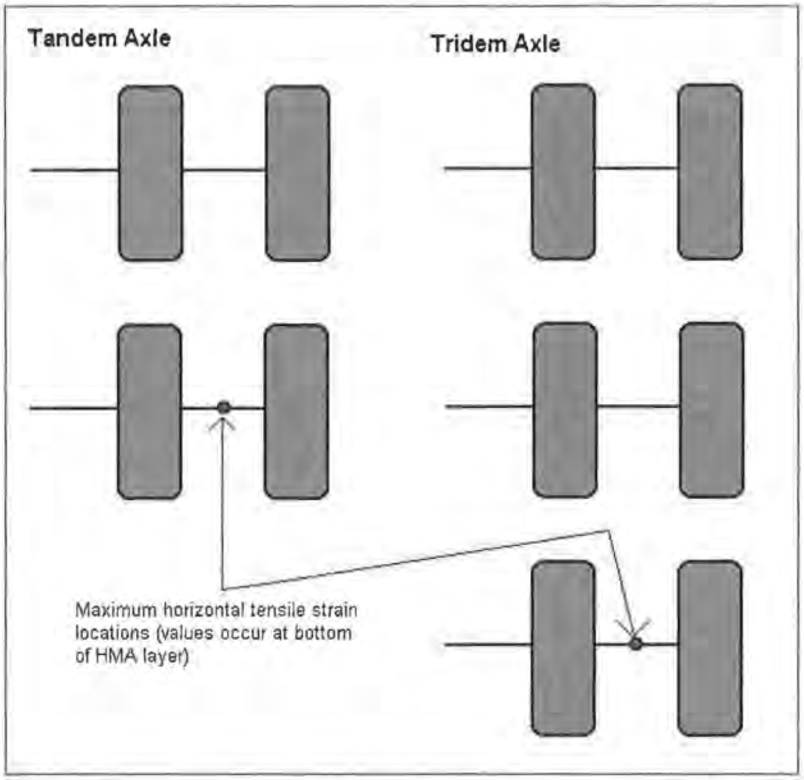


Figure 4.43 Location of Maximum Horizontal Tensile Strains.

Note that the tandem axle created a slightly higher tensile strain than the tridem axle. Away from the center of each axle, the pavement experiences a region of compressive (negative) strain. These compressive strains overlap with the tensile strain in close vicinity to the adjacent axle and act to lower the net tensile strain of the adjacent axle. Figures 4.44 and 4.45 illustrate this concept. In each figure, the critical (maximum) strain location is positioned under the first axle. As can be seen when comparing the two, the tandem strain is slightly higher than the tridem. Also, Figure 4.45 shows that the strain beneath the center axle in the tridem configuration has a slightly lower strain. This also results from the strain overlap from adjacent axles.

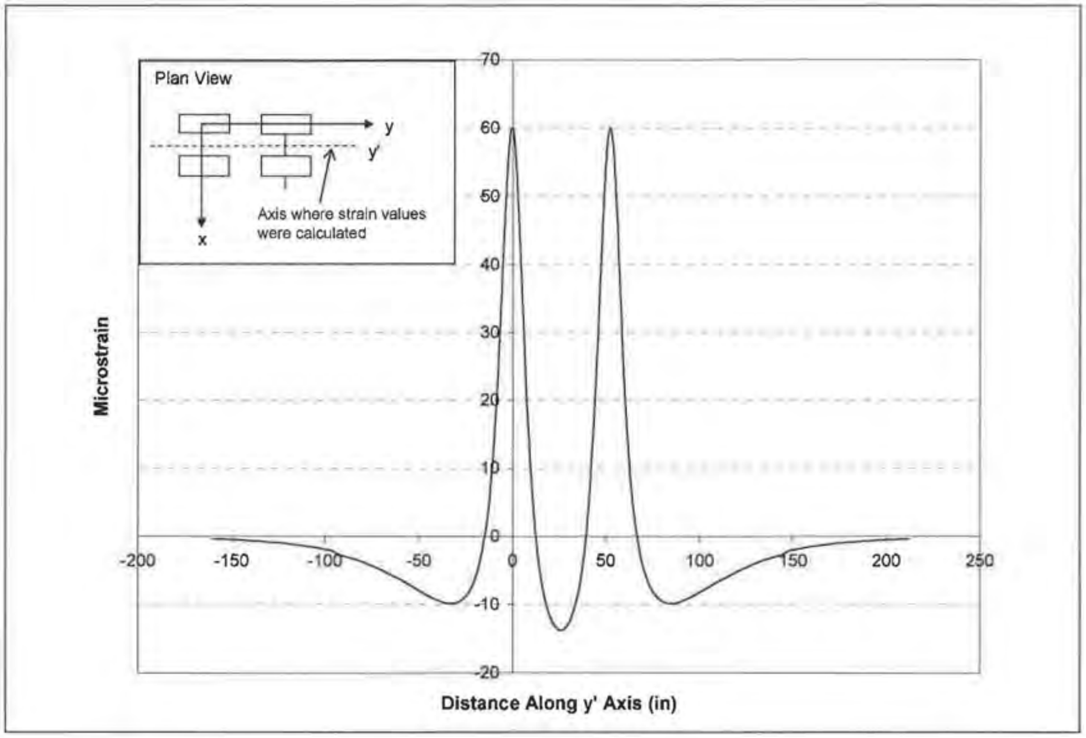


Figure 4.44 Tandem Axle Strain Values and Locations.

