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16. Abstract

The second year of this study focused on Texas highway corridors carrying heavy truck volumes and examined LCV operations—particularly infrastructure costs—to identify where the highest economic advantage from LCV implementation would be achieved. First an Executive Summary of the entire study describes the two-year findings. Then Chapter 1 provides a short background and summarizes the outlines of both reports. Chapter 2 covers the potential LCV impacts on the pavements of the high volume heavy truck corridors (segments) on the TxDOT system. The work is a fundamental contribution to estimating pavement life and critical in ensuring that any recommended increase of truck size or weight meets the marginal cost rule. The chapter covers the method used to determine pavement life, together with the collection and analysis of data required in the evaluation. It reports the characteristics, analysis, results, and conclusions for each of five Texas corridors. The chapter then summarizes the findings for both rigid and flexible pavement and closes with limitations and recommendations for additional research. Chapter 3 identifies LCV impacts on the bridges identified on the corridors specified in Chapter 2. It describes the method selected to determine bridge impacts, including the traditional moment analysis method and a fatigue moment analysis method that promises greater precision. The LCV types selected for study analysis—97,000 lb tridem, 138,000 lb double 53ft, and a 90,000 lb double 53ft—are then introduced sequentially and the results for both moment methods given. Results are then summarized with one surprising result. Chapter 4 provides the findings regarding users, pavements, and bridges. The major recommendation of the advisory panel was a pilot study of LCV types over a selection of Texas corridors that are economically attractive to truckers. Finally, a series of appendices covered supporting material to the analytical work undertaken in the second year and the presentations made at the final study workshop.

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The study team, over the entire two-year period, was guided by technical help in the operational aspects of heavy truck operations by Ken Allen and Mike Moynahan (H-E-B), Doug Miller (Frito-Lay), and Matt Stalter (Pepsi/Quaker Oats) who helped refine the types of productive trucks that could raise trucking efficiencies in Texas. Drs. Peter Sweatman and John Woodrooffe (University of Michigan Transportation Research Institute) also provided help consistently over the two-year period. Sarah Lind patiently put the work together and Maureen Kelly undertook the final edit and formatting.

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Potential Use of Longer Combination Vehicles in Texas: Executive Summary

Background

Like most other U.S. states, Texas is facing a highway funding shortfall, which means fewer miles of new highway and higher levels of congestion. Moreover, freight movement is expected to increase 40% by 2030. One way to move the additional freight without constructing new highway lanes is to allow more productive trucks on the current highway system. More productive trucks will mean an increase in size and weight. This change would reduce the number of trucks, the fuel consumed to move the goods, and the emissions created by the trucking sector. Many other countries, such as Canada and Australia, have successfully increased truck productivity by using Longer Combination Vehicles (LCVs).

The use of LCVs in the United States has been controversial. Although some western states allow them under a grandfather clause, federal law does not allow an increase in truck size and weight beyond 80,000 lb gross vehicle weight. However, a 2002 Transportation Research Board (TRB) Special Report concluded that an opportunity exists for larger trucks to operate under a carefully monitored system. Different interest groups have also expressed concern about safety, infrastructure impacts, and mode competition. In an attempt to address these concerns for Texas, this study focuses on specific key routes, safety issues, pavement impacts, bridge impacts, and industry feedback concerning the potential use of LCVs.

Chosen Routes

With increased freight traffic predicted for Texas, key corridors will play an important role. To obtain a broad cross-section, the Project Monitoring Committee (PMC) wanted sections of an existing state corridor, sections of the IH 35 route, and an existing state highway evaluated for LCV operations. The routes are short-haul distances—not competing with rail or other mode types. After discussing with Texas industry shippers such as H-E-B and PepsiCo/Frito-Lay, the research team chose the following five key routes, depicted in Figure ES.1:

- El Paso to Dallas (IH 20/IH 10)
- Dallas to San Antonio (IH 35)
- San Antonio to Laredo (IH 35)
- Dallas to Houston (IH 45)
- San Antonio to McAllen (IH 37/US 281)

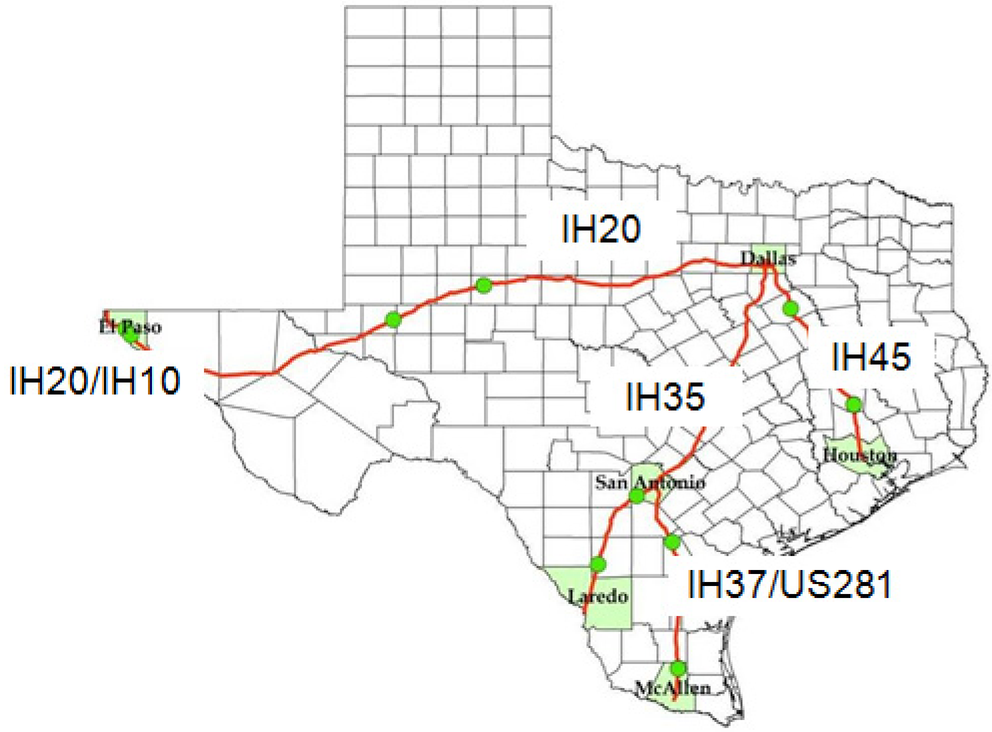

Figure ES.1: Selected Study Routes

Representative Vehicles

Different types of LCVs are used in Canada, Australia, and Europe. To decide which types would be safe and appropriate for Texas, the research team contacted companies interested in using LCVs. H-E-B and PepsiCo were both interested in using their current equipment in different configurations to create the LCVs. The first vehicle chosen was a 97,000 lb tridem semi-trailer (subsequently referred to in this report as *97-kip*). Next, the standard 53ft trailer was used for a combination double 53ft (termed a *double 53* in this report) at a maxed-out weight of 138,000 lbs (subsequently referred to as *138-kip*). After discussing with other companies such as Frito-Lay, researchers realized that not all double 53 trailers would be maxed out. Therefore, the idea of a "light" double 53, one that cubes out at 90,000 lbs (subsequently referred to as *90-kip*), was also incorporated. Note: the term "cube out" refers to a truck whose volumetric capacity has been reached, while some load capacity remains.

It is impossible to foresee in detail what the industry response would be if LCVs were permitted in Texas. Based on operator surveys and input from industry contacts, the researchers decided, in concert with the PMC, that the following LCV scenario would be realistic for this study:

- LCV approval would affect primarily FHWA (Federal Highway Administration) Class 9 vehicles ("18-wheelers");
- 15% of current truck cargo currently hauled by FHWA Class 9 vehicles would remain in this vehicle class;
- 35% would be transferred to the 97-kip tridems;
- 20% would be transferred to the light doubles; and,

- The remaining 30% would become the 138-kip double 53s.

For the purpose of this analysis, the total amount of cargo remains the same, and so do the proportions of axle loads observed at the WIM stations.

Safety

Safety is recognized as critical for all highway users. In Canada, LCVs have demonstrated a lower crash rate compared to other vehicle classes. In the states where LCVs are currently allowed, crash rates were lower due to operational restrictions. Results of the *Large Truck Crash Causation Study* (Ref. ES-1) suggested that LCVs enjoy significantly lower crash costs, per vehicle-mile traveled. Current LCV operations in the United States have different operational standards than the typical truck to increase safety. Different performance-based operations can limit when LCVs run (time of day, weather conditions), where they can run (specific routes, urban areas), and who can run them (more experienced, certified drivers). An accident means loss of time, possible loss of equipment, and possible loss of cargo, which, in general, results in productivity losses. Moreover, crashes also affect the industry's good relationship with their customers and the public. Therefore, many truck operations have more safety technology than required by law. Introducing LCVs will decrease the number of overall trucks on the highway system for a given tonnage of freight, likely resulting in a decrease in the number of crashes.

Research undertaken by the team over the two-year period suggests that two-trailer LCVs enjoy significantly lower crash rates but higher crash costs. They also offer the lowest crash costs per mile driven[1]. However, these findings were based on data collected several years ago and a fundamental change appears to be now taking place in the area of truck safety. A confluence of several factors is causing severe truck accident rates to fall and this development is critical to the public policy debate on more productive, larger trucks if it can be maintained over the next years, particularly as the U.S. economy recovers. Long combination vehicles have high safety rates in the latest data (2008) on fatal truck crashes from the Federal Motor Carrier Safety Administration (FMCSA) (Ref. ES-2). Out of a database comprising 4,066 fatal truck crashes, only 115 involved a double LCV and 2 involved triples. That figure is less than 3% of crashes. Truck crashes *in general* are also declining. Crash data from 2009 show the largest decrease in fatal truck crashes since the NHTSA's Fatality Analysis Reporting System (FARS) started collecting data in 1975. FARS indicates that 2009 saw a 20% decrease in fatalities, a 20% decrease in crashes, and a 21% decrease in large truck crashes. Some of these decreases can be attributed to the economic downturn and the resulting fewer vehicle miles traveled. However, the reduction in crashes is far greater than the reduction in vehicle miles of travel. Enforcement and outreach programs from FMCSA, NHTSA, the motor carrier industry, and safety groups have all contributed to this decline. Although analysis of information on LCV crash statistics is difficult because they are few compared to Class 9 truck crashes, the current information does not show LCVs to be any less safe than standard trucks—an argument often made, or inferred, by those objecting to their introduction. The data shows the value of emulating the current characteristics of LCV operations in the U.S. and enforcing them consistently.

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[1] A revised version of the paper entitled "Analysis of Large Truck Crash Severity Using Heteroskedastic Ordered Probit Models" by J. Lemp, K. Kockelman, and A. Unnikrishnan was submitted to the *Journal of Accident Analysis and Prevention* in June 2010. For an abstract of this paper, please see Appendix F.

External Costs

The external costs of allowing LCVs in Texas are those incurred in relation to public health and environment and therefore are borne by society as a whole. These costs are typically not included when deciding on a charging structure; however, they are important and warrant study. The environmental impacts of LCVs are demonstrably beneficial to society. The fuel economy of an LCV is lower than a standard 5-axle truck; however, the LCV carries more cargo. Therefore, operating LCVs lowers fuel consumption in terms of ton-mile units. Emissions can also be reduced through LCV use. Moving freight using a 140,000 lb LCV instead of two 80,000 lb single-trailer configurations reduced emissions by an estimated 27% per ton-mile of freight moved. Although a full impact analysis was not done on the external costs, research indicates that LCVs should have a positive impact on the environment.

Pavement Impacts

This study performed pavement analysis to estimate potential LCV impacts on the chosen routes. Each of the routes was divided into segments with uniform truck traffic, pavement, and subgrade type. Load spectra were developed for the existing conditions as well as for the LCV scenario. Additional information was obtained, including material properties, tire pressures, and detailed axle configurations. In total, the researchers analyzed 152 highway segments. For rigid pavements, the LCV scenario had no impact on pavement life. For flexible pavements, the LCV scenario improved the pavement life for all but one route.

The estimated pavement lives were used in calculating the annualized cost of a thick hot-mix overlay at the end of each cycle. Given the wide variation in overlay costs observed in previous studies, the researchers decided to calculate the annualized costs for three unit overlay costs (cost per lane-mile):

- Median price: $400,000 per lane-mile;
- 60% percentile: $607,000 per lane-mile; and
- Third quartile (75% overlays in the database cost up to that value): $1,219,000 per lane-mile.

Table ES.1 shows the estimated changes in the annualized costs of periodically overlaying the pavements if LCVs were allowed on these routes. The change in overlay costs is the difference between the annual cost of pavement with LCVs and the annual cost of pavement without LCVs. If the change is negative, it indicated a reduction in cost, shown in the table in red. The Dallas/Houston route does not see a change because it is rigid pavement.

The pavement analysis was conducted using data from available weigh-in-motion stations, whose locations are indicated by the green dots in Figure ES.1. Some stations, especially one on IH 20 between Dallas and El Paso, have a considerable amount of overweight tandem axles in Class 9. Some of the positive LCV impacts are due to the fact that the LCV scenario used in this study transfers 85% of this total cargo to LCVs with either legal tandem axles, or heavier tridems, and both are less detrimental to existing pavements than overweight tandems. Accurately ascertaining pavement impacts requires a more detailed study encompassing several scenarios, including some that maintain all overweight class 9 axles as well as some that include overweight LCVs.

Table ES.1: Pavement Costs

	Length (mi)	Overlay Δ_{cost} (\$millions/year)		
		$0.4m	$0.6m	$1.2m
Dallas El Paso	667	$(15.00)	$(22.77)	$(45.76)
Dallas Laredo	446	$(2.54)	$(3.85)	$(7.75)
Dallas Houston	261	$ -	$ -	$ -
San Antonio McAllen	243	$0.14	$0.22	$0.44
TOTAL	1,617	$(17.40)	$(26.40)	$(53.07)

Bridge Results

In total, this study analyzed 1,713 bridges in the route segments. Axle spacing and axle loads were established for each of the LCV configurations discussed previously in the Representative Vehicles paragraph. The bridge analysis was based on moment ratios. Research indicates that newer bridges (built since the 1980s) can support a 20% overstress, while older bridges can support a 10% overstress. Accordingly, two moment ratios of 1.1 and 1.2 were used to determine which bridges are deemed deficient under the proposed LCV scenarios—1.1 and 1.2 moment ratios indicative of 10% and 20% overstress respectively. Two sets of results were calculated using this traditional approach and by excluding the bridges that are already deficient for the existing traffic. One set used the 10% overstress criteria and one set used the 20% overstress criteria. Both overstress scenarios used the bridge inventory rating recorded in the Bridge Inspection and Appraisal Program (BRINSAP) as the basis. The 90-kip double 53 configuration showed no impact on the bridges of the selected case study routes for both overstress ratios. Results for the 97-kip tridem and 138-kip double 53 are summarized in Tables ES.2 and ES.3 respectively for the 20% overstress over Inventory Rating limit. Cost estimates for replacement of these bridges are based on an estimated cost of $190/sq ft of deck area. This unit cost of replacement was determined during a recent 2030 Committee study (Ref. ES-3). Impacts for a 10% overstress over Inventory Rating are higher and amount to $2.8 billion and $1.2 billion for the 97-kip tridem and 138-kip double 53 respectively.

Table ES.2: Cost to Replace Deficient Bridges for the 97-kip Tridem LCV Configuration (20% Overstress, Inventory Rating)

Highway	Deficient Bridges	Deck Area (sq ft)	Cost $
IH 10	126	836,570	158,948,357
IH 20	189	1,274,125	242,083,712
IH 35	183	2,938,770	558,366,357
IH 45	47	643,122	122,193,237
IH 37	14	137,679	26,158,972
US 281	23	164,369	31,230,015
Total	582	5,994,635	1,138,980,650

Table ES.3: Cost to Replace Deficient Bridges for the 138-kip Double 53 LCV Configuration (20% Overstress, Inventory Rating)

Highway	Deficient Bridges	Deck Area (sq ft)	Cost $
IH 10	7	148,468	28,208,844
IH 20	27	370,349	70,366,348
IH 35	95	3,772,940	716,858,562
IH 37	30	644,855	122,522,488
IH 45	14	316,820	60,195,876
Total	173	5,253,432	998,152,118

Not all bridges would have to be replaced immediately depending on the overstress level. To incorporate this concept in the analysis, the project developed a new fatigue approach with the assumption of a 75-year fatigue design life for a bridge. If the moment ratio is between 1.2 and 1.4, the bridge is assumed to have its life shortened by fatigue effects and, depending on its age, trigger an earlier replacement than the assumed 75-year life. Bridges with a moment ratio greater than 1.4 would have to be replaced immediately. Results for this analysis approach amount to $1.0 billion and $0.8 billion for the 97-kip tridem and 138-kip double 53 respectively, with no impacts for the 90-kip double 53 configuration.

Performance-Based Programs and the Industry

Communication with people in the trucking industry has been vital to this study. Along with giving details on which vehicles and routes were feasible, H-E-B and PepsiCo/Frito-Lay provided some of their own company-specific input. They currently use more advanced safety technology than is required by law. The companies stated interest in possibly expanding their operations, bringing more jobs and more money to Texas, if LCVs were allowed. The companies also expressed their interest in having a certification program for LCVs. They agree that performance-based standards would be the most efficient and safe way to operate LCVs. They are also big supporters of a possible pilot test.