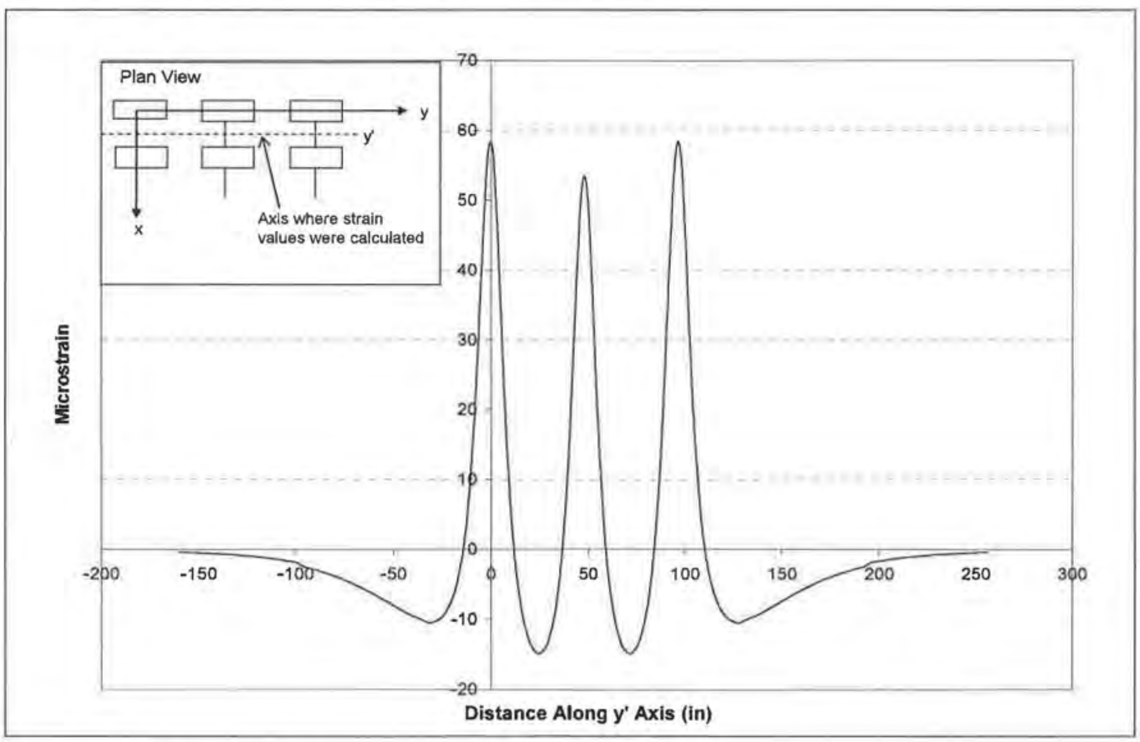


Figure 4.45 Tridem Axle Strain Values and Locations.

The maximum tensile strain values were then used in a transfer function for fatigue cracking in order to find the number of load repetitions until pavement failure, which is a more tangible quantity. This transfer function, along with the input values used, can be found in Appendix H. From this function, the number of load applications until failure calculated was 0.95 billion for the tandem axle, and 1.1 billion for the tridem axle, which is a relatively small difference in practical terms. Further, since there was no change in pavement life due to altering the axle configuration, a cost analysis was not performed.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

This research investigated the impact of heavy vehicles on highway pavement damage and corresponding costs. In this study, a literature review and survey of states' practice demonstrated a wide range of approaches to quantifying these parameters. From these findings, a framework was developed that utilized the MEPDG and conventional life cycle cost analysis to quantify the effects of alternative loading scenarios on pavement damage and cost. A number of conclusions and recommendations can be drawn from the data assembled within this report. They have been subdivided into their respective sources in the following three subsections.

LITERATURE REVIEW

1. Weight regulations are highly state specific and many states have legal limits exceeding federal limits for GVW, single and tandem axles, respectively.
2. Most states issue so-called "Routine Permits" that require no special investigation, but only payment of a permitting fee to operate at a heavier weight.
3. GVW and axle weights are typically considered separately in permitting.
4. Permitting fee structures have not historically been damage-based and typically only recover administrative costs.
5. Current estimates of overloads, from government agencies, range from 15-30% of all trucks. This estimate may be low since a survey of trucking companies in New York found a 50% overload percentage. Since there is some disparity between government

and industry estimates, more well-defined information regarding the occurrence of overloads is warranted.

6. Pavement damage assessment due to overloads has traditionally been based upon an empirical, equivalent single axle load (ESAL), approach or a mechanistic-empirical (M-E) approach. Both approaches have been used with reasonable success. Given that pavement engineering is progressing toward mechanistic-empirical pavement design, using an M-E approach is considered state-of-the-art.