Summary of Findings

The findings of this study reference the chosen routes in Texas and representative LCVs defined in this paper. The following is a summary of these findings:

- No causality link can be shown from current data between LCVs and an increase in crashes.
- Allowing LCVs can reduce fuel consumption and emissions.
- LCVs do not have a significant impact on pavement and may reduce pavement costs.
- The light double 53 has no impact on bridges.
- The 97-kip tridem and 138-kip double 53 would require either bridge replacement or strengthening on 880 (for $1.0 Billion) or 201 (for $0.8 Billion) bridges, respectively.
- Performance-based standards have been successful in other LCV operations and are favored by Texas shippers.

Recommendations

Now that the initial research has been completed, we recommend the following:

- Conduct bridge inspections on chosen routes.
- Determine precise first and last mile routes for origins and destinations.
- Develop a detailed bridge and pavement analysis of additional LCV scenarios, which would include other LCV types as well as different levels of LCV impact on the existing overweight tandems and other trucks.
- If most of the existing overweight tandems detected in the WIM stations are from special permits, conduct detailed bridge and pavement inspections on these routes. This step would give important practical information about heavy axle impacts on bridges and pavements.
- Determine adequacy of the geometric characteristics of the routes.
- Specify performance-based standards.
    - Time, weather, experience of driver
- Collect empirical data on operations of the trucks.

These recommendations could be studied through an LCV pilot test on the routes analyzed. A pilot program would allow engineers and researchers to gain hard data on LCV operations while working closely with the trucking industry to learn what is feasible as well as the safest, most efficient way to allow LCVs in Texas.

Chapter 1. Introduction

1.1 Background

A significant portion of economic activity in the United States depends on efficient commercial truck operations.[2] The U.S. Bureau of Transportation Statistics (BTS) estimates that trucking, in terms of all commercial freight activity in the U.S., accounts for 64% of the value, 58% of the weight, and 32% of the ton-miles. Yet the industry faces serious problems maintaining an effective role in the competitive world of transportation services. The 2006–07 rise in fuel prices, when combined with the 2008–09 economic recession, drove many companies to bankruptcy. The sector is also now adjusting to new hours of service rules, an impending driver shortage, and 10% higher prices for Class 9 2010 tractors—the workhorse of the sector—as a result of new EPA exhaust emission standards. Compounding the problem is the widely accepted position that those responsible for the provision of highways have insufficient revenues to maintain truck routes to the highest standards and build new lane miles of pavement to carry the forecasted future demand.

The future of U.S. trucking was the subject of December 2010 meeting of trucking executives at the Swedish Embassy[3] in Washington D.C. where these issues were fully discussed. American Trucking Association President Bill Graves stated that "A growing nation means more people so we are going to have more autos, more freight and more trucks and that means more traffic and congestion. Truck productivity is a critical element of how we resolve that growing issue." Texas has one of the highest projected state population rates of growth in the U.S. so his remarks carry a clear message for TxDOT and the state legislature, a point noted by Dr. John Woodrooffe of the University of Michigan Transportation Research Institute (UMTRI), who is also an external advisor to study 0-6095-1.

> "Truck size regulations are limited by policy and not technology, and it's as simple as that."
>
> John Woodrooffe, UMTRI

Truck size and weight regulations therefore play an important role in determining the efficiency and productivity of the U.S. economy. Truck productivity is impacted by vehicle technologies, changes in size and weight, fuel costs, and operational regulations such as driver hours. Large truck operations are made more complicated and thus more expensive by different regulations at both national (e.g., North American Free Trade Agreement [NAFTA]) and state levels. In Texas, trucks play a critical role in supporting the state economy although trucks on the federal-aid system must adhere to vehicle size and weight laws that are little changed since 1982[4].

Size and weight dimensions, however, have changed substantially in the rail[5], vessel[6], and air[7] modes, allowing them to benefit from economies of scale. Texas, like almost all other U.S. states, is facing a highway funding shortfall, which translates to fewer miles of new

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2 http://www.bts.gov/publications/national_transportation_statistics/html/table_01_46b.html
3 Sponsored by Volvo Trucks North America and the American Trucking Association
4 Paradoxically, higher gross and axle loads are permitted under permit under HB 2060.
5 Union Pacific recently tested an 18,000 ft intermodal train using seven locomotives from Texas to California.
6 Maersk introduced an 11,000 TEU containership in 2006.
7 The Airbus A380 can carry over 600 passengers and a freighter version is being developed.

highway and higher levels of congestion. The trucking sector—or at least that portion representing the largest companies—has recently asked federal and state governments to increase the truck size and weight limits. This issue is not new. In the late 1980s, trucking companies strongly argued for similar increases that were the subject of several research studies, sponsored by the Transportation Research Board, the American Trucking Association, and the Association of American Railroads. The technical debate, at times acrimonious, was finally shelved when Congress decided to "freeze" the federal limits in 1993, effectively passing the debate to individual states and non-federal-aid highways. So why has the question re-emerged as a relevant policy issue now?

The answer, or at least a good part of it, is a two-fold consequence of reduced highway funding and falling truck productivity. All state Departments of Transportation are experiencing the current full-blown funding crisis caused by shortfalls in fuel tax revenues and registration fees, neither of which have changed in Texas in over 15 years. Strategies to address mobility (congestion) have therefore been cut back, creating the specter of higher passenger and freight delays in the coming decade. Legislation that permits higher truck productivity, in the form of increased size and weight would, some argue, reduce the *numbers* of trucks, the *fuel consumed* to move the goods, and the *emissions* created by the trucking sector. Many countries, ranging from our NAFTA partners, the European Union, much of Latin America, and Australia permit heavier trucks and appear to keep their highways in good condition. TxDOT decided that it was time to re-visit the issue and sponsored study 0-6095 to evaluate the consequences for the Texas highway infrastructure of allowing heavier vehicles to operate in the state. These vehicles collectively are known as Long Combination Vehicles (LCVs), which actually includes the most prolific heavy vehicle type in the world: a semi-trailer vehicle with a tridem trailer axle that is not a true LCV[8].

The current commercial vehicle regulatory framework in the U.S. can be regarded as a "hybrid" and reflects compromises reached in framing and passing the 1982 truck size and weight regulations. A gross limit of 80,000 pounds (with various axle limits) became the standard interstate truck, thus allowing truckers in many states—including Texas—to raise their total payloads. However, a small number of states—mostly in the western U.S.—allowed higher trucking weight and longer vehicles. A compromise was reached whereby these states were "grandfathered" for LCV operation. Trucking fleets moving freight between jurisdictions that allow LCV operations, and those that do not, must either make additional investments in maintaining equipment or forfeit the efficiencies of heavier and longer trucks (Mercier, 2007). Critics of LCVs are concerned with LCV safety, notwithstanding the likelihood that larger trucks could reduce the numbers of trucks on any highway segment shared with smaller lighter vehicles. The experiences of operators in the grandfathered states provide a survey opportunity to evaluate actual experiences with LCV use.

1.2 First-Year Report

The study is documented in two reports. The first 0-6095-1 entitled "*Potential Use of Longer Combination Vehicles in Texas: First Year Report*"[9] (Ref. 1-1) details the work completed in four broad areas. First, the background to LCV studies was noted and the wide

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8 The term *LCV* typically refers to a tractor pulling at least two trailers, while tractors pulling multiple trailers, like those in Australia, are termed *road trains*.

9 www.utexas.edu/research/ctr/pdf_report/0_6095_1.pdf

range of truck regulations governing size and weight in the U.S. described. Previous studies balanced the productivity gains from heavy truck use with the incremental consumption of highway infrastructure to balance the costs and benefits and to identify the range of additional fees that LCV users would be required to pay[10]. This approach would address one of the main complaints raised by the rail industry—that current highway cost allocation methods fail to charge trucks the full cost of their infrastructure consumption. Bridge replacement[11], together with related user costs arising from traffic disruption, are the greatest financial obstacles to the adoption of heavier vehicles although the type of truck and the precise route are generally not evaluated in such studies. However, clearly pavement and bridge costs are critical in LCV evaluations and these comprised the second group examined in the first year report. The experiences of those operators currently permitted to operate LCVs in the U.S., together with operational characteristics of LCVs, were the subject of the third focus area. Finally, the recommendations of a project workshop evaluating the first year's work—comprising TxDOT, operators, and research staff—and the proposed evaluation method to be adopted in the second year completed the fourth, and final, area of activities completed in the first year.

Specifically, the contents for the first-year report were as follows. Chapter 2 examined the U.S. federal LCV regulations, vehicle dimensions, and LCV operations by state. Chapter 3 examined the LCV impacts on pavements, identifying methods for computing life expectancy and pavement costs with a proposed pavement analysis method. Chapter 4 examined a major issue with LCV operations, namely that of bridge impacts. It reports the design loads and rating system used in bridge design and the proposed method for estimating LCV bridge impacts in this study. It describes several analytical methods used in previous size and weight studies, reports on bridge strengthening and bridge fatigue, and closes with relating the proposed bridge impact method to the second-year case studies of Texas LCV routes. Chapter 5 examined LCV operational characteristics, including acceleration, off-tracking, and stability. Chapter 6 examined environmental and energy issues as LCV operations decrease both in terms of ton-mile emissions. Chapter 7 examined the safety aspects of LCV operations based on previous studies, available accident databases, and analyses undertaken as part of this study. Chapter 8 reported the findings of a U.S. LCV operator survey conducted as part of this study, capturing views on vehicles, drivers, performance, and safety. Chapter 9 described a workshop of LCV experts, LCV operators, larger truck users, researchers, and the TxDOT advisory team. Finally, five appendices describe LCV regulations and operations in the European Union (EU) and Australia, NAFTA size and weight harmonization, the proposed project database for pavement and bridge analyses, and the workshop agenda and attendees and a CD of the workshop presentations.

1.3 Second-Year Report

This second-year report, 0-6095-2, is entitled "*Potential Use of Longer Combination Vehicles in Texas: Second Year Report*" and reflects the proposed second-year program approved at the first-year study workshop by the advisory panel. Previous research on the implementation of LCV legislation generally analyzed large networks (state or federal) and, as a result, reported large costs—especially in the bridge area. Yet large trucks concentrate on only a segment of any state network. In a recent TxDOT study examining state multi-tier pavement maintenance goals, over 72% of large trucks were estimated to use just 22% of the on-system network. With this in

[10] Assuming that heavy trucks pay the marginal cost and no cross-subsidization is contemplated.
[11] This term covers a range of actions from retro-fitting modest improvements to full replacement.

mind, it was decided to focus on those segments carrying heavy truck volumes in the state and examine LCV operations—particularly infrastructure costs—to identify where the highest economic advantage from LCV implementation would be achieved. Accordingly, a number of key segments on the TxDOT network were selected as case studies to be analyzed in detail during the second year to determine both pavement and bridge impacts. A critical outcome of approaching the issue through infrastructure costs is the determination of the marginal costs that LCVs would need to cover for an entirely equitable cost allocation solution.[12]

The contents of the second-year report are as follows. First, an Executive Summary of the entire study precedes this chapter and describes the two-year findings. Chapter 2 covers the potential LCV impacts on the pavements of the high volume heavy truck corridors (segments) of the TxDOT system. The work is a fundamental contribution to estimating pavement life and critical in ensuring that any recommended increase of truck size or weight meets the marginal cost rule. The chapter covers the method used to determine pavement life, together with the collection and analysis of data required to undertake the evaluation. It then reports the corridor characteristics, pavement analysis, results, and conclusions for each of five corridors: Dallas–San Antonio; San Antonio–Laredo; Dallas–Houston; Dallas–El Paso; and finally San Antonio–McAllen. The chapter then summarizes the findings for both rigid and flexible pavement and closes with the limitations of study and recommendations for additional research.

Chapter 3 is the second critical element of the study, identifying the LCV impact on the bridges located on the corridors specified in Chapter 2. It describes the method for determining bridge impacts, including the traditional moment analysis method[13] and a fatigue-based moment analysis method that promises greater precision. The LCV types selected for study analysis—comprising a 97-kip tridem, a 138-kip double 53, and a 90-kip double—are then introduced sequentially and the results for both the traditional moment and fatigue-based moment methods are reported. Results are then combined and summarized with one surprising result. Chapter 4 provides the findings on the topics of users, safety, pavements, and bridges. The major recommendation of the advisory panel was a pilot study of LCV types over a selection of Texas corridors that were economically attractive to truckers. This pilot would be monitored over a period long enough to determine the full equitable cost to be paid by the trucking industry to cover all infrastructure costs and safety issues. Finally, a series of appendices are given that cover supporting material to the analytical work undertaken in the second year and the presentations made at the final study workshop.

[12] This determination has been the most serious demand from the railroad industry when debating LCV implementation.
[13] Used in almost all LCV impact analysis since 1990.

Chapter 2. Potential LCV Impacts on Pavements

2.1 Chapter Objective and Organization

This chapter documents the estimated impacts of LCV on the pavement infrastructure of the following Texas corridors, selected in concert with the Project Management Committee (PMC) and other stakeholders as case studies (Ref. 2-21):

- IH 20/IH 10 from Dallas to El Paso.
- IH 35 from Dallas to Laredo. This corridor was divided into two sections for pavement analysis purposes:
  - Dallas to San Antonio, averaging approximately 8,000 trucks per day, and
  - San Antonio to Laredo, averaging approximately 4,000 trucks per day.
- IH 37/US 281 from San Antonio to Weslaco/McAllen.
- IH 45 from Dallas to Houston.

This chapter discusses the pavement analysis methodology, hypotheses, and assumptions; summarizes the data collected and explains data reduction procedures; documents the results of the pavement and cost analyses for each study corridor; and closes with conclusions and recommendations.

2.2 Analysis Methodology

2.2.1 Objective

The objective of this pavement analysis is to estimate the relative cost impacts a LCV scenario, developed in concert with the PMC, would have on pavements, using the four aforementioned corridors as case studies. This is neither a full-cost nor a cost allocation study; rather, its objective is to develop comparative measures of LCV impacts that can be used to aid in the decision of allowing (or not) LCVs in Texas. The study's calculations are limited to the effects of axle loads on pavements. Nevertheless, if LCVs are approved, it will be necessary to estimate their share of environment-related pavement damage for permit and registration pricing purposes.

2.2.2 Pavement Life Estimates

The choice of an approach to analyze the pavements was based on a combination of technical and practical criteria, taking into account the fact that the Mechanistic-Empirical Pavement Design Guide (MEPDG) has not been adopted in Texas yet. As a matter of fact, a study is in progress with the objective of adapting this methodology to Texas conditions (study 0-6622).

No approach is error-free or uncertainty-free, especially because the main input in pavement analysis, the traffic data, is always affected by considerable yet inevitable uncertainties. Several recent studies emphasize and quantify this point. Papagiannakis et al. examined the impact of traffic input on MEPDG pavement life predictions. They estimated pavement life as well as prediction errors using mean traffic input and low-percentile input to the National Cooperative Highway Research Program (NCHRP) 1-37A design guide for three levels of confidence (75%, 85%, and 95%). Prediction errors from mean traffic input ranged from as low as 5.3% to as high 81.2% depending on data availability for the pavement under consideration (Ref. 2-12).

This impact of traffic data uncertainties on pavement performance predictions was also examined in a recent Arizona study that performed a sensitivity analysis of MEPDG input traffic parameters to the pavement performance predictions using different data sources. Findings showed that average daily truck traffic (ADTT) varied significantly between two data sources (the Long-Term Pavement Performance [LTPP] program and the Arizona Department of Transportation), resulting in large differences in predicted cracking. A further sensitivity analysis revealed that the predicted cracking increased by a larger factor with respect to increases in ADTT. The use of national default load distribution factors revealed a similar result, such that the errors associated with predicting cracking were large (Ref. 2-1).

The mechanistic-empirical approach itself is not without controversy. In 2008, the Texas Transportation Institute (TTI) developed and tested a parametric Bayesian formulation to predict distress values used to plan road repairs. Large inherent variability in measured cracking and a small number of observations are the nature of pavement cracking data, necessitating special approaches that mathematically include prior engineering knowledge. The study showed that a Bayesian formulation gives sensible predictions with defendable uncertainty statements. The method was demonstrated on data collected by TTI at several sites in Texas and its predictions behaved in a reasonable and statistically valid manner (Ref. 2-13).

The facts discussed above led to the decision to evaluate LCV impacts based on the pavement structural response to the axle loads. After considering and investigating several methods (Ref. 2-4, 2-9, 2-15, 2-22, 2-23), the team selected KENPAVE, a package that calculates the mechanistic response (stresses and strains) and applies the fatigue formulas discussed below to estimate the pavement life under a certain load spectra. This package has two programs, KENLAYER and KENSLABS, respectively for flexible and rigid pavements (Ref. 2-8).

KENLAYER accepts as input the load spectra and calculates strains at locations specified by the user. For flexible pavements, the consensus is that the most critical strains are:

- The horizontal tensile strain at the bottom of the asphalt layer, which causes alligator cracking, and
- The vertical compressive strain on top of the subgrade, which causes permanent deformation or rutting.