SURVEY OF STATES' PRACTICE

1. The most common type of permitting fee structures currently used in the U.S. is based on GVW and mileage. The second most common type is based solely on mileage.
2. 44% of responding states stated that permitting fee structures were set by legislative act.
3. There does not appear to be any widespread efforts to coordinate permitting activities in multiple states.
4. Most states (74%), as part of the permit application process, conduct analyses of the impact of the subject vehicle on pavements, bridge, or both.
5. Six states reported using or experimenting with mechanistic-empirical approaches, and most states report typically involving their bridge design units, and many the pavement design units.
6. There is an incredible diversity among states with respect to factors affecting weight limits associated with climate and commodity type. For example, changes in legal weight limits for the spring thaw period are very common in the northern states. The

variety of commodities specifically addressed in regulations (typically statutes) may reflect the importance of various industries to their respective state's economies.

PAVEMENT DAMAGE AND COST FRAMEWORK

1. The MEPDG can be used to study effects of overloads on pavement damage, but it requires the analyst to have detailed knowledge of the MEPDG-generated temporary files and develop a separate spreadsheet template to modify the load spectra files.
2. Cost analysis must be handled separately from the MEPDG.
3. Cost factors are an effective means of comparing baseline to alternative loading scenarios. These factors can be used as multipliers to scale costs to different sized projects.
4. Small shifts in entire load spectra, such as 3,000 lb/axle, can have significant impacts on damage and cost.
5. Accurate load characterization is critical to accurate life and cost predictions.
6. With respect to reduced pavement life and cost increases, the overall truck volume is less important than the load magnitudes and numbers of overloaded axles.
7. For the cases considered in this investigation, reduced pavement life was well characterized by a negative exponential relationship with the amount of load shift.
8. For the cases considered in this investigation, reduced pavement life was well characterized by a negative exponential relationship with the percentage of permitted axles operating at a specific weight.

9. In this investigation, changing rear 34,000 lb tandem axles to 51,000 lb tridem axles had no effect on pavement performance when the total weight of the traffic stream was held constant.
10. In this study, cost factor was well characterized by a positive exponential relationship with the amount of load shift.
11. In this investigation, cost factor was well characterized by a positive exponential relationship with the percentage of permitted axles operating at a specific weight.
12. Further investigation is warranted to identify and characterize other loading scenario conditions to be simulated with the MEPDG.
13. Changing the analysis period and interest rate can be further evaluated for its impact on cost factors.
14. Additional study is needed to explore methodologies to more accurately predict changes in load spectra resulting from changes in legal limits, or from permitting heavier axles
15. Though trends were observed in the data, it is important to evaluate pavements on a case-by-case basis for reduction in pavement life and associated cost factors.

REFERENCES

1. "Truck Weight Limits," Special Report 225, Transportation Research Board, 1990.
2. U.S. Department of Transportation, "Comprehensive Truck Size and Weight Study Volume I – Summary Report," FHWA-PL-00-029 (Volume I), 2000.
3. Abdel Halim, A.O. and F.F. Saccomanno, "Axle Load Limits in Ontario: Long Term Analysis," Transportation Research Record No. 1038, 1985, pp. 26-33.
4. "State Laws and Regulations on Truck Size and Weight," NCHRP Report 198, National Cooperative Highway Research Program, 1979.
5. USDOT, "Comprehensive Truck Size and Weight Study Volume II – Issues and Background," FHWA-PL-00-029 (Volume II), 2000.
6. Hajek, J.J. and J.R. Billing, "Changes in Trucking that Affect Pavements," Transportation Research Record No. 1816, 2002, pp. 96-103.
7. Parkinson, S., J. Finnie, D. Horn and R. Lottman, "Procedure to Calculate the Economic Benefit of Increased Pavement Life Resulting from Port of Entry Operations in Idaho," Transportation Research Record No. 1359, 1992, pp. 49-56.
8. Barros, R.T., "Analysis of Pavement Damage Attributable to Overweight Trucks in New Jersey," Transportation Research Record No. 1038, 1985, pp. 1-9.
9. Jackson, J. and M. Murphy, "Implementation of Falling Weight Deflectometer Load-Zoning Procedure in Texas," Transportation Research Record No. 1377, 1992, pp. 143-149.
10. Luskin, D.M. and C.M. Walton, "Effects of Truck Size and Weights on Highway Infrastructure and Operations," Project Summary Report 2122-S, Center for Transportation Research, University of Texas at Austin, March 2001.

11. Straus, S.H. and J. Semmens, "Estimating the Cost of Overweight Vehicle Travel on Arizona Highways," Transportation Research Board Preprint, 2006.

12. Chen, D-H, J. Bilyeu, and J-R. Chang, "A Review of the Superheavy Load Permitting Programme in Texas," International Journal of Pavement Engineering, Vol. 6, No. 1, March 2005, pp.47-55.

13. Jooste, F.J. and E.G. Fernando, "Investigation of the Effects of Superheavy Loads on Pavements," 4th Bearing Capacity of Roadways and Airfields, Minneapolis, MN, 1994, pp. 1566-1574.

14. Roberts, F.L. and L. Djakfar, "Cost of Pavement Damage Due to Heavier Loads on Louisiana Highways – Preliminary Assessment," Transportation Research Record No. 1732, 2000, pp. 3-11.

15. Terrell, R.L. and C.A. Bell, "Effects of Permit and Illegal Overloads on Pavements," NCHRP 131, National Cooperative Highway Research Program, 1987.

16. Rufolo, A.M., L. Bronfman and E. Kuhner, "Effect of Oregon's Axle-Weight-Distance Tax Incentive," Transportation Research Record No. 1732, 2000, pp. 63-69.

17. Wyatt, J.J. and M.U. Hassan, "Some Tentative Findings About the Effect of Level Enforcement on Compliance with Truck Weight Regulations," Transportation Forum, Vol. 2 – 3, 1985, pp. 48 – 52.

18. Meyburg, A.H., J-D.M Saphores and R.E. Schuler, "Collecting Usage Data for Analyzing a Heavy-Vehicle, Divisible-Load Permit System," Transportation Research Record No. 1522, 1996, pp. 9-17.

19. General Accounting Office. 1979. Excessive Truck Weight: An Expensive Burden We Can No Longer Support. Washington, D.C.: General Accounting Office.

20. United States Federal Highway Administration. 1995. Comprehensive truck size and weight study: Summary Report for Phase I--Synthesis of Truck Size and Weight (TS&W) Studies and Issues. Available from: http://ntl.bts.gov/DOCS/cts.html.

21. Highway Research Board. *The AASHO Road Test, Report 5, Pavement Research.* Special Report 61E. National Academy of Sciences – National Research Council, Washington, DC, 1962.

22. Chen, D-H, E.R. Cortez, F. Zhou, W-S Yang and K. Petros, "Analysis of Overload Damages in Thin Flexible Pavements," Transportation Research Board preprint, 2006.

23. American Association of State Highway and Transportation Officials. *AASHTO Guide for Design of Pavement Structures*. Washington, D.C., 1993.

24. Carmichael, R.F.III, F.L. Roberts and H.J. Treybig, "Procedure for Evaluating the Effects of Legal Load Limits on Pavement Costs," Transportation Research Record No. 725, 1979, pp. 1-8.

25. Eres Consultants Division. *Guide For Mechanistic-Empirical Pavement Design of New and Rehabilitated Pavement Structures*. Final Report, NCHRP 1-37A, 2004.

26. Kilareski, W.P., "Heavy Vehicle Evaluation for Overload Permits," Transportation Research Record No. 1227, 1989, pp. 194-204.

27. Fernando, E., W. Liu and J. Leidy, "Analysis Procedure for Load-Zoning Pavements," Transportation Research Record No. 1860, 2003, pp. 117-125.

28. USDOT, "Comprehensive Truck Size and Weight Study Volume III – Scenario Analysis," FHWA-PL-00-029 (Volume III), 2000.

29. Merriss, J., "New Research on Pavement Design Factors," Policy Notes, Oregon Department of Transportation, June 2003, http://www.oregon.gov/ODOT/CS/EA/policy_notes/03_policy_notes/0603_New_Research_on_Pavement_Damage.pdf, accessed September 21, 2006.

30. Walls, J. and M.R. Smith, "Life Cycle Cost Analysis in Pavement Design – Interim Technical Bulletin," FHWA Report No. FHWA-SA-98-079, Federal Highway Administration, Washington, D.C., 1998.

31. Huang, Y.H., Pavement Analysis and Design, Second Edition, Prentice Hall, New Jersey, 2004.

APPENDIX A – CRUSHED STONE AND SUBGRADE SOIL PROPERTIES

Layer 2 -- Crushed stone

Unbound Material:	Crushed stone
Thickness(in):	15

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	30000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	1
Liquid Limit (LL)	6
Compacted Layer	No
Passing #200 sieve (%):	8.7
Passing #40	20
Passing #4 sieve (%):	44.7
D10(mm)	0.1035
D20(mm)	0.425
D30(mm)	1.306
D60(mm)	10.82
D90(mm)	46.19

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	8.7
#100	
#80	12.9
#60	
#50	
#40	20
#30	
#20	
#16	
#10	33.8
#8	
#4	44.7
3/8"	57.2
1/2"	63.1
3/4"	72.7
1"	78.8
1 1/2"	85.8
2"	91.6
2 1/2"	
3"	
3 1/2"	97.6
4"	97.6

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	127.2 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	0.05054 (derived)
Optimum gravimetric water content (%):	7.4 (derived)
Calculated degree of saturation (%):	61.2 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	7.2555
b	1.3328
c	0.82422
Hr.	117.4

Layer 3 -- A-6

Unbound Material:	A-6
Thickness(in):	Semi-infinite

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	14500

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	16
Liquid Limit (LL)	33
Compacted Layer	No
Passing #200 sieve (%):	63.2
Passing #40	82.4
Passing #4 sieve (%):	93.5
D10(mm)	0.000285
D20(mm)	0.0008125
D30(mm)	0.002316
D60(mm)	0.05364
D90(mm)	1.922