The allowable number of axle repetitions is estimated using Equations (2.1) and (2.2), respectively for rutting and alligator cracking (Ref. 2-8, 2-11).

$$N_r = 1.365 * 10^{-9}\ \varepsilon_c^{-4.477} \tag{2.1}$$

$$N_a = 0.0795\ \varepsilon_t^{-3.291} E^{-0.854} \tag{2.2}$$

where:

N_r = number of allowable repetitions for rutting (rut depths of ½ inch)

N_a = number of allowable repetitions for alligator cracking (50% of the wheel path area cracked)

ε_c = compressive strain on top of the subgrade

ε_t = tensile strain at the bottom of the asphaltic layer

E = elasticity modulus of the top layer (psi)

Pavement life expectancy is estimated as the minimum of two ratios:

$$\frac{N_r}{N} \quad \text{and} \quad \frac{N_a}{N} \tag{2.3}$$

where N is the number of repetitions in the load spectra of the pavement section in question.

KENSLABS also takes as input the load spectra. It calculates the maximum stress in the concrete layer, then applies the Portland Cement Association's fatigue criteria to estimate the number of allowable repetitions according to Equations 2.4, 2.5, and 2.6 (Ref. 2-8, 2-11, 2-15).

For $$\frac{\sigma}{S_c} \geq 0.55 : \log N_f = 11.737 - 12.077\left(\frac{\sigma}{S_c}\right) \tag{2.4}$$

For $$0.45 < \frac{\sigma}{S_c} < 0.55 : N_f = \left(\frac{4.2577}{\sigma / S_c - 0.4325}\right)^{3.268} \tag{2.5}$$

For $$\frac{\sigma}{S_c} \leq 0.45 : N_f = \infty \tag{2.6}$$

Where σ is the stress in slab and S_c is the concrete's modulus of rupture. Units use in the calculations were psi.

This approach is based on mechanistic responses (i.e., stress/strain) and clearly does not account for the effects of the environment; hence, it is meaningful in relative terms, which is appropriate as well as necessary for this study.

The calculations were made for the most critical traffic direction, assuming that all trucks are in the design (rightmost) lane. The most critical direction was determined using the equivalent single axle loads (ESALs) reported for each weigh-in-motion (WIM) station (Ref. 2-6, 2-7, 2-20). The calculations utilized the direction with the highest ESALs, as the other direction would be less critical.

2.2.3 LCV Utilization Scenario

It is impossible to foresee in detail what the vehicle mix will be after a possible approval of LCVs in Texas. Some operators will not change from the current 80K five-axle semi-trailer vehicles (Class 9) because of routes, urban deliveries, costs of LCV registration and fees, etc. Operators that do adopt the new configurations will probably do so gradually, although some will certainly modify their fleets almost immediately. The LCV analysis scenario reflects the point of "long-term equilibrium" for the industry. In other words, it reflects a situation where all operators have fully adopted the truck configurations that best suit their business models. Within this concept, this is a critical but still realistic LCV analysis scenario. It was developed in concert with TxDOT, and is based on information provided by industry stakeholders, operator surveys, and information about LCV use in other states (Ref. 2-10). The LCV scenario and its underlying assumptions are listed here and summarized in Figure 2.1.

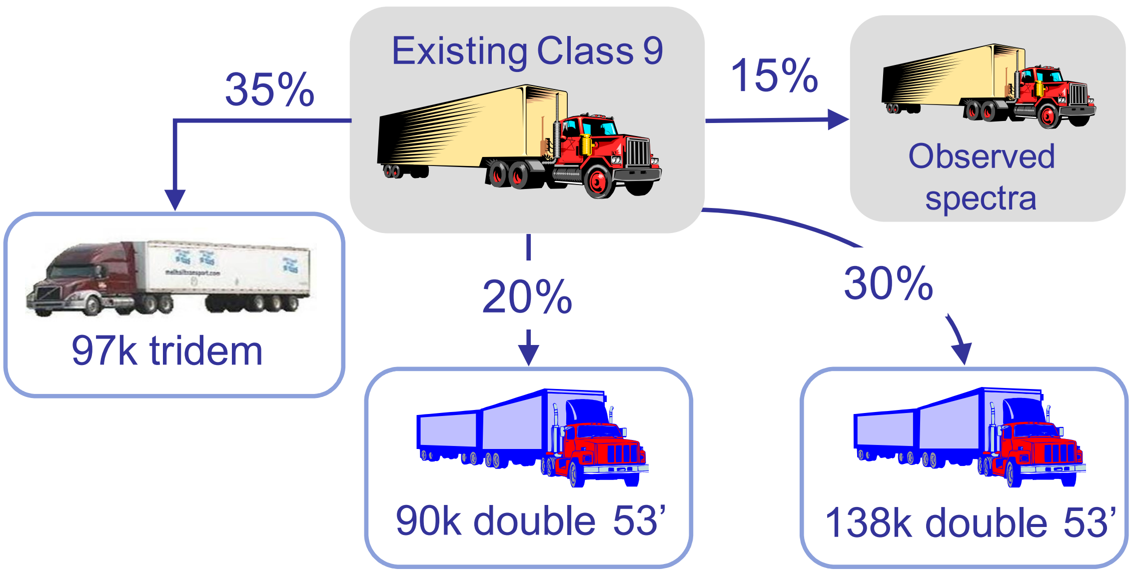


Figure 2.1: LCV Scenario

- LCVs will impact FHWA Class 9 vehicles (“18-wheelers”). Other vehicle classes either will not change, or the change will be insignificant. In addition, the proportions of axle loads observed in the existing WIM stations will remain the same.
- 50% of all cargo hauled by FHWA Class 9 vehicles will be transferred to double 53 trailers, possibly with A trains. B trains might be preferred or mandated, but the articulation type is not relevant for pavement and bridge analysis.
- In the group above, 40% will cube out at 90 kips (65% of the maximum gross weight) and 60% will gross out at 138 kips. Note: the term “cube out” refers to a truck whose volumetric capacity has been reached, but some load capacity remains.
- 15% of all cargo hauled by FHWA Class 9 vehicles will stay with this type of vehicle.
- 35% of all cargo hauled by FHWA Class 9 vehicles will run as the 97-kip tridem depicted in Figure 2.2.
- The analysis horizon was initially proposed as 10 years. However, this horizon proved too short when compared with most estimated fatigue lives; the cost annualization was developed for periodic overlays, as explained later in this chapter.

The 97-kip tridem follows the specification depicted in Figure 2.2. In the analysis, axle loads were rounded to 7 kips in the steering axle, 34 kips in the tandem axle, and 56 kips in the tridem axle.

For the double 53 configuration, the steering axle is assumed to weigh 12 kips. The remainder of the vehicle weight is assumed to be distributed evenly among the four tandem axles. With the maximum gross weight of 138 kips, each of the remaining 4 axles has 31.5 kips. With the maximum of 90 kips (assumed for cubed-out LCVs), each remaining axle weighs 19.5 kips.

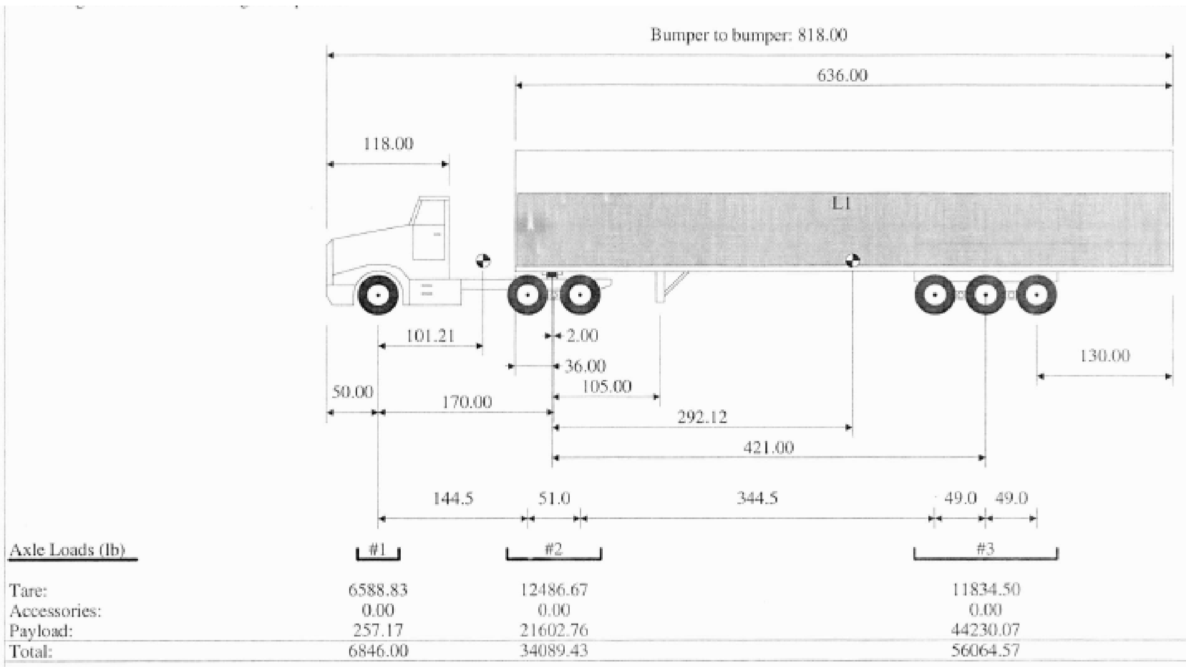


Figure 2.2: Proposed 97-kip Tridem Axle Configuration and Load Limits

Note: all weights are in pounds and all lengths in inches.

2.2.4 Implications of the LCV Scenario on the Load Spectra

The LCV scenario changes the pavement load spectra in the following ways:

- Tandem and single axle weight limits remain the same (ref. 2.19);
- Single and tandem axle repetitions decrease;
- Number of overweight single axles decrease;
- Number of overweight tandem axles decrease;
- Maximum tridem axle weight increases; and
- Heavy tridem repetitions increase considerably.

Figures 2.3, 2.4, and 2.5 illustrate the points above, respectively for single, tandem, and tridem axles. They compare the load spectra with and without the LCV scenario for WIM station 539, located on IH 45 near Dallas. Figures 2.3 and 2.4 show the decrease in overweight axles caused by the LCV scenario, with Figure 2.3 clearly indicating a smoother axle weight distribution for the LCV scenario. The increase in tridem axle repetitions is so significant for the LCV scenario that Figure 2.5 required a logarithmic scale in the vertical axis in order to display the two-scenario comparison.

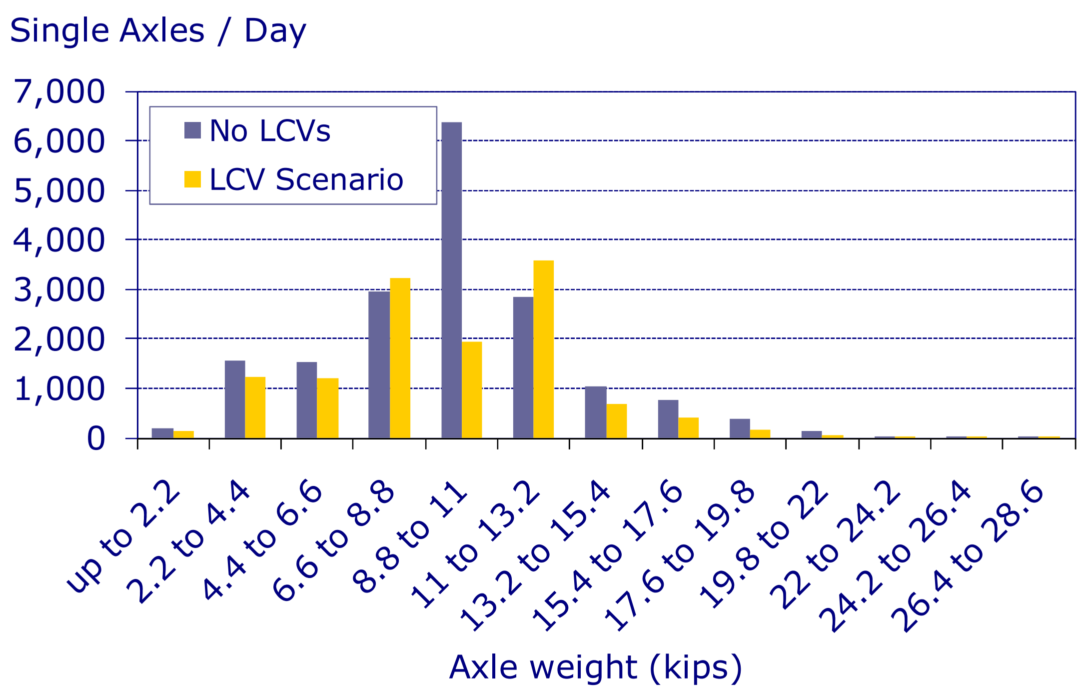


Figure 2.3: Single Axle Load Spectra at WIM Station 539

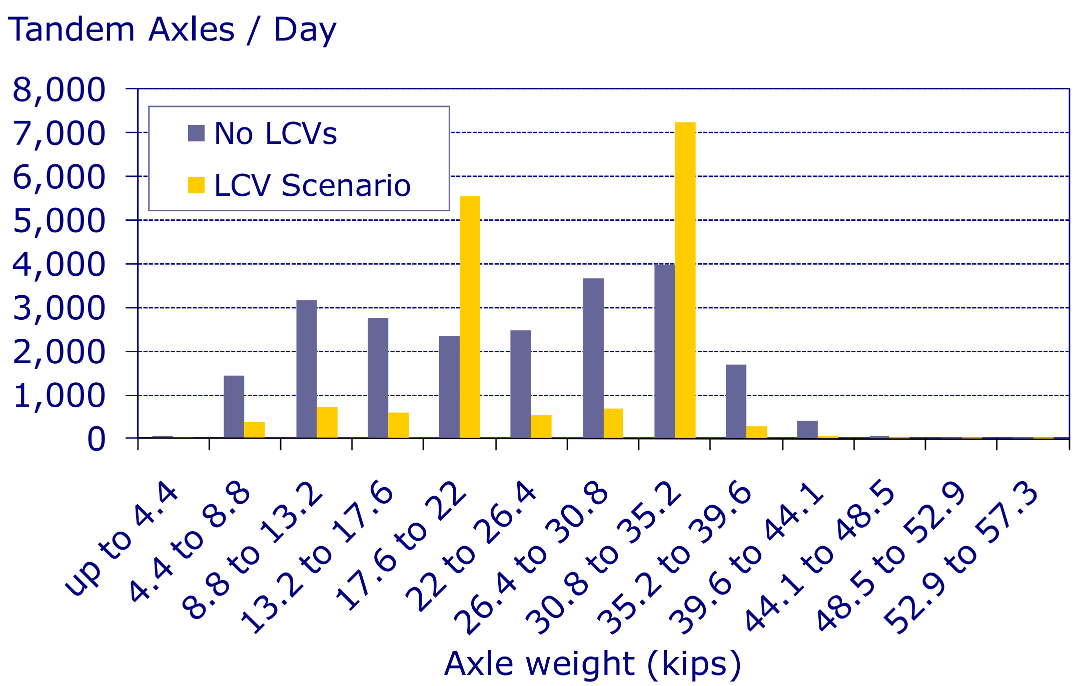


Figure 2.4: Tandem Axle Load Spectra at WIM Station 539

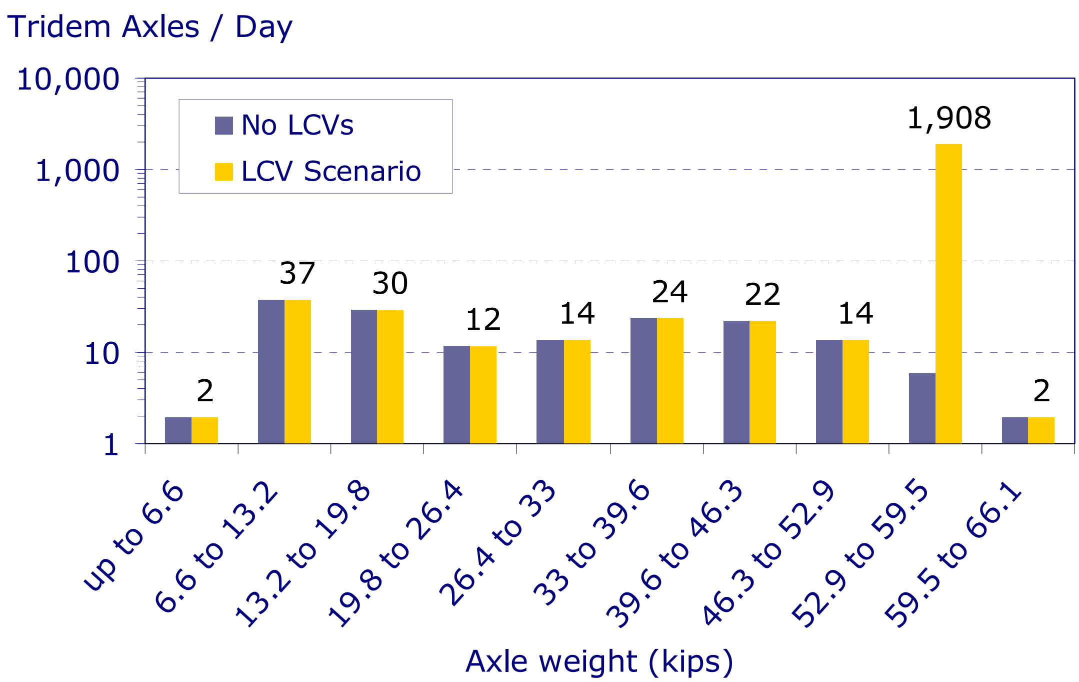


Figure 2.5: Tridem Axle Load Spectra at WIM Station 539

2.2.5 Measures of LCV Impacts: Δ_{life} and Δ_{cost}

The basic *measure of impacts* (MOI) in this LCV analysis is Δ_{life}, the difference between the expected lives (in years) of an existing pavement with and without the LCV utilization scenario. Three possible outcomes for Δ_{life} are:

1. *Non-significant* $\Delta_{life.}$. This outcome happens in two ways: (1) Δ_{life} is less than one year, or (2) lives are greater than 30 years in both scenarios, i.e., the pavement is insensitive to axle load repetitions of the magnitude observed in this project (this happened primarily for rigid pavements, as discussed later in this chapter). Cost impact is zero in both cases.

2. *Significant and negative* Δ_{life}. Pavement life with the LCV scenario is less than that calculated for the existing traffic by one year or more ($\Delta_{life} \leq 1$). In this case, maintenance would be required sooner, and the annualized cost would increase.

3. *Significant and positive* Δ_{life}. Pavement life with the LCV scenario is greater than that calculated for the existing traffic by one year or more ($\Delta_{life} \geq 1$). In this case, maintenance would be required later, and the annualized cost would decrease.

Whenever a significant difference arose in expected life with and without the LCV scenario, i.e., whenever $|\Delta_{life}| \geq 1$, the annualized cost of maintaining the pavement was calculated for both scenarios. The difference between both annualized costs is the second measure of impact (MOI), the Δ_{cost}. Figure 2.6 depicts the relationship of both MOIs and their meaning.

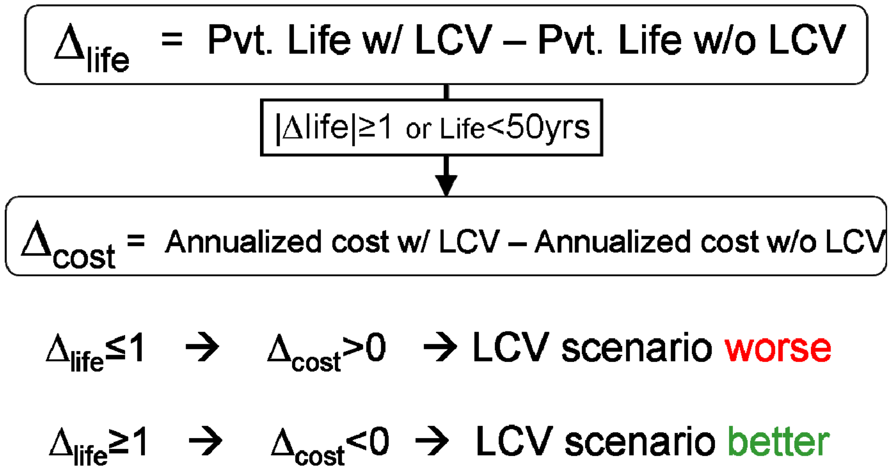


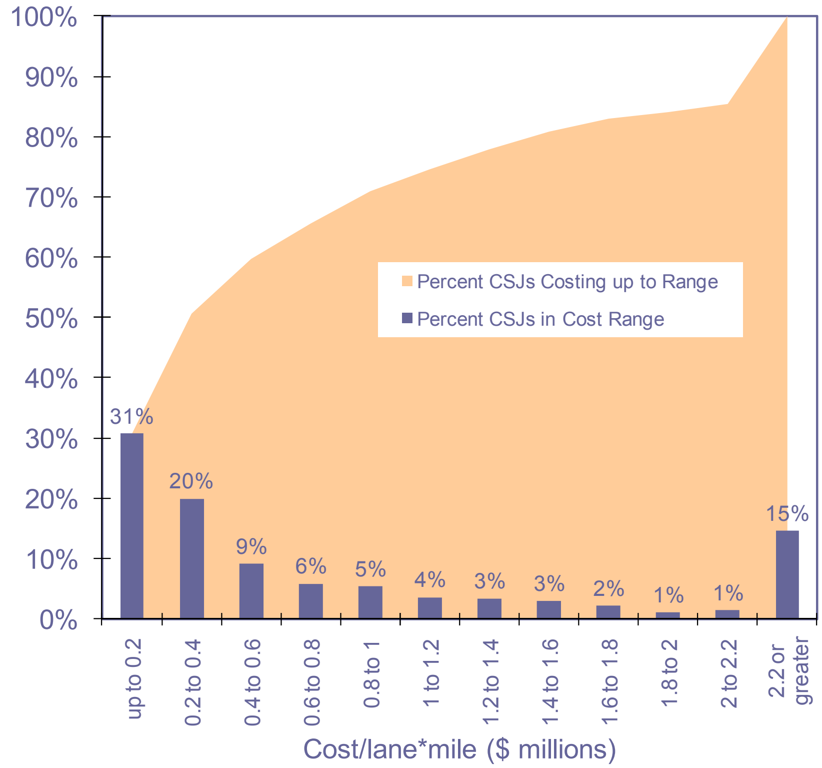
Figure 2.6: Measures of LCV Impacts on Pavements: Δ_{life} and Δ_{cost}

2.2.6 Annualized Cost Calculation

The cost of maintaining the pavements was calculated assuming that the pavement would be overlaid at the end of the estimated life, and that this cycle would be repeated. Overlay costs were obtained from TxDOT Construction Division specifically for this project as well as those from a previous project (Ref. 2-16). In addition, the UTSA team consulted with Construction Division personnel on several issues, including indices to correct the available costs data for inflation or deflation (Ref. 2-17).

Researchers analyzed a database consisting of over 2,000 overlay control section jobs (CSJs) statewide obtained for a previous project (Ref. 2-16). Considerable unit cost variation occurred, as Figure 2.7 shows. The researchers thus decided to calculate three Δ_{cost} MOIs, one for each value of overlay unit cost listed here:

1. $400,000 per lane-mile, the value recommended by the Construction Division as the average unit cost, which is slightly above the median of the available database;
2. $607,000 per lane-mile, the value corresponding to the 60% percentile of the database (see cumulative distribution depicted in Figure 2.7); and
3. $1,219,000 per lane-mile, the value corresponding to the third quartile (75% percentile) of the database (see cumulative distribution depicted in Figure 2.7).



Sources: TxDOT Construction Division 2030 Project Database

Figure 2.7: Overlay Costs

Overlay costs were calculated for each analysis segment by multiplying the number of lane-miles by the cost for each of the three cases in each scenario. Then, the costs were annualized according to the method depicted in Figure 2.8, considering that the overlay expenditure is made at the end of the estimated life (refs. 2-5, 2-14).

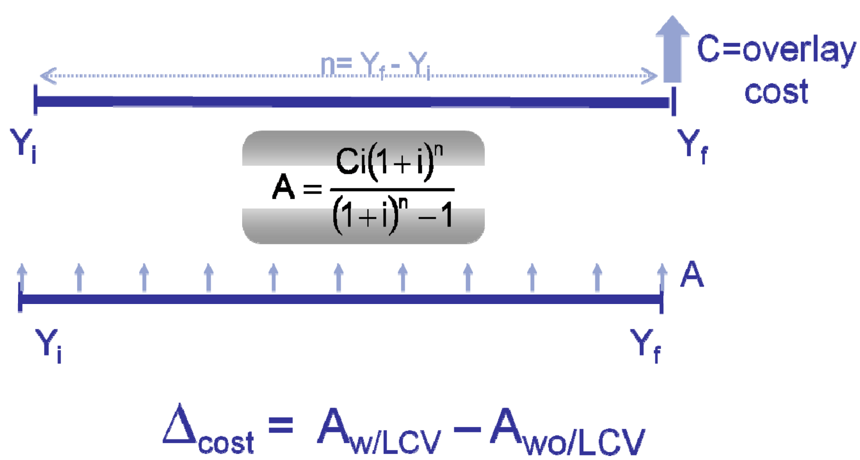


Figure 2.8: Cost Annualization and Measure of Cost Impact

2.3 Data Collection and Treatment

2.3.1 Objective

The objective of the data treatment was to prepare input files for the pavement analysis. The following tasks were necessary:

1. Divide each corridor into segments that have uniform truck traffic, the same pavement and the same subgrade type;
2. Develop load spectra for existing traffic and LCV scenario; and
3. Obtain subgrade and material properties, tire pressures, and detailed axle configurations of all vehicle classes in the load spectra.

2.3.2 Data Overview

For each corridor uniform segment, the UTSA team obtained the following data (summarized in Table 2.1).

- Most recent AADT and truck percentages (2008), from RhiNo and PMIS databases.
- Most recent WIM data reports from all stations located along the four corridors, with assistance from TxDOT'S Transportation Planning and Programming (TP&P). The oldest data are from 2002 and the newest from 2009.
- Pavement cross-sections, retrieved manually from TxDOT construction records (CSJs) with assistance from TxDOT San Antonio District personnel.
- Soil types, from Texas geological maps.
- Typical tire pressure recommended by FHWA and NCHRP (Ref. 2-9).
- Overlay costs, with assistance from TxDOT, already discussed in the "Analysis Methodology" section.

Table 2.1: Data Overview

Data	*Type*	*Main Sources*
Traffic	Average daily truck traffic	PMIS/RHiNo databases
	Vehicle classification Axle load distribution	TP&P / FHWA WIM data reports
	Tire pressures	FHWA's ME-PDG
	Axle configurations	-FHWA's vehicle classes -This project's LCV scenario
Pavement	Pavement cross sections	CSJ database
	Subgrade types	PMIS database
	Pavement layers' properties	Refs. 2-3, 4, 8, 9, 11, 15, 22
	Pavement rehabilitation costs	-TxDOT Expressway- 6/2010 -TxDOT Construction Division -Data from Ref. 2-16

2.3.3 Analysis Segments

Pavement cross-sections, truck demand, and subgrade type vary along the corridors, necessitating segmentation. The team utilized average daily traffic and truck percentages available in the RHiNo and PMIS databases to obtain the ADTT for every reference marker section. Next, these sections were grouped into segments with statistically uniform ADTT.

The team then obtained pavement cross-sections for each of these uniform segments. We are grateful to the San Antonio District for allowing us to access TxDOT's CSJ database through their system. A uniform traffic segment often had more than one pavement cross-section, so this overall segmentation procedure resulted in a total of 152 analysis segments, as summarized in Figure 2.9 for each corridor. For the purposes of this analysis, IH 35 was divided into two sub-corridors, Dallas-San Antonio and San Antonio-Laredo, based on the ADTT grand-average as shown in Figure 2.9.

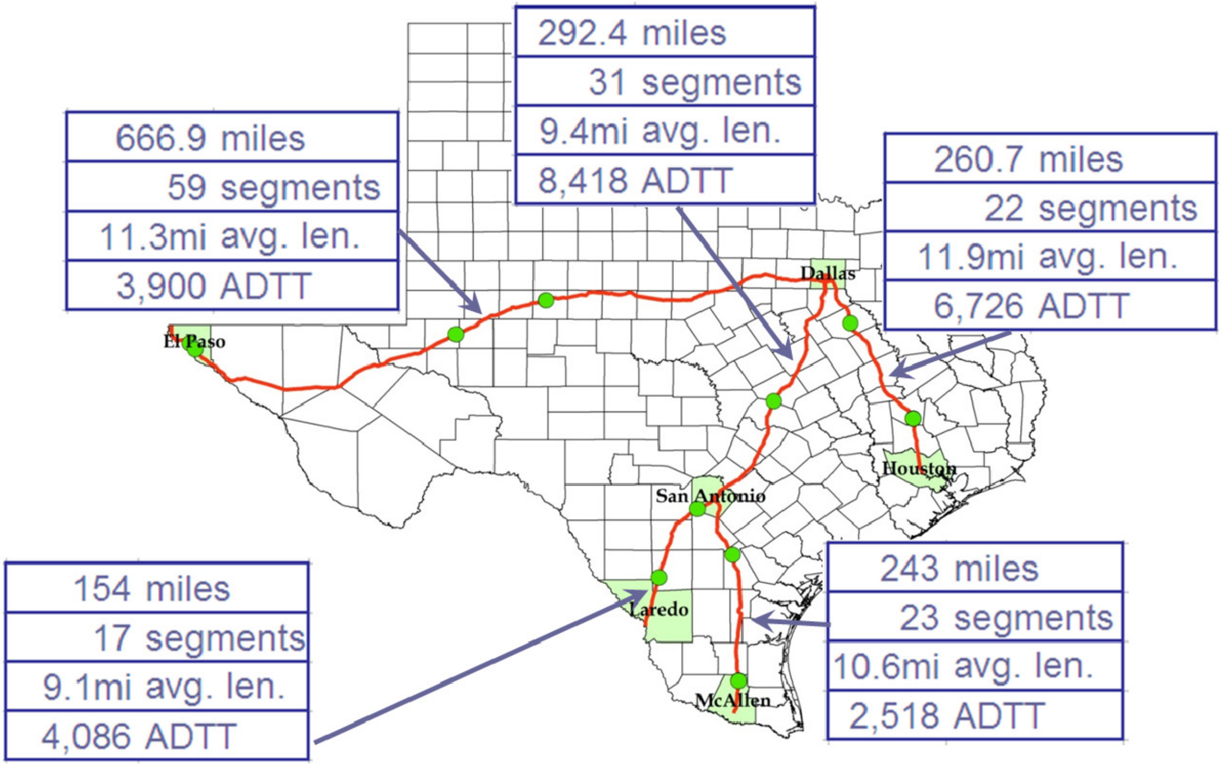


Figure 2.9: Analysis Segments

Some segments were not in the computerized database and had to be looked up in the District Office files. Table 2.2 shows some examples of pavement cross-section data for some IH 35 analysis segments near San Antonio.

Table 2.2: Example of Pavement Cross Section Data (IH 35 near San Antonio)

Segment	From	To	Length (mi)	ADTT	Pavement cross section
3	SL13 & IH 35	SS 422 & IH 35	2	5,805	HMACP 2" Concrete pavement 10" Flex base 8" CH subgrade
4	SS 422 & IH 35	3.27 mi S. of IH 410 & IH 35	7.3	4,211	HMAC 5" Flex Base 8" Stabilized subgrade 8" CH subgrade
5	3.27 mi S. of IH 410 & IH 35	Bexar Co. Line & IH 35	8.2	3,841	HMAC 7" Asphalt Stabilized Base 4" Stabilized subgrade 6" SM subgrade
6	Bexar Co. Line & IH 35	US 57 & IH 35	23	3,197	HMAC 7" Asphalt Stabilized Base 4" Stabilized subgrade 6" SP-SM subgrade

Pavement cross-section source: TxDOT's CSJ Database

2.3.4 Weigh-in-Motion (WIM) Data and Load Spectra Calculations

TxDOT's TP&P provided the locations of all WIM stations along the analysis corridors, as well as all explanations necessary to understand and utilize WIM data reports. Figure 2.10 shows the locations of the WIM stations on the study corridors. They include WIM stations that have already been deactivated, but are relevant for this study.