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	63.2
#100	
#80	73.5
#60	
#50	
#40	82.4
#30	
#20	
#16	
#10	90.2
#8	
#4	93.5
3/8"	96.4
1/2"	97.4
3/4"	98.4
1"	99
1 1/2"	99.5
2"	99.8
2 1/2"	
3"	
3 1/2"	100
4"	100

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	107.9 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	1.95e-005 (derived)
Optimum gravimetric water content (%):	17.1 (derived)
Calculated degree of saturation (%):	82.1 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	108.41
b	0.68007
c	0.21612
Hr.	500

APPENDIX B – HOT MIX ASPHALT PARAMETERS

Layer 1 -- Asphalt concrete

Material type:	Asphalt concrete
Layer thickness (in):	Variable

General Properties

General

Reference temperature (F°):	70

Volumetric Properties as Built

Effective binder content (%):	11
Air voids (%):	7.5
Total unit weight (pcf):	148
Poisson's ratio:	0.35 (user entered)

Thermal Properties

Thermal conductivity asphalt (BTU/hr-ft-F°):	0.67
Heat capacity asphalt (BTU/lb-F°):	0.23

Asphalt Mix

Cumulative % Retained 3/4 inch sieve:	5
Cumulative % Retained 3/8 inch sieve:	37
Cumulative % Retained #4 sieve:	53
% Passing #200 sieve:	5

Asphalt Binder

Option:	Superpave binder grading
A	9.7150 (correlated)
VTS:	-3.2080 (correlated)

High temp. °C	Low temperature, °C						
	-10	-16	-22	-28	-34	-40	-46
46							
52							
58							
64							
70							
76							
82							

APPENDIX C – CONCRETE PROPERTIES

Structure--Design Features

Permanent curl/warp effective temperature difference (°F):	-10

Joint Design

Joint spacing (ft):	15
Sealant type:	Liquid
Dowel diameter (in):	1
Dowel bar spacing (in):	12

Edge Support

Edge Support	Tied PCC shoulder
Long-term LTE(%):	40
Widened Slab (ft):	n/a

Base Properties

Base type:	Granular
Erodibility index:	Erosion Resistant (3)
PCC-Base Interface	Full friction contact
Loss of full friction (age in months):	245

Structure--ICM Properties

Surface shortwave absorptivity:	0.85

Structure--Layers

Layer 1 -- JPCP

General Properties

PCC material	JPCP
Layer thickness (in):	Variable
Unit weight (pcf):	150
Poisson's ratio	0.2

Thermal Properties

Coefficient of thermal expansion (per F° x 10- 6):	5.5
Thermal conductivity (BTU/hr-ft-F°) :	1.25
Heat capacity (BTU/lb-F°):	0.28

Mix Properties

Cement type:	Type I
Cementitious material content (lb/yd^3):	600
Water/cement ratio:	0.42
Aggregate type:	Limestone
PCC zero-stress temperature (F°)	111
Ultimate shrinkage at 40% R.H (microstrain)	Derived
Reversible shrinkage (% of ultimate shrinkage):	50
Time to develop 50% of ultimate shrinkage (days):	35
Curing method:	Curing compound

Strength Properties

Input level:	Level 3
28-day PCC modulus of rupture (psi):	690
28-day PCC compressive strength (psi):	n/a

APPENDIX D – BASELINE TRAFFIC DATA

Traffic

Initial two-way aadtt:	Variable
Number of lanes in design direction:	2
Percent of trucks in design direction (%):	50
Percent of trucks in design lane (%):	95
Operational speed (mph):	60

Traffic -- Volume Adjustment Factors

Monthly Adjustment Factors

(Level 3, Default MAF)

Month	Vehicle Class									
	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
February	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
March	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
May	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
June	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
July	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
August	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
September	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
October	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
November	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
December	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Vehicle Class Distribution

(Level 3, Default Distribution)

AADTT distribution by vehicle class

Class 4	1.8%
Class 5	24.6%
Class 6	7.6%
Class 7	0.5%
Class 8	5.0%
Class 9	31.3%
Class 10	9.8%
Class 11	0.8%
Class 12	3.3%
Class 13	15.3%

Hourly truck traffic distribution

by period beginning:

Midnight	2.3%	Noon	5.9%
1:00 am	2.3%	1:00 pm	5.9%
2:00 am	2.3%	2:00 pm	5.9%
3:00 am	2.3%	3:00 pm	5.9%
4:00 am	2.3%	4:00 pm	4.6%
5:00 am	2.3%	5:00 pm	4.6%
6:00 am	5.0%	6:00 pm	4.6%
7:00 am	5.0%	7:00 pm	4.6%
8:00 am	5.0%	8:00 pm	3.1%
9:00 am	5.0%	9:00 pm	3.1%
10:00 am	5.9%	10:00 pm	3.1%
11:00 am	5.9%	11:00 pm	3.1%

Traffic Growth Factor

Vehicle Class	Growth Rate	Growth Function
Class 4	4.0%	Compound
Class 5	4.0%	Compound
Class 6	4.0%	Compound
Class 7	4.0%	Compound
Class 8	4.0%	Compound
Class 9	4.0%	Compound
Class 10	4.0%	Compound
Class 11	4.0%	Compound
Class 12	4.0%	Compound
Class 13	4.0%	Compound

Traffic -- Axle Load Distribution Factors

Level 1: Site Specific

Traffic -- General Traffic Inputs

Mean wheel location (inches from the lane marking):	18
Traffic wander standard deviation (in):	10
Design lane width (ft):	12

Number of Axles per Truck

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.62	0.39	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.02	0.99	0.00	0.00
Class 7	1.00	0.26	0.83	0.00
Class 8	2.38	0.67	0.00	0.00
Class 9	1.13	1.93	0.00	0.00
Class 10	1.19	1.09	0.89	0.00
Class 11	4.29	0.26	0.06	0.00
Class 12	3.52	1.14	0.06	0.00
Class 13	2.15	2.13	0.35	0.00

Axle Configuration

Average axle width (edge-to-edge) outside dimensions,ft):	8.5
Dual tire spacing (in):	12

Axle Configuration

Tire Pressure (psi) :	120

Average Axle Spacing

Tandem axle(psi):	51.6
Tridem axle(psi):	49.2
Quad axle(psi):	49.2

Wheelbase Truck Tractor

	Short	Medium	Long
Average Axle Spacing (ft)	12	15	18
Percent of trucks	33%	33%	34%

APPENDIX E – PAVEMENT DAMAGE REGRESSION ANALYSIS

Table E.1 Multiple Linear Regression – Flexible Designs

Regression Statistics	
Multiple R	0.974657135
R Square	0.949956531
Adjusted R Square	0.942257536
Standard Error	1.271202532
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	398.77615	199.38807	123.38708	3.51E-09
Residual	13	21.00743	1.61596		
Total	15	419.78358			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	21.0795	0.6625	31.8204	0.0000	19.6483	22.5106	19.6483	22.5106
Load Shift	-0.0019	0.0001	-15.6855	0.0000	-0.0022	-0.0017	-0.0022	-0.0017
AADTT	-0.0001	0.0001	-0.8601	0.4053	-0.0003	0.0001	-0.0003	0.0001

Table E.2 Multiple Linear Regression – Rigid Designs

Regression Statistics	
Multiple R	0.785
R Square	0.617
Adjusted R Square	0.558
Standard Error	3.903
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	318.5100731	159.2550366	10.45400035	0.001966381
Residual	13	198.0405019	15.23388476		
Total	15	516.550575			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	24.9750	2.0340	12.2789	0.0000	20.5808	29.3691	20.5808	29.3691
Load Shift	-0.0015	0.0004	-4.0959	0.0013	-0.0024	-0.0007	-0.0024	-0.0007
AADTT	-0.0006	0.0003	-2.0327	0.0630	-0.0013	0.0000	-0.0013	0.0000

APPENDIX F – WESLEA STRAIN ANALYSIS

Tandem Axle Analysis

WESLEA for Windows - Simulation Output

*****STRUCTURAL INFORMATION*****

Layer	Modulus (psi)	Poisson	Height (in)	Slip
1	766796	0.35	10.25	1
2	30000	0.35	15	1
3	14000	0.35	999	1
4	14000	0.35	999	1
5	14000	0.35	Infinite	

*****LOADING CONFIGURATION*****

Axle Type: Other

Tire#	X (in)	Y (in)	Load (lb)	Pressure (psi)
1	0	0	4250	120
2	12	0	4250	120
3	12	51.6	4250	120
4	0	51.6	4250	120

*****ENGINEERING RESPONSES*****

		Coordinates (in)			Normal MicroStrain		
Loc#	Layer	X	Y	Z	X	Y	Z
1	1	0	0	10.25	-48.6	-56.11	60.25
2	1	6	0	10.25	-44.2	-59.07	59.53
3	1	6	25.8	10.25	-31.82	13.43	11.51
4	1	0	25.8	10.25	-29.09	11.96	10.77
5	3	0	0	25.25	-52.02	-36.92	133.88
6	3	6	0	25.25	-55.73	-38.04	139.66
7	3	6	25.8	25.25	-55.07	-13.54	115.09
8	3	0	25.8	25.25	-52.13	-13.94	111.78

Tridem Axle Analysis

WESLEA for Windows - Simulation Output

STRUCTURAL INFORMATION

Layer	Modulus (psi)	Poisson	Height (in)	Slip
1	766796	0.35	10.25	1
2	30000	0.35	15	1
3	14000	0.35	999	1
4	14000	0.35	999	1
5	14000	0.35	Infinite	

LOADING CONFIGURATION

Axle Type: Other

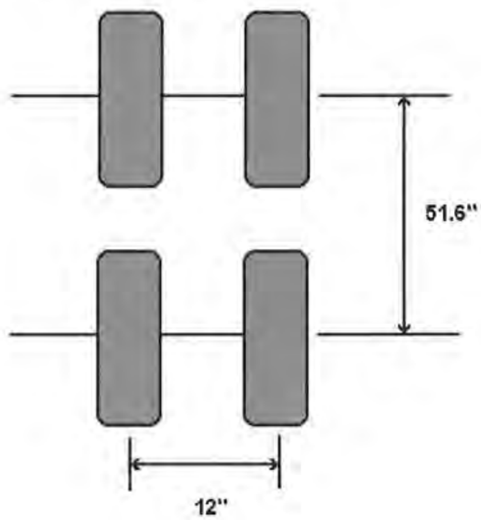
Tire#	X (in)	Y (in)	Load (lb)	Pressure (psi)
1	0	0	4250	120
2	12	0	4250	120
3	12	49.2	4250	120
4	0	49.2	4250	120
5	0	98.4	4250	120
6	12	98.4	4250	120