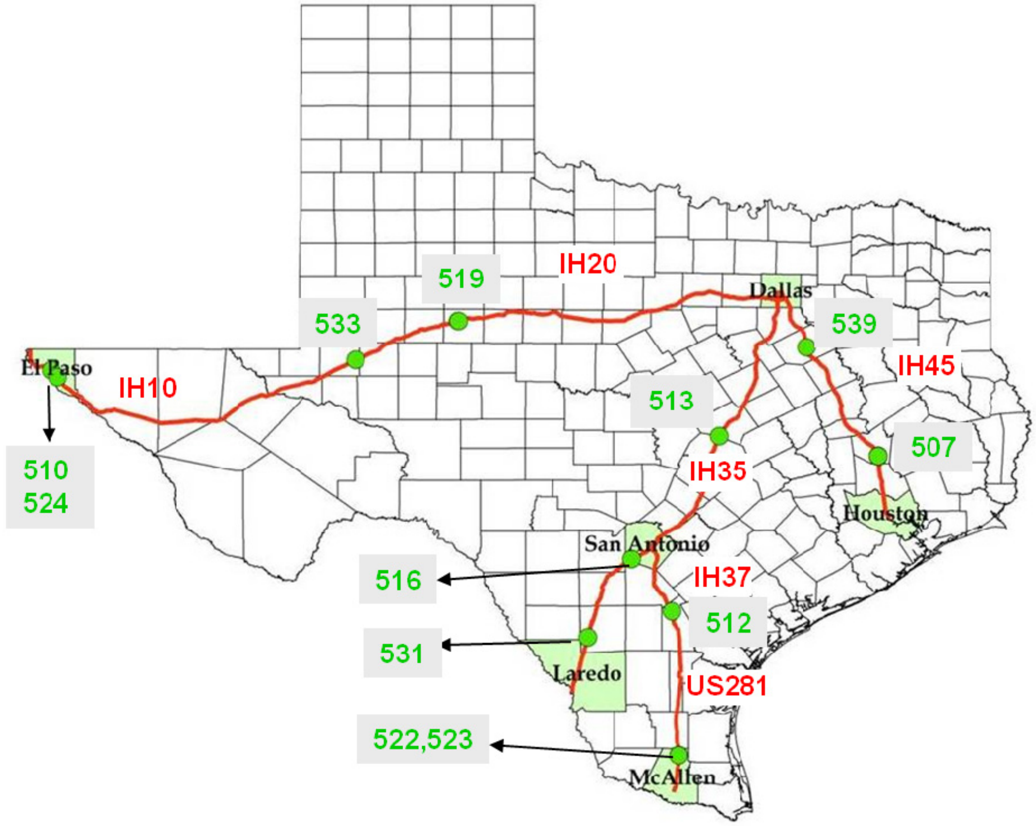


Figure 2.10: WIM Stations on Study Corridors

The research team used data from each WIM station to determine the load spectra for existing traffic as well as LCV scenario for all segments within the station range. The most significant WIM data extrapolation occurred on IH 20, where axle load proportions observed at station 519 were assumed valid for all analysis segments located between the Dallas and Midland county line near Odessa (approximately 350 miles).

For the scenarios without LCVs, load spectra were calculated by applying the axle weight distributions observed in each FHWA vehicle class to the ADTT. For the LCV scenarios, the calculations were more complex. After the team agreed on the basic scenario hypotheses discussed in the previous section, additional assumptions were necessary in order to estimate the new load spectra with LCVs.

The number of double 53 trailers and 97-kip tridem vehicles that would replace 85% of the existing Class 9 tonnage was determined by apportioning 85% of the measured Class 9 "trailer load" (sum of the tandem axle loads) in each WIM station to the LCVs. After that, the load was further apportioned to each type of LCV considered in this analysis (double 53 weighed out, double 53 cubed out, and 97-kip tridem). This method ensured that the amount of cargo was kept constant at all analysis locations, i.e., load was neither "created" nor "eliminated" in the LCV scenario.

The observed load spectra proportions were maintained for the 15% of the total tandem axle weight that remained in Class 9. All 97-kip tridems were assumed weighed out, as their only advantage over a Class 9 truck is the heavier load limit on the tridem axle. Therefore, it seems very unlikely that a company would switch to this new truck if it cubes out. Therefore, following are the categories for all 97-kip tridem load spectra:

- Single (steering) axles: 7 kips
- Tandem axles: 34 kips
- Tridem axles: 56 kips

As for the double 53's, the single (steering) axles were all assumed to weigh 12 kips, and the tandem axles were either cubed out at 19.5 kips each (40%) or weighed out at 31.5 kips each (60%).

Finally, the available WIM data does not distinguish between single and double-wheel single axles; based on the most common configurations, it was assumed that all axles above 14.3 kips had double wheels.

2.3.5 Vehicles Characteristics

FHWA and NCHRP recommend 120 psi as hot tire pressure when other data are lacking (Ref. 2-9). This value was used throughout the analysis.

The available WIM and vehicle classification data are organized into the 13 FHWA vehicle classes. The most commonly found configurations in each class were assumed and are depicted in Table 2.4. The two LCV configurations were discussed in the previous sections and are also summarized in Table 2.4.

Table 2.3: Number of Axles by Vehicle Type

	FHWA Vehicle Class													LCV	
Axle	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13	Double 53'	Tridem 97-kip
Single	2	2	2	2	2	1	1	2	1	1	5	4	3	1	1
Tandem						1		1	2	1		1	2	4	1
Tridem							1			1				0	1

All axles weighing in the 6 to 7 metric tons category (midpoint 14.3 kips) or less were assumed to be steering axles with single tires (two tires per axle). Heavier axles were assumed dual tires (four tires per axle). All tandem axles were assumed to have a total of 8 tires like the typical 18-wheeler tandem axle. All tridem axles were assumed to have a total of 12 tires (Ref. 2-6).

2.3.6 Materials' Characteristics

For concrete pavements, the slab width, modulus of elasticity, Poisson's ratio, unit weight, and coefficient of thermal expansion were set respectively to 12ft, 4,000,000 psi (baseline value is 4,500,000 psi), 0.15, 0.087 lb/in^3, and 5.5*10^{-6}/^{o}F, respectively (Ref. 2-8, 2-9, 2-11, 2-15). The modulus of reaction on top of various sub-bases was obtained from correlations found in the same references.

Subgrade types (CL, CH, etc) were obtained from the PMIS database, and their characteristics were obtained from pavement engineering literature (Ref. 2-2, 2-4, 2-8, 2-9, 2-11, 2-15, 2-18, 2-22). For flexible pavement layers, typical values found in the literature based on the material description were used. Appendix B shows the properties of soil types and pavement materials found in the study corridors.

2.4 Dallas–San Antonio Corridor

2.4.1 Corridor Characteristics

The northern portion of the Dallas–Laredo study corridor starts at the Denton County Line northwest of Dallas and ends at the IH 35–IH 10 intersection in San Antonio. It is 292 miles long and contains 31 segments averaging 9.4 miles in length and 8,418 trucks/day. These segments were determined according to the methodology previously explained.

Table 2.4 and Figure 2.11 depict the summary statistics of the daily number of trucks for all segments in this corridor. The most frequent daily volume is between 8,000 and 9,000 trucks, defining the typical volume for this corridor. Table 2.7 shows the corridor segments in detail.

This corridor has 467 lane-miles of rigid pavement (segments 1 through 11 near Dallas, and segments 29 and 30 near San Antonio). The remaining 1,127 lane-miles are flexible pavement. The predominant soil type is clay: mostly CH, with some segments having CL subgrade.

Table 2.4: Dallas–San Antonio: Truck Traffic Summary

Statistics	**Trucks/Day**
Minimum	3,738
Maximum	14,110
Mean	8,418
First Quartile	6,807
Second Quartile (median)	8,175
Third Quartile	9,818
95% Percentile	12,774
90% Percentile	11,533
85% Percentile	11,374

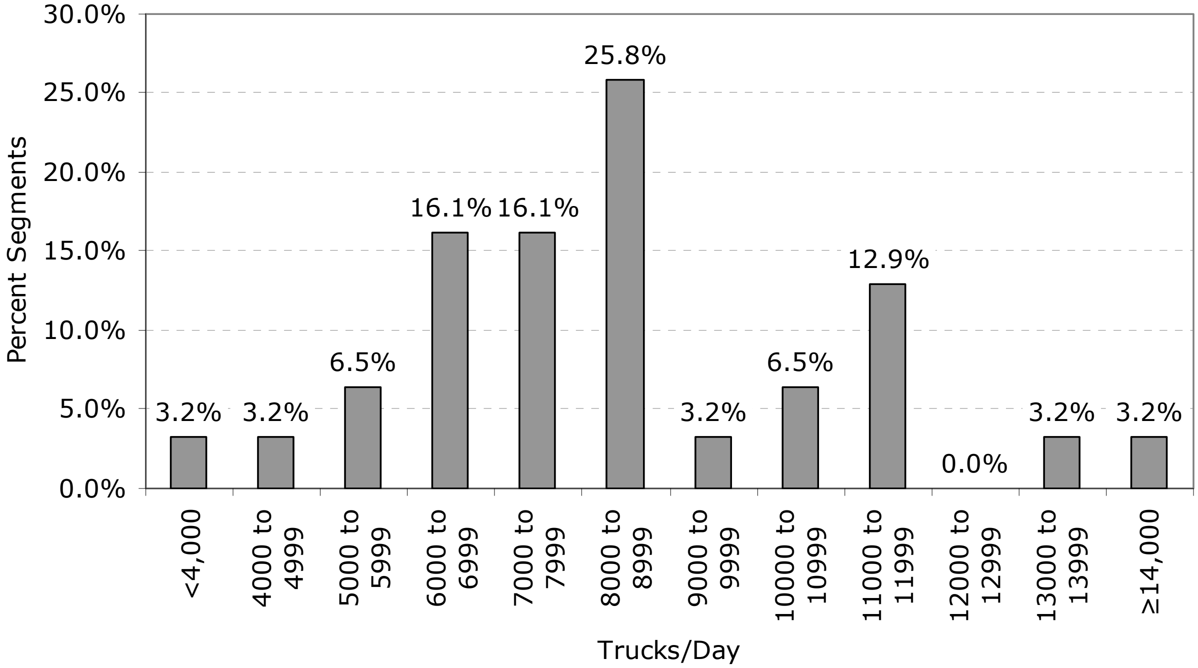


Figure 2.11: Dallas–San Antonio Corridor: Truck Traffic Occurrence

2.4.2 Analysis Direction

Table 2.5 shows a summary of the ESALs reported for the section located between Dallas and San Antonio. The southbound direction was more critical in both cases (Ref. 2-7), and is therefore the analysis direction in this corridor.

Table 2.5: Dallas–San Antonio Corridor: Reported ESALs by Traffic Direction

Direction	ESALs	Station 513		Station 516	
		Rigid	*Flexible*	*Rigid*	*Flexible*
N	*Single*	514	533	525	540
	Tandem	2,017	1,165	2,562	1,474
	Tridem	14	6	6	2
	Total	2,545	1,705	3,093	2,017
S	*Single*	1,431	1,446	565	585
	Tandem	6,363	3,636	3,214	1,847
	Tridem	11	5	6	2
	Total	7,806	5,087	3,785	2,434

2.4.3 Results and Conclusions

Table 2.6 shows the summary results for the Dallas–San Antonio corridor, and Table 2.7, the results by analysis segment. The LCV scenario had no impact on rigid pavements but had some negative impact on the flexible pavements, decreasing flexible pavement life, on average, by half a year, and increasing pavement overlay costs by the amounts shown in the last three columns of Table 2.6. For overlay unit costs of $0.4 million per lane-mile, the LCV scenario increased costs by \$0.94 million/year. For overlay unit costs of \$0.6 million per lane-mile, the cost increase was \$1.43 million/year, and for overlay unit costs of \$1.2 million per lane-mile, the cost increase was \$2.87 million/year.

Table 2.6: Dallas–San Antonio: Summary Results

	Lane-miles	***Average Δ_{life}***	***Overlay Δ_{cost} (millions/year)***		
			\$0.4m	*\$0.6m*	*\$1.2m*
Rigid	467	0	\$0	\$0	\$0
Flexible	1,127	-0.5	\$0.94	\$1.43	\$2.87
Total	1,594		\$0.94	\$1.43	\$2.87

Table 2.7: Dallas-San Antonio: Results by Analysis Segment

Analysis Segment	Trucks/ day	Lane Miles	Pavement Type	Pavement Life(yrs)		Δlife (yrs)	Overlay Δcost (millions/yr)		
				Existing	LCV		$0.4m	$0.6m	$1.2m
1	6233	66.4	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
2	4484	18	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
3	6760	50.2	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
4	5996	40.2	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
5	3738	32.6	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
6	13917	3	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
7	10698	27	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
8	8683	28	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
9	7895	18.8	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
10	6233	25.6	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
11	5364	103.6	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
12	8256	64	Flexible	12.5	11.4	-1.1	$0.199	$0.303	$0.608
13	8700	67	Flexible	8.0	7.7	-0.3	$0.119	$0.181	$0.363
14	11533	36	Flexible	8.5	8.2	-0.3	$0.068	$0.103	$0.207
15	8426	47	Flexible	6.9	6.7	-0.2	$0.078	$0.118	$0.237
16	8022	35.4	Flexible	8.0	8.0	0.0	$0.000	$0.000	$0.000
17	7955	90	Flexible	7.5	7.2	-0.2	$0.163	$0.247	$0.496
18	8297	84.8	Flexible	12.0	10.9	-1.1	$0.278	$0.422	$0.847
19	11245	40.2	Flexible	10.3	10.4	0.1	($0.014)	($0.021)	($0.042)
20	14110	7.8	Flexible	10.3	10.4	0.1	($0.003)	($0.004)	($0.008)
21	8175	78	Flexible	12.2	11.1	-1.1	$0.255	$0.386	$0.777
22	10403	20.8	Flexible	9.6	8.7	-0.9	$0.088	$0.133	$0.268
23	8312	63.4	Flexible	12.0	10.9	-1.1	$0.210	$0.319	$0.641
24	7042	29.4	Flexible	7.7	7.1	-0.6	$0.130	$0.197	$0.396
25	6421	93	Flexible	8.3	8.4	0.1	($0.070)	($0.106)	($0.214)
26	7233	53.4	Flexible	7.3	7.5	0.2	($0.084)	($0.127)	($0.256)
27	6854	61.6	Flexible	7.8	8.0	0.2	($0.071)	($0.108)	($0.218)
28	7620	93.6	Flexible	7.5	6.6	-0.9	$0.683	$1.037	$2.084
29	9233	13.2	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
30	11503	41.6	Rigid	>60	>60	0.0	$0.000	$0.000	$0.000
31	11631	87	Flexible	8.6	8.8	0.1	($0.065)	($0.099)	($0.199)
TOTAL		1,594					$1.964	$2.980	$5.989

Note: red font indicates a negative amount, or a decrease in annual costs.

2.5 San Antonio–Laredo Corridor

2.5.1 Corridor Characteristics

This southern portion of the IH 35 Dallas–Laredo corridor starts at the IH 10–IH 35 interchange in San Antonio, and ends 1.2 miles south of the SL13–IH 35 junction. It is 154 miles long and contains 17 analysis segments averaging 9.1 miles in length and 4,086 trucks/day. These segments were determined according to the methodology previously explained.

Table 2.8 and Figure 2.12 depict the summary statistics of the daily number of trucks for all segments in this corridor. The most frequent daily volume is between 1,500 and 2,500 trucks, defining the typical volume for this corridor.

This corridor has 29 lane-miles of rigid pavement (segments 1, 2, and 3 near San Antonio). The remaining 614 lane-miles are flexible pavement; 167 of these have a thick (14 in.) hot-mix asphalt concrete top layer (segments near Laredo). Clay subgrade predominates (5 segments are CH and 3 are CL). One segment is clayey gravel (GC), four are silt (ML), and four are sand (SC, SM, and SP-SM).

Table 2.8: San Antonio–Laredo Corridor: Truck Traffic Summary

Statistics	Trucks/Day
Minimum	1,830
Maximum	11,795
Mean	4,086
First Quartile	2,134
Second Quartile (median)	3,197
Third Quartile	5,201
95% Percentile	9,266
90% Percentile	7,103
85% Percentile	5,971

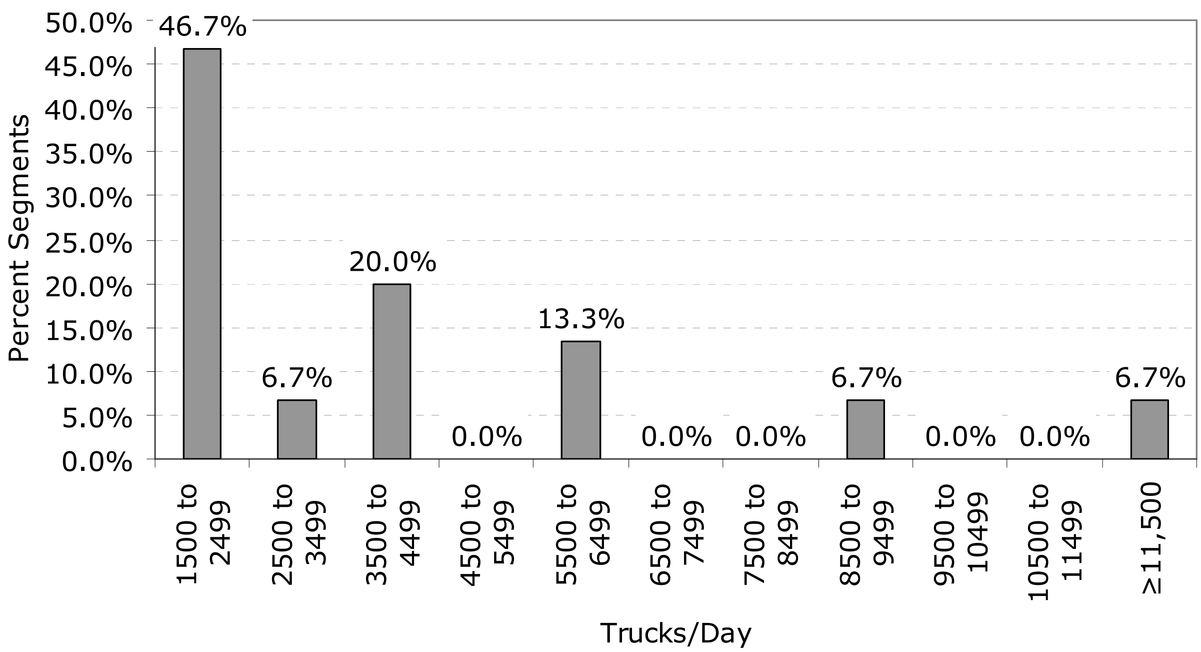


Figure 2.12: San Antonio–Laredo Corridor: Truck Traffic Occurrence

2.5.2 Analysis Direction

Table 2.9 shows ESALs reported for the two WIM stations located between San Antonio and Laredo 516 (segments 1 and 4) and 531 (remaining segments). The southbound direction is more critical in terms of ESALs at both WIM stations.