ENGINEERING RESPONSES

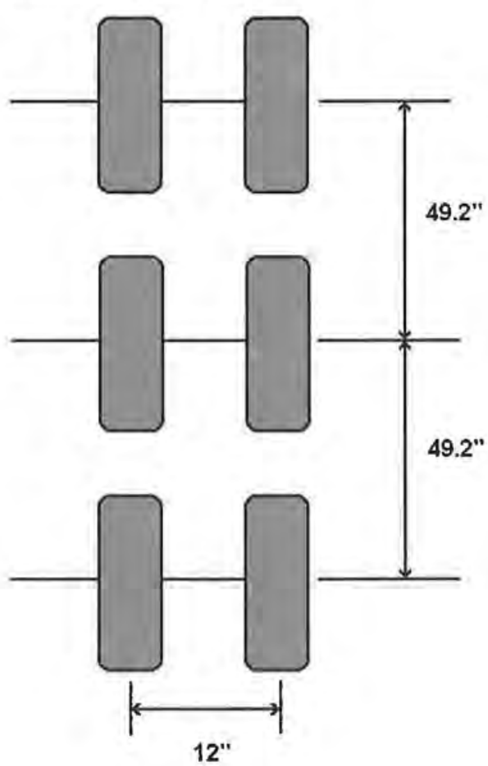
		Coordinates (in)			Normal MicroStrain		
Loc#	Layer	X	Y	Z	X	Y	Z
1	1	0	0	10.25	-49.73	-53.84	59.66
2	1	6	0	10.25	-45.37	-56.76	58.95
3	1	6	24.6	10.25	-35.57	16.43	12.08
4	1	0	24.6	10.25	-32.51	14.82	11.23
5	1	6	49.2	10.25	-49.95	-51.88	58.97
6	3	0	0	25.25	-55.78	-31.79	135.78
7	3	6	0	25.25	-59.58	-32.85	141.63
8	3	6	24.6	25.25	-63.04	-8.5	124.85
9	3	0	24.6	25.25	-59.79	-8.96	121.2
10	3	6	49.2	25.25	-70.06	-30.14	158.72

APPENDIX G – AXLE CONFIGURATIONS

Tandem Axle Configuration



Tridem Axle Configuration



APPENDIX H – FATIGUE CRACKING TRANSFER FUNCTION

$$N_f = k_{f1}(C)(C_H)(\beta_f 1)(\varepsilon_t)k_{f2}\beta_{f2}(E_{HMA})^{kf3\beta f3}$$

Where:

N_f = Allowable number of load applications

k_{f1}, k_{f2}, k_{f3} = Global field calibration parameters (from the NCHRP 1-40D recalibration; k_{f1} = 0.007566, k_{f2} = -3.9493 and k_{f3} = -1.281)

β_{f1}, β_{f2}, β_{f3} = Local or mixture specific field calibration constants; for this equation, these were set to 1.0

ε_t = Tensile strain at critical location (in/in) = 59.07 $\mu\varepsilon$ for tandem, 56.76 $\mu\varepsilon$ for tridem

E_{HMA} = Dynamic modulus of the HMA measured in compression (psi) = 767 ksi (weighted average from MEPDG output data)

$$C = 10^M$$

$$M = 4.84\left(\frac{V_{be}}{V_a + V_{be}} - 0.69\right)$$

V_{be} = Effective asphalt content by volume (%) = 11%

V_a = Percent air voids in the HMA mixture = 7.5%

$$C_H = \frac{1}{0.000398 + \frac{0.003602}{1 + e^{(11.02 - 3.49H_{HMA})}}}$$

Results:

$N_{f(tandem)}$ = 947,208,549 load applications

$N_{f(tridem)}$ = 1,108,827,164 load applications

APPENDIX I – SURVEY

CONTACT INFORMATION

- Name:
- Title: ____________________
- Agency: ____________________
- Address: ____________________
- City: ____________________
- State: ____________________
- Zip Code: ____________________
- Telephone : ____________________
- Fax: ____________________
- E-mail: ____________________

I would like to receive a summary of this survey when it is complete.

☐Yes ☐No

PERMITTING

1] Please check and briefly describe the criteria for your agency's overweight permit fee policy. Please attach (or enclose) relevant documentation, if available.

☐ Gross Vehicle Weight

____

☐ Axle Weight

____

☐ Axle Configuration

____

☐ Mileage

____

2] Have any efforts been made to relate the permitting fee structure to the cost of damage caused to highway or bridge by permitted overweight vehicles? If so, please briefly describe these efforts (cite reports, if available).

3] What are the typical types of overweight permits issued by your agency and associated costs for them? Specify sub-types if applicable.