Table 2.9: San Antonio–Laredo Corridor: Reported ESALs by Traffic Direction

Direction	ESALs	Station 531		Station 516	
		Rigid	Flexible	Rigid	Flexible
N	*Single*	513	538	525	540
	Tandem	3,312	1,902	2,562	1,474
	Tridem	3	1	6	2
	Total	3,827	2,441	3,093	2,017
S	*Single*	487	508	565	585
	Tandem	3,439	1,978	3,214	1,847
	Tridem	12	5	6	2
	Total	3,938	2,491	3,785	2,434

2.5.3 Results and Conclusions

Table 2.10 shows the summary results for the San Antonio–Laredo corridor, and Table 2.11, the results by analysis segment. Both tables indicate the flexible pavements with the 14 in. top layer to emphasize a difference in results.

The LCV scenario had no impact on rigid pavements, increased the life of the "regular" flexible pavements, on the average, by 2.6 years, and decreased the life of the thick-surfaced pavements near Laredo, on the average, by 3.6 years. Nevertheless, this difference in flexible pavement results was due to a combination of surface thickness and a different load spectra. Note that, although the absolute value of Δ_{life} is larger for the thicker pavement, its average life is also greater (see Table 2.11): the 14 in. HMAC pavement life averaged 25 years without LCVs and 22 years with the LCV scenario, and respectively 14 and 16 years for the 6 in. to 8 in. HMAC pavements in this corridor. The thick HMAC pavement failed due to rutting, while the thinner ones failed due to alligator cracking in the top layer.

For overlay unit costs of $0.4 million per lane-mile, the LCV scenario decreased costs by $3.482 million/year. For unit costs of $0.6 million per lane-mile, the cost decrease was $5.284 million/year, and for overlay unit costs of $1.2 million per lane-mile, the cost decrease was $10.62 million/year.

Table 2.10: San Antonio–Laredo: Summary Results

	Lane-miles	*Average Δ_{life}*	*Overlay Δ_{cost} (millions/year)*		
			$0.4m	*$0.6m*	*$1.2m*
Overlaid CRCP	29	0	$0.000	$0.000	$0.000
Flexible, 6-8" HMAC	447	2.6	($3.890)	($5.903)	($11.864)
Flexible 14" HMAC	159	-3.6	$0.408	$0.619	$1.243
Total	635		($3.482)	($5.284)	($10.620)

Note: red font indicates a negative amount, or a decrease in annual costs

Table 2.11: San Antonio–Laredo: Results by Analysis Segment

Analysis Segment	**Trucks/ day**	**Lane Miles**	**Pavement Type**	**Pavement Life(yrs)**		**Δlife (yrs)**	**Overlay Δcost (millions/yr)**		
				Existing	**LCV**		**$0.4m**	**$0.6m**	**$1.2m**
1	11,795	6	Overlaid CRCP	>60	>60	0.0	$0.000	$0.000	$0.000
2	8,634	10.8		>60	>60	0.0	$0.000	$0.000	$0.000
3	5,805	12		>60	>60	0.0	$0.000	$0.000	$0.000
4	4,211	32.2	Flexible: 6" to 8" HMAC	9	9	0.0	$0.000	$0.000	$0.000
5	3,841	32.8		21	21	0.0	$0.000	$0.000	$0.000
6	3,197	92		25	25	0.0	$0.000	$0.000	$0.000
7	2,282	38		8	12	4.0	($0.637)	($0.966	($1.942)
8	2,172	68.4		6	16	10.0	($2.866)	($4.349	($8.741)
9	1,830	112.4		19	23	4.0	($0.387)	($0.587	($1.180)
10	1,871	71.6		9	9	0.0	$0.000	$0.000	$0.000
11	2,003	63.6		26	22	-4.0	$0.163	$0.247	$0.497
12	2,022	32.8	Flexible: 14" HMAC 6" base	26	22	-4.0	$0.084	$0.128	$0.256
13	2,134	30		25	21	-4.0	$0.085	$0.128	$0.258
14	3,630	7.8		14	12	-2.0	$0.037	$0.056	$0.112
15	6,082	2.4		20	17	-3.0	$0.008	$0.012	$0.025
16	2,758	15		44	39	-5.0	$0.013	$0.020	$0.039
17	5,201	7.5		23	20	-3.0	$0.018	$0.028	$0.056
TOTAL		635.3					($3.482)	($5.284)	($10.620)

Note: red font indicates a negative amount, or a decrease in annual overlay costs

2.6 Dallas–Houston Corridor

2.6.1 Corridor Characteristics

This IH 45 corridor starts at the IH 30–IH 45 interchange and ends at the Harris county line. It is 260.7 miles long and contains 22 analysis segments averaging 11.9 miles in length and 6,726 trucks/day. These segments were determined according to the methodology previously explained.

Table 2.12 and Figure 2.13 depict the summary statistics of the daily number of trucks for all segments in this corridor. The most frequent daily volume is between 5,000 and 7,000 trucks, defining the typical volume for this corridor. This corridor is entirely paved in concrete, overlaid in some segments.

Table 2.12: Dallas–Houston Corridor: Truck Traffic Summary

Statistics	Trucks/Day
Minimum	4,491
Maximum	13,587
Mean	6,726
First Quartile	5,607
Second Quartile (median)	6,206
Third Quartile	7,436
95% Percentile	9,173
90% Percentile	8,771
85% Percentile	8,110

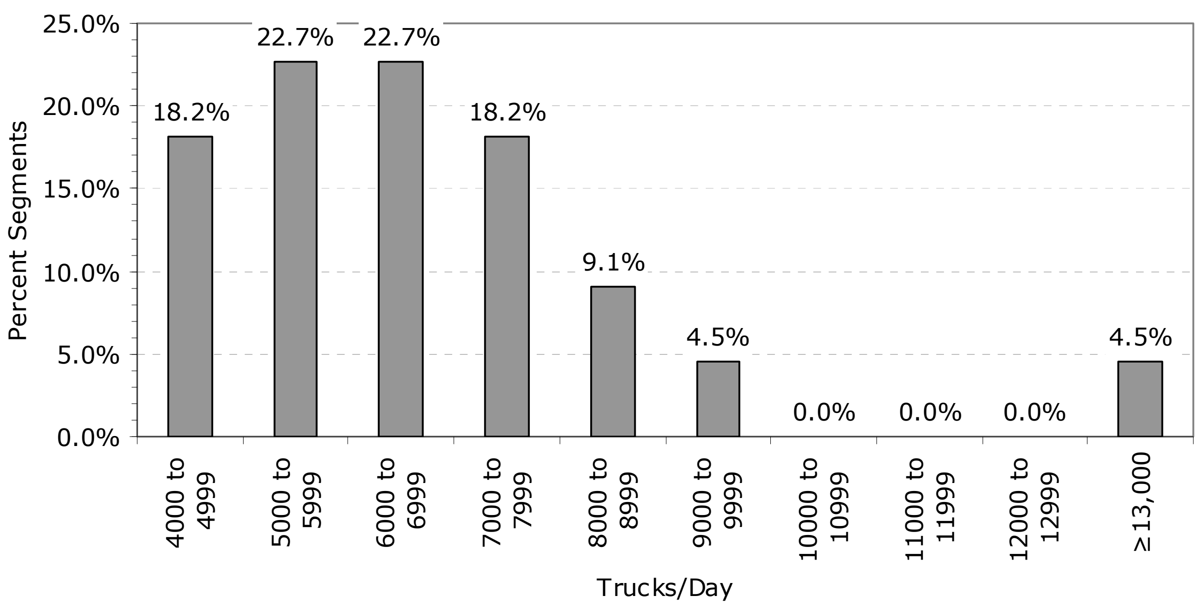


Figure 2.13: Dallas–Houston Corridor: Truck Traffic Occurrence

2.6.2 Analysis Direction

Table 2.13 shows a summary of the reported ESALs for rigid as well as flexible pavements; the northbound direction has more ESALs in both WIM stations. The analysis direction for all segments is the northbound, the most critical in terms of ESALs reported for WIM stations number 507 and 539.

Table 2.13: Dallas–Houston Corridor: Reported ESALs by Traffic Direction

Direction	ESALs	539		507	
		Rigid	*Flexible*	*Rigid*	*Flexible*
N	*Single*	2,181	2,236	1,186	1,090
	Tandem	11,838	6,732	2,948	1,675
	Tridem	74	31	17	7
	Total	14,093	8,999	4,150	2,772
S	*Single*	939	971	689	704
	Tandem	5,307	3,033	2,648	1,515
	Tridem	31	13	10	4
	Total	6,277	4,017	3,347	2,223

2.6.3 Results and Conclusions

The LCV scenario had no impact on rigid pavements, primarily because the Texas-required concrete slab thicknesses usually lead to stress ratios less than 0.45, which results in unlimited load applications. Cost differential (Δ_{cost}) is zero for the entire corridor.

2.7 Dallas–El Paso Corridor

2.7.1 Corridor Characteristics

This section is the longest corridor analyzed by this project, with 667 miles: 480 on IH 20 and 187 on IH 10. The corridor starts at the US 175–IH 20 intersection and ends at the El Paso county line. It has 59 analysis segments averaging 11.3 miles in length and 3,900 trucks/day. These segments were determined according to the methodology previously explained.

This corridor has 1,534 lane-miles of rigid pavement, comprising the following segments: 1 through 14 (segment 1 is in Dallas); segment 32; and segments 41 through 57. The 1,400 lane-miles are flexible pavement. The researchers could not obtain detailed cross-section data for three segments totaling 42 miles in length (segments 18, 24, and 35). The pavement type is known from RhINo and PMIS databases. The most frequent subgrade type is sand (28 segments), followed by clay (21 segments). Table 2.14 shows a summary of the subgrade types found along this IH 20/IH 10 east-west corridor.

Table 2.14: Soil Types on the Dallas–El Paso Corridor

Soil Type	Number of Analysis Segments	Soil Type	Number of Analysis Segments	Soil Type	Number of Analysis Segments
CH	5	GC and rock	1	Rock	3
CL	16	GM-GC	1	SM	27
GC	3	ML-CL	2	SM-SC	1

Table 2.15 and Figure 2.14 depict the summary statistics of the daily number of trucks for all segments in this corridor. The most frequent daily volume is between 3,500 and 4,000 trucks, defining the typical volume for this corridor.

Table 2.15: Dallas–El Paso Corridor: Truck Traffic Summary

Statistics	Trucks/Day
Minimum	1,142
Maximum	8,181
Mean	3,900
First Quartile	2,941
Second Quartile (median)	3,670
Third Quartile	4,283
95% Percentile	7,274
90% Percentile	6,370
85% Percentile	5,410

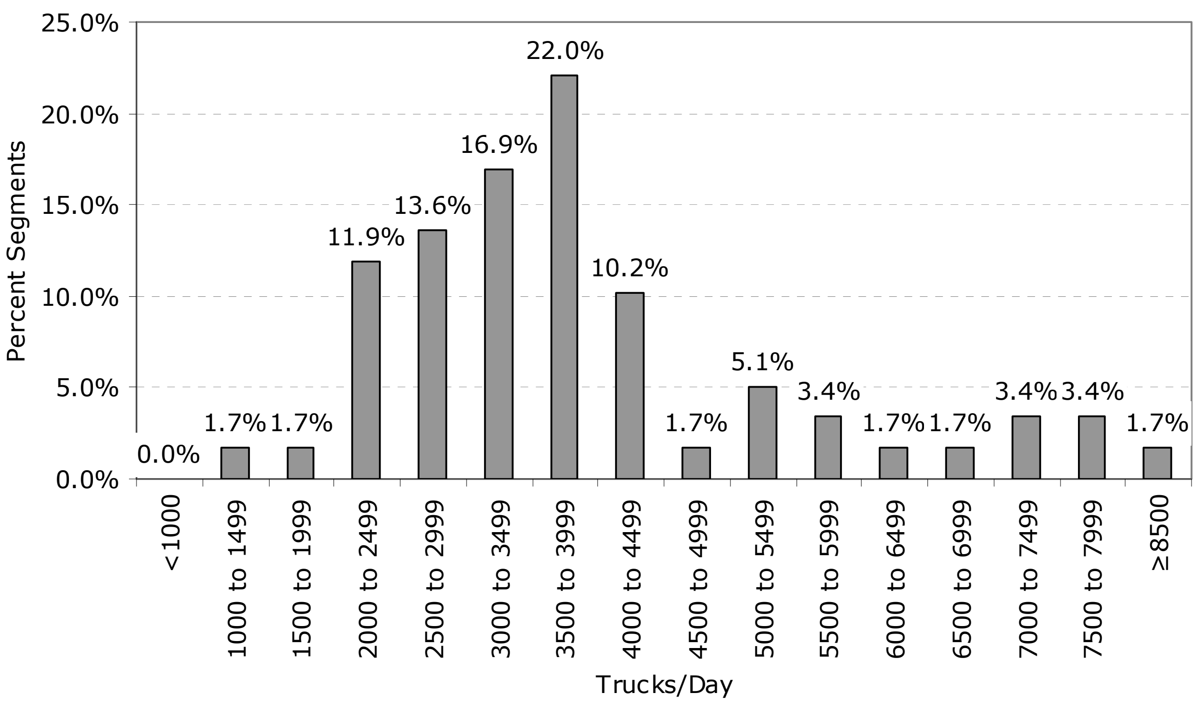


Figure 2.14: Dallas–El Paso Corridor: Truck Traffic Occurrence

2.7.2 Analysis Direction

Table 2.16 shows a summary of the reported ESALs for rigid as well as flexible pavements in the corridor's three WIM stations for which recent axle weight data are available (519, 524, and 533). The analysis direction changes as the corridor approaches El Paso. For stations 519 and 533, the westbound direction is more critical, while for station 524 (in the El Paso area), the eastbound direction is the critical one (see Figure 2.10 for the WIM station locations).

Table 2.16: Dallas–El Paso Corridor: Reported ESALs by Traffic Direction

Direction	ESALs	519		533		524	
		Rigid	*Flexible*	*Rigid*	*Flexible*	*Rigid*	*Flexible*
E	*Single*	1,137	1,068	863	860	1,101	1,110
	Tandem	3,554	2,045	3,110	1,785	5,141	2,934
	Tridem	9	4	53	22	32	13
	Total	4,700	3,116	4,026	2,667	6,273	4,057
W	*Single*	1,711	1,605	1,454	1,378	1,141	1,120
	Tandem	8,240	4,401	3,193	1,828	4,368	2,501
	Tridem	15	6	51	21	26	11
	Total	9,966	6,012	4,699	3,228	5,536	3,631

2.7.3 Results and Conclusions

Table 2.17 shows the summary results for the Dallas–El Paso corridor, and Table 2.18, detailed results by analysis segment. Table 2.18 takes two pages, given the length of this corridor.

The LCV scenario had no impact on rigid pavements and either increased or did not change the life of flexible pavements. On the average, the LCV scenario increased flexible pavement life by 5.7 years. For overlay unit costs of $0.4 million per lane-mile, the LCV scenario decreased costs by $17.120 million/year. For unit costs of $0.6 million per lane-mile, the cost decrease was $25.980 million/year, and for overlay unit costs of $1.2 million per lane-mile, the cost decrease was $52.217 million/year.

Nevertheless, the decrease in flexible pavement annualized overlay costs results was due primarily to the fact that the LCV scenario removed some of the overweight axles from the load spectra,. WIM station 519, located approximately at the corridor mid-point, is the only available source of axle weight data for half of this corridor and it has a significant number of overweight trucks. These data had to be apportioned to the entire Dallas–Midland portion of this corridor, but it may be possible that the overweight trucks observed at that station are due to a local need for overweight permits. In short, results of this study need to be interpreted with care due to the required data extrapolations. As already mentioned, it is impossible to forecast in detail what the vehicle mix will be after a possible approval of LCVs in Texas, and results reflect possible scenarios considered realistic by the PMC.

Table 2.17: Dallas–El Paso Corridor: Summary Results

	Lane-miles	*Average Δ_{life}*	*Overlay Δ_{cost} (millions/year)*		
			\$0.4m	*\$0.6m*	*\$1.2m*
Rigid	1,534	0	\$0.000	\$0.000	\$0.000
Flexible	1,400	5.4	(\$17.120)	(\$25.980)	(\$52.217)
Total	2,934		(\$17.120)	(\$25.980)	(\$52.217)

Table 2.18: Dallas–El Paso Corridor: Results by Analysis Segment

Analysis Segment	Trucks/ day	Lane Miles	Pavement Type	Pavement Life(yrs)		Δlife (yrs)	Overlay Δcost (millions/yr)		
				Existing	LCV		\$0.4m	\$0.6m	\$1.2m
1	116.8	7647	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
2	78.4	7233	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
3	47.2	7978	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
4	53	8181	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
5	49.6	3065	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
6	46.8	2250	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
7	26	1142	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
8	39	6533	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
9	29.2	5249	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
10	21.2	4012	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
11	64	3505	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
12	92	2977	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
13	56	2988	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
14	24.8	3004	Rigid	>30	>30	0	\$0.000	\$0.000	\$0.000
15	78	3015	Flexible	6	9	3	(\$1.757)	(\$2.667)	(\$5.360)
16	122	2489	Flexible	7	11	4	(\$2.559)	(\$3.883)	(\$7.804)
17	39	2678	Flexible	6	9	3	(\$0.879)	(\$1.333)	(\$2.680)
18	36	4349	Flexible						
19	57.2	4327	Flexible	8	13	5	(\$1.104)	(\$1.676)	(\$3.368)
20	64	3981	Flexible	24	38	14	(\$0.338)	(\$0.512)	(\$1.030)
21	26	4224	Flexible	24	37	13	(\$0.131)	(\$0.199)	(\$0.401)
22	38	3006	Flexible	34	52	18	(\$0.113)	(\$0.172)	(\$0.346)
23	88	2800	Flexible	27	40	13	(\$0.352)	(\$0.535)	(\$1.075)
24	40	2753	Flexible						
25	77	3299	Flexible	6	10	4	(\$2.079)	(\$3.155)	(\$6.342)
26	57	3298	Flexible	7	10	3	(\$0.988)	(\$1.499)	(\$3.012)
27	54	3037	Flexible	12	18	6	(\$0.589)	(\$0.894)	(\$1.797)

Analysis Segment	Trucks/ day	Lane Miles	Pavement Type	Pavement Life(yrs)		Δlife (yrs)	Overlay Δcost (millions/yr)		
				Existing	LCV		$0.4m	$0.6m	$1.2m
28	23	3078	Flexible	7	9	2	($0.296)	($0.449)	($0.902)
29	30.8	3797	Flexible	7	9	2	($0.396)	($0.601)	($1.207)
30	17.68	5022	Flexible	13	19	6	($0.168)	($0.254)	($0.511)
31	40	3997	Flexible	16	24	8	($0.317)	($0.481)	($0.966)
32	22.8	3958	Rigid	>30	>30	0	$0.000	$0.000	$0.000
33	72.4	4239	Flexible	8	11	3	($0.994)	($1.509)	($3.033)
34	38	4138	Flexible	9	13	4	($0.520)	($0.790)	($1.587)
35	93.2	2348	Flexible						
36	16	2260	Flexible	9	13	4	($0.219)	($0.332)	($0.668)
37	52.8	2904	Flexible	7	10	3	($0.915)	($1.388)	($2.790)
38	60	2343	Flexible	9	13	4	($0.822)	($1.247)	($2.506)
39	102.8	2376	Flexible	9	12	3	($1.146)	($1.739)	($3.495)
40	32	2322	Flexible	9	13	4	($0.438)	($0.665)	($1.337)
41	32.48	3410	Rigid	>30	>30	0	$0.000	$0.000	$0.000
42	72.4	3670	Rigid	>30	>30	0	$0.000	$0.000	$0.000
43	81.6	3722	Rigid	>30	>30	0	$0.000	$0.000	$0.000
44	18.8	3792	Rigid	>30	>30	0	$0.000	$0.000	$0.000
45	12	3762	Rigid	>30	>30	0	$0.000	$0.000	$0.000
46	134.4	3588	Rigid	>30	>30	0	$0.000	$0.000	$0.000
47	43.2	3727	Rigid	>30	>30	0	$0.000	$0.000	$0.000
48	103.6	3766	Rigid	>30	>30	0	$0.000	$0.000	$0.000
49	27.2	2792	Rigid	>30	>30	0	$0.000	$0.000	$0.000
50	21.72	1729	Rigid	>30	>30	0	$0.000	$0.000	$0.000
51	27.2	2830	Rigid	>30	>30	0	$0.000	$0.000	$0.000
52	19.6	4501	Rigid	>30	>30	0	$0.000	$0.000	$0.000
53	15.68	5617	Rigid	>30	>30	0	$0.000	$0.000	$0.000
54	16.8	7127	Rigid	>30	>30	0	$0.000	$0.000	$0.000
55	35.4	5525	Rigid	>30	>30	0	$0.000	$0.000	$0.000
56	57.8	6329	Rigid	>30	>30	0	$0.000	$0.000	$0.000
57	46.8	5361	Rigid	>30	>30	0	$0.000	$0.000	$0.000
58	8.8	3982	Flexible	7	7	0	$0.000	$0.000	$0.000
59	36.4	3078	Flexible	9	9	0	$0.000	$0.000	$0.000
TOTAL	2933.6						($17.120)	($25.980)	($52.217)

Note: red font indicates a negative amount, or a decrease in annual costs.

2.8 San Antonio–McAllen Corridor

2.8.1 Corridor Characteristics

This corridor comprises a section of IH 37 from its IH 35 junction in San Antonio, followed by US 281, starting at its IH 37 junction near Pleasanton. It ends near the Pharr

International Bridge near McAllen. This 243-mile-long corridor contains 23 analysis segments averaging 10.6 miles in length and 2,518 trucks/day. These segments were determined according to the methodology previously explained. This corridor has 65 lane-miles of rigid pavement, comprising the first four segments near San Antonio. The remaining 961 lane-miles are flexible pavement.

Table 2.19 and Figure 2.15 depict the summary statistics of the daily number of trucks for all segments in this corridor. The most frequent daily volume is between 1,500 and 3,000 trucks, defining the typical volume for this corridor.

Table 2.19: San Antonio–McAllen Corridor: Truck Traffic Summary

Statistics	Trucks/Day
Minimum	228
Maximum	7,960
Mean	2,518
First Quartile	1,629
Second Quartile (median)	2,175
Third Quartile	2,922
95% Percentile	5,386
90% Percentile	4,116
85% Percentile	3,243

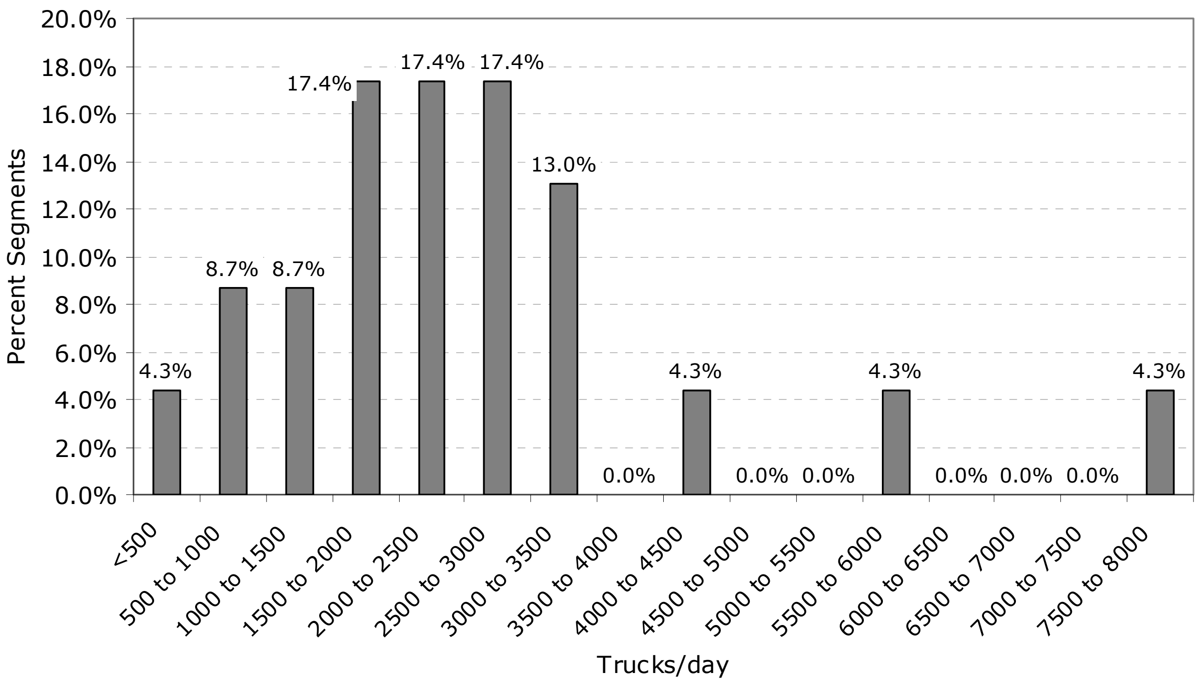


Figure 2.15: San Antonio–McAllen Corridor: Truck Traffic Occurrence

2.8.2 Analysis Direction

Table 2.20 shows a summary of the reported ESALs for rigid as well as flexible pavements; the southbound direction has more ESALs in both WIM stations. The analysis direction for all segments is the southbound, the most critical in terms of ESALs reported for both WIM stations in this corridor.

Table 2.20: San Antonio–McAllen Corridor: Reported ESALs by Traffic Direction

Direction	ESALs	Station		Station	
		Rigid	*Flexible*	Rigid	Flexible
N	*Single*	136	144	333	339
	Tandem	950	544	2,170	1,238
	Tridem	0	0	12	5
	Total	1,086	687	2,515	1,583
S	*Single*	319	331	344	356
	Tandem	950	544	2,157	1,230
	Tridem	0	0	39	16
	Total	1,268	874	2,540	1,602

2.8.3 Results and Conclusions

Table 2.21 shows the summary results for the Dallas–El Paso corridor, and Table 2.22, detailed results by analysis segment. The LCV scenario had no impact on rigid pavements. It also had no impact on the life of flexible pavements for all but three segments. On the average, the LCV scenario decreased flexible pavement life by 0.2 years. For overlay unit costs of $0.4 million per lane-mile, the LCV scenario increased costs by $0.144 million/year. For unit costs of $0.6 million per lane-mile, the cost increase was $0.219 million/year, and for overlay unit costs of $1.2 million per lane-mile, the cost increase was $0.440 million/year.

Table 2.21: San Antonio–McAllen: Summary Results

	Lane-miles	***Average Δ_{life}***	***Overlay Δ_{cost} (millions/year)***		
			$0.4m	*$0.6m*	*$1.2m*
CRCP	65	0	$0.000	$0.000	$0.000
Flexible	961	-0.2	$0.144	$0.219	$0.440
Total	1,026		$0.144	$0.219	$0.440

Table 2.22: San Antonio–McAllen: Results by Analysis Segment

Analysis Segment	Trucks/ day	Lane Miles	Pavement Type	Pavement Life(yrs)		Δlife	Overlay Δcost (millions/yr)		
				Existing	LCV	(yrs)	$0.4m	$0.6m	$1.2m
1	2359	20.8	CRCP	>30	>30	0.0	$0.000	$0.000	$0.000
2	2104	16.8	CRCP	>30	>30	0.0	$0.000	$0.000	$0.000
3	1736	12.8	CRCP	>30	>30	0.0	$0.000	$0.000	$0.000
4	1522	14.4	CRCP	>30	>30	0.0	$0.000	$0.000	$0.000
5	3088	57.2	Flexible	7.2	6.8	0.0	$0.000	$0.000	$0.000
6	2764	30.8	Flexible	8.0	7.6	0.0	$0.000	$0.000	$0.000
7	2507	146.8	Flexible	8.8	8.3	0.0	$0.000	$0.000	$0.000
8	228	17.6	Flexible	>30	>30	0.0	$0.000	$0.000	$0.000
9	974	54.6	Flexible	41.0	39.0	-2.0	$0.021	$0.031	$0.063
10	866	77.6	Flexible	6.7	6.3	0.0	$0.000	$0.000	$0.000
11	1292	78.0	Flexible	17.2	16.2	-0.9	$0.101	$0.153	$0.308
12	1205	19.2	Flexible	18.4	17.4	-1.0	$0.023	$0.035	$0.070
13	2175	12.0	Flexible	7.2	6.8	0.0	$0.000	$0.000	$0.000
14	1864	108.4	Flexible	>30	>30	0.0	$0.000	$0.000	$0.000
15	2028	104.4	Flexible	9.4	10.1	0.0	$0.000	$0.000	$0.000
16	1887	96.0	Flexible	8.1	8.7	0.0	$0.000	$0.000	$0.000
17	2512	38.8	Flexible	11.1	12.0	0.0	$0.000	$0.000	$0.000
18	3080	20.8	Flexible	6.9	7.4	0.0	$0.000	$0.000	$0.000
19	4318	24.6	Flexible	9.1	9.8	0.0	$0.000	$0.000	$0.000
20	5505	12.5	Flexible	8.6	9.3	0.0	$0.000	$0.000	$0.000
21	7960	18.7	Flexible	7.8	8.5	0.0	$0.000	$0.000	$0.000
22	3310	34.2	Flexible	7.9	8.6	0.0	$0.000	$0.000	$0.000
23	2632	9.0	Flexible	7.3	7.9	0.0	$0.000	$0.000	$0.000
TOTAL		1026.0					$0.144	$0.219	$0.440

2.9 Summary of Findings, Recommendations, and Discussion

2.9.1 Summary of Findings

The LCV scenario resulted in an overall decrease in overlay costs of $17.40, $26.40, and $53.07 million/year, respectively for overlay unit costs of $0.4, $0.6, and $1.2 million per lane-mile. The summary results are shown in Table 2.23.

Table 2.23: Summary Results

	Length (mi)	Overlay Δ_{cost} ($millions/year)		
		$0.4m	*$0.6m*	*$1.2m*
Dallas–El Paso	667	$(15.00)	$(22.77)	$(45.76)
Dallas–Laredo	446	$(2.54)	$(3.85)	$(7.75)
Dallas–Houston	261	$ -	$ -	$ -
San Antonio–McAllen	243	$0.14	$0.22	$0.44
TOTAL	1,617	$(17.40)	$(26.40)	$(53.07)

Note: red font indicates a negative amount, or a decrease in annual costs

Table 2.24 summarizes the reasons for these differences in impacts for corridors, which can be stated as follows:

- The LCV scenario causes a decrease in overweight axles; therefore, the more overweight axles in the WIM data, the better the impact of the LCV scenario on annualized overlay costs.
- The LCV scenario had no impact on rigid pavements, as previously discussed; therefore, the greater the number of rigid pavement lane-miles, the less impact from the LCV scenario.

Table 2.24: Major Causes for Differences in LCV Impacts Among Study Corridors

	LCV Impact on Annual Overlay Costs	Reasons		
		% Rigid Pavement	*% Singles >20K*	*%Tandems >34K*
Dallas El Paso	decrease	52.3%	3.99%	31.29%
Dallas Laredo	decrease	19.8%	0.38%	6.85%
San Antonio McAllen	small increase	6.6%	0.28%	2.87%
Dallas Houston	(none)	100%		

2.9.2 Rigid Pavements

The recommended minimum thickness for Texas's rigid pavements is 8 inches. "The reasoning behind this is that slabs less than 8 in. have substantially higher deflections and stresses than an 8-in. slab, which makes using an 8-in. slab cost-effective" (Ref. 2-18). This recommendation is indeed important. Rigid pavements' fatigue equations (see Equations 2.4, 2.5 and 2.6) are extremely sensitive to minor variations in the stress ratio, as shown in Figure 2.16, which depicts these equations in graphic format. It is therefore essential to ensure that the pavement remains in the area of the graph that maximizes the number of allowable repetitions to fatigue.

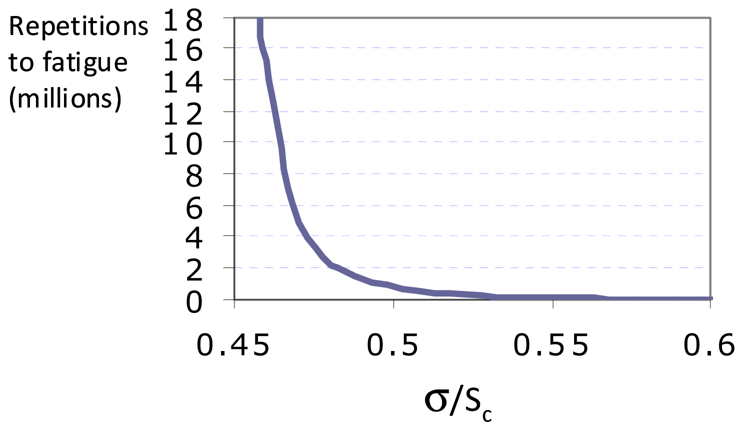


Figure 2.16: Rigid Pavement Fatigue

This practice is the basic reason why the LCV scenario did not have a significant impact on rigid pavements: according to the state-of-the-practice in mechanistic analysis, relatively thick slabs on good foundations have long fatigue lives (due to vertical loads). For these cases, LCV cost allocation should be based on environmental damage and the overall cost of providing and maintaining the pavement.

2.9.3 Flexible Pavements

The LCV scenario improved expected life in all corridors except San Antonio–McAllen. In flexible pavements with 14 in. HMAC over good bases and sub-bases, fatigue was due to rutting and the LCV scenario decreased pavement life. However, as expected, the pavement lives are large compared to the lives of flexible pavements with thinner top layers in both scenarios. In segments with overweight axles, the LCV scenario improved pavement lives; generally, the thicker the HMAC layer the greater the Δ_{life}. Finally, heavy tridem axles appeared to cause little additional damage to the pavement.