☐ Overweight Single Trip Permit ______________________________

☐ Overweight Annual Permit ______________________________

☐ Combined Over Dimension Plus Over Weight Permit ______________________________

4] Do you have an online application process for overweight vehicle permits?

☐Yes ☐No If yes, please give web address: ______________________________

5] Which medium do you use to communicate change in regulations to your overweight permitted customers? (e.g., closure of route or change in route regulation)

☐ Posted signs along route
☐ Electronic message signs along route
☐ Website
☐ e-mail
☐ Broadcast Radio
☐ CB Radio
☐ Fax
☐ Mail
☐ Newspaper
☐ Television
☐ Telephone

6] Do you work together with neighboring state DOTs on maintaining uniform standards for overweight vehicle regulations? If yes, please briefly identify the efforts.

INFRASTRUCTURE DAMAGE ANALYSIS AND ASSESSMENT

7] Please identify from the following list an approach used by your agency to analyze the effect of overloads on pavement structure. Please mention the software used, if any.

☐ AASHTO Pavement Design Guide ______________________________

If AASHTO, which version? ☐ 1972 ☐ 1986 ☐ 1993

☐ Agency-Specific Pavement Design Method ______________________

☐ Mechanistic-Empirical Analysis ______________________________

☐ Bridge Analysis Rating System ______________________________

8] Have there been instances where pavement distress was caused, in your opinion, by a permitted overload vehicle? If so, please describe the instance, the distress, how it was measured (e.g., manual survey, automated road profiler, etc.) and how soon after the permitted vehicle traveled the route the distress was identified (attach additional pages if necessary).

9] What measures do you take to avoid pavement distress (e.g., spraying water on unpaved route, mandating additional axles, rerouting, etc.)?

10] Are the vehicle weight data collected as part of your monitoring and enforcement program shared with the DOT pavement and bridge design offices? ☐ Yes ☐ No

11] Are DOT pavement and bridge designers involved in the permitting process, either before permits are authorized or for engineering purposes?

Pavement Designers ☐ Yes ☐ No

Bridge Designers ☐ Yes ☐ No

12] Does the Department of Transportation estimate (measure, analyze and assess) infrastructure damage caused by trucks which exceed 80,000 lb and individual axle loads? If so, please explain.

WEIGHT LIMITS AND ENFORCEMENT

13] Do your agency's legal weight limits vary according to the following parameters? Please elaborate as necessary on the policy for each answered, "Yes."

A] Season / time of year (spring-thaw, harvest etc.) ☐ Yes ☐ No

B] Type of Commodity (e.g. hauling livestock, coal, sugarcane etc.) ☐ Yes ☐ No

C] Type of industry to which material is supplied (Government/private) ☐ Yes ☐ No

D] Particular routes ☐ Yes ☐ No

E] Type of vehicle (bus, truck, construction vehicle, etc.) ☐ Yes ☐ No

F] Type of load (divisible/ non-divisible) ☐ Yes ☐ No

G] Pavement condition (e.g., pavement with no distress vs. pavement with cracking, faulting, etc.) ☐ Yes ☐ No

14] How are vehicle weight limits enforced in your state? Check all that apply and indicate the number of operating stations.

☐ Weigh Stations (static scales) ________

☐ Weigh Stations (weigh-in-motion scales) __________

☐ Mainline Weigh-In-Motion __________

15] How are vehicle weights monitored in your state? Check all that apply and indicate the number of operating stations.

☐ Roadside Weigh Stations (static scales) __________

☐ Roadside Weigh Stations (weigh-in-motion scales) __________

☐ Mainline Weigh-In-Motion Stations __________

Please fill out the following tables regarding vehicle size and weight limits within your state.

16] WIDTH LIMITS

a. National Network	
Exceptions	
Grandfather Provisions	
b. Other Roads	
Exceptions	

17] HEIGHT LIMITS

State/Local Roads	
Exceptions	

18] LENGTH LIMITS (OTHER THAN 23 CFR PART 658 APPENDIX B AND C)

a. National Network	
Exceptions	
Grandfather Provisions	
b. Other Roads	
Exceptions	

19] GROSS VEHICLE WEIGHT LIMITS

a. Interstate	
Exceptions	
Grandfather Provisions	
b. Other Roads	
Exceptions	

20] SINGLE AXLE WEIGHT LIMITS

a. Interstate	
Exceptions	
Grandfather Provisions	
b. Other Roads	
Exceptions	

21]TANDEM AXLE WEIGHT LIMITS

a. Interstate	
Exceptions	
Grandfather Provisions	
b. Other Roads	
Exceptions	

22] FEDERAL BRIDGE FORMULA APPLICABILITY

a. Interstate	YES NO
Exceptions	
Grandfather Provisions	
b. Applicable to Other Roads	YES NO
Exceptions	

23] MAXIMUM PERMIT WEIGHT AND DIMENSIONS ALLOWED, NON-DIVISIBLE AND DIVISIBLE (IF APPLICABLE)

a. Interstate (for weight)	
b. National Network (for size)	
c. Other roads	

On behalf of the Federal Highway Administration and the Auburn University Highway Research Center, we thank you very much for your precious time!