2.9.4 Study Limitations and Recommendations for Additional Research

Awareness of this study's limitations is a major factor in correctly interpreting and utilizing its recommendations. A great responsibility underlies the estimation of changes in load spectra and traffic due to the introduction of larger, heavier, and more productive trucks. The possible scenarios are numerous, and so are their implications in pavement maintenance.

Besides well-known, exhaustively discussed uncertainties underlying any study that relies on traffic, soil, and material characterization data, this study has the following particular limitations:

1. Only one LCV scenario was examined; the possibilities are nearly infinite, so it is recommended to develop a study to determine possible, realistic additional LCV scenarios, in concert with TxDOT and based on stakeholder information, as done in this study.
2. A major factor underlying the study results is the assumption that the LCV scenario will substitute, in some cases, most overweight Class 9's; however, this substitution may not actually occur. Additional scenarios should be examined with different rates of overweight trucks remaining in the traffic mix. A possible scenario worth exploring would be as follows: rather than maintaining the overweight class 9's proportions observed at the WIM stations after decreasing their overall number due to the LCV scenario, assume that all observed overweight class 9 axles would remain in the mix, and redistribute the load only among the legally loaded ones. Another interesting scenario would be to look at impacts of overloaded LCVs
3. This study's objective was to examine the impacts of the LCV scenario on pavements; as such, calculations were limited to strains and stresses due to axle loads. However, pavement distress is also due to environmental factors; an examination of the interactions of the LCV axle loads and environmental factors will be necessary in order to allocate costs and estimate registration and permit prices.
4. Pavement maintenance scenarios were limited to periodic HMAC overlays in the annualized cost analysis. Other scenarios should be developed in concert with the Districts and analyzed.
5. Even within this single maintenance scenario, a rather large variation arose in overlay unit price. This variation would need to be accounted for when estimating LCV costs for registration purposes.
6. WIM data were extrapolated as observed to corridor segments that do not have a WIM station. Ideally, a comprehensive study should also conduct a sensitivity analysis of the WIM data, and develop scenarios, especially in the case of WIM stations with a significant percentage of overweight trucks.

Chapter 3. Potential LCV Impacts on Bridges

3.1 Chapter Objective and Organization

The previous chapter reported that pavements on the selected Texas corridors were able to carry the additional LCV axle loads without insurmountable costs being imposed on the system and its users. Historically, however, bridge impacts been the key constraint that limited adoption of programs to introduce large, more productive trucks in the United States. In Texas, less than 20 percent of the on-system network carries over 70 percent of the truck vehicle miles travel (VMT); therefore, trucking depends on key parts of the network—which can be viewed as "corridors"—to remain efficient and competitive. Axle and gross weights impose different loads on a structure and bridge designers have developed a variety of designs[14] since the inception of the interstate system to accommodate changes in vehicle design, size and mass. Bridges deemed to be sensitive to the heavier loads must be strengthened or replaced and all structures on a specific corridor must be evaluated, corrected (if necessary) and approved prior to the operation of the first heavier vehicle. This can lead to high "up-front" costs which are difficult to fund from trucking fees. If this is the case in Texas, the costs will be incorporated into trucking fees that meet equity and productivity targets.

This chapter benefits from a substantial amount of prior work undertaken by the research team—principally at UTSA—over a twenty year period for Federal (Ref. 3-9) and state agencies (Ref. 3-11). TxDOT has used elements of the models in various strategic studies (Ref. 3-8) and they are understood and accepted by staff at the Bridge Division. The chapter discusses the bridge analysis methodology, hypotheses, and assumptions; summarizes the data collected and explains data reduction procedures; documents the results of the bridge and cost analyses for each study corridor; and closes with conclusions and recommendations.

3.2 Analysis Methodology

3.2.1 Objective

The objective of this bridge analysis is to estimate the cost impacts of the proposed LCV scenario on bridges, using the four aforementioned corridors as case studies. This effort is neither a full-cost nor a cost allocation study; rather, its objective is to develop comparative measures of LCV impacts that can be used to aid in deciding whether to allow LCVs in Texas. The study's calculations are limited to the effects of the LCV configurations on bridge replacement. Nevertheless, if LCVs are approved, it will be necessary to estimate the impacts of delays caused by work zones and detours to upgrade bridges deficient for LCV operations.

3.2.2 Data Availability and Approach

Bridge analysis for policy studies must rely on readily available bridge data. The FHWA's National Bridge Inventory (NBI), known as BRINSAP in Texas, is the only dataset that meets this objective. Unfortunately, BRINSAP does not contain detailed data on the individual bridge design elements, thus ruling out the analysis of fatigue, shear, or other stresses that require this level of detail. However, the NBI/BRINSAP does contain sufficient data describing the

[14] The BRINSAP data base captures the design on each structure which allows the method used in this study to undertake a bridge by bridge moment analysis.

bridge length, support type, design type, and material that permits the accurate estimation and computation of the live load and total bending moments. This is an additional reason why previous studies of national truck size and weight (TS&W) policy issues have either ignored fatigue effects and other less critical stresses, or handled them in a very simplified manner. But, as discussed before, little is gained by considering fatigue or other stresses, because the bending stress is a defensible surrogate for all stresses. In the Texas case, the nature of bridge construction and the existing bridge inventory further minimizes the fatigue issue. Most bridges in the main routes of Texas are pre-stressed concrete bridges that are less prone to fatigue effects than steel bridges. Nevertheless, this report summarizes a proposed fatigue approach that allows for the estimation of reductions in remaining life for bridges due to the introduction of LCV configurations. This fatigue approach is included in addition to the traditional approach of rendering bridges deficient due to overstress triggered by live load bending moment ratios.

3.2.3 Analysis Scope

This section documents the estimated impacts of LCV on the bridge infrastructure of the following Texas corridors, selected in concert with the PMC and other stakeholders as case studies (Ref. 3-1):

- IH 20/IH 10 from Dallas to El Paso.
- IH 35 from Dallas to Laredo.
- IH 37/US 281 from San Antonio to Weslaco/McAllen.
- IH 45 from Dallas to Houston.

The research team retrieved existing bridge data on the routes selected for the case studies using a combination of SAS programming and a GIS system with data sets prepared separately for the four case study routes. Figure 3.1 summarizes the bridge statistics for the proposed case study routes. The total number of bridges encompassed by the case study routes is 1,713. Bridge construction year statistics and bridge type statistics—relevant for the fatigue analysis presented later in this chapter—are respectively presented in Figures 3.2 and 3.3 for the study routes. Numeric codes in Figure 3.3 are consistent with the definitions in the BRINSAP coding guide.

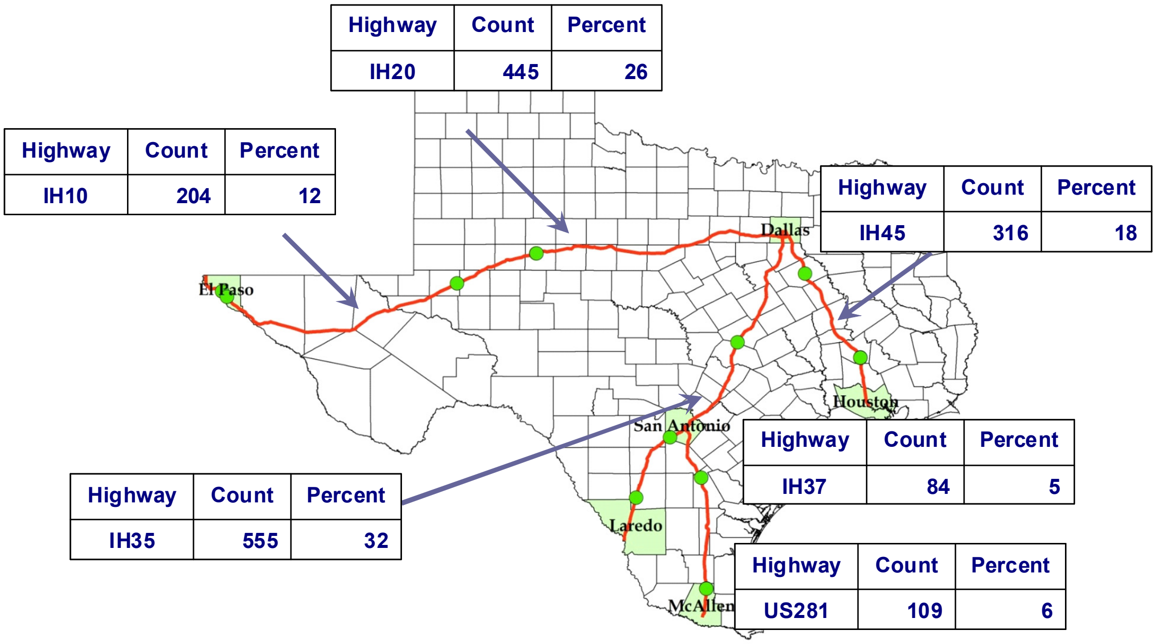


Figure 3.1: Study Routes Bridge Statistics

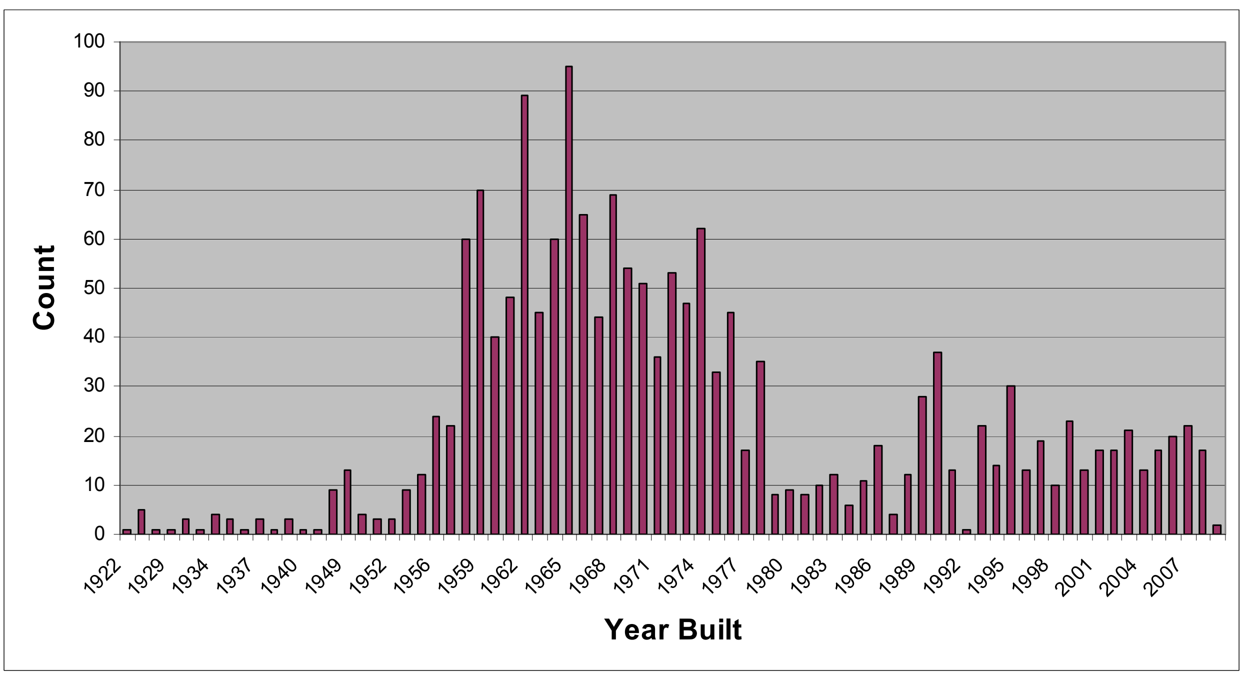


Figure 3.2: Study Routes Bridge Construction Year Statistics

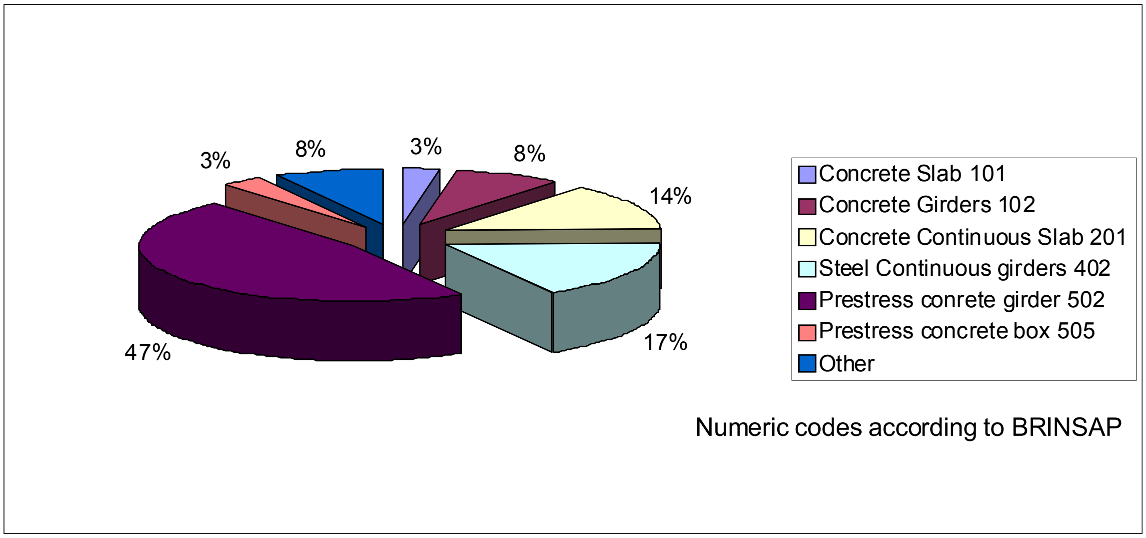


Figure 3.3: Study Routes Bridge Type Statistics

3.2.4 LCV Configurations

The same LCV configurations described in the pavement impacts chapter were used to model the bridge impacts. However, axle spacing information was added because it influences the results of the bending moment methodology described later in this chapter. Figure 3.4 summarizes the proposed LCV configurations for this study.

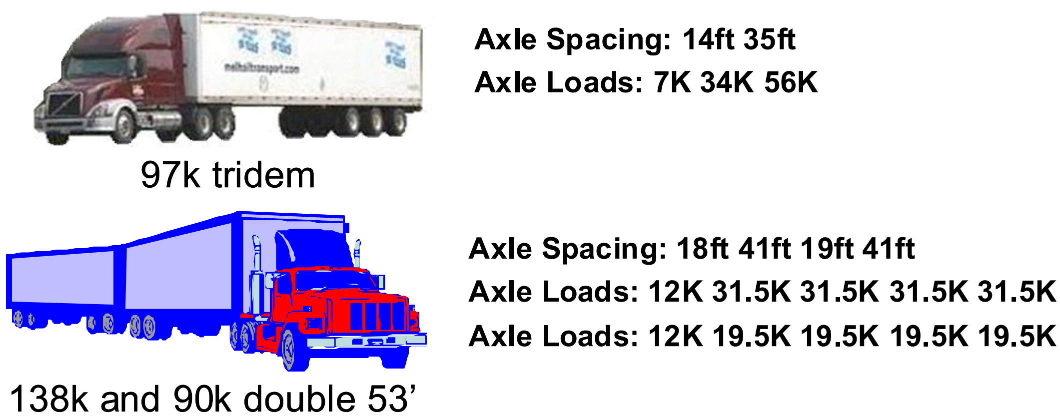


Figure 3.4: LCV Configurations for Bridge Analysis

Due to the nature of the available bridge analysis methodology to screen deficiencies to LCV operations, load spectra changes due to proposed LCV configurations will not be taken into account. However, the simplified fatigue methodology described later in this chapter could be improved by the consideration of modified load spectra.

3.2.5 Traditional Moment Analysis Methodology

The data available in the NBI/BRINSAP allows the application of simplified methodologies to screen deficient bridges for proposed traffic configurations at the policy level. Several authors have implemented this process for evaluating bridges along a given route or for a given highway system for specific truck configurations (Refs. 3-11, 3-12, 3-3, 3-5). The essence of all these methods is summarized by the following formula:

$$OR = \frac{MLLIO + MDL}{MLLIR + MDL} \tag{3.1}$$

where:

OR—the overstress ratio,

MLLIO—maximum bending moment caused by the live load plus the impact factor of the proposed vehicle,

MDL—maximum moment due to the dead load, and

MLLIR—maximum bending moment caused by the live load plus impact factor of the rating load from operating or inventory levels recorded in the NBI (Ref. 3-7).

An overstress ratio equal to or less than one means that the bridge is within the selected stress limit (operating or inventory). An overstress ratio greater than one means that the proposed vehicle—the LCV configuration in this case—causes stresses above the selected stress limit.

None of the size and weight studies available in the literature carry out an analysis incorporating the dead loads due to the unavailability of this information in the NBI. The Moment Analysis of Structures (MOANSTR) (Ref. 3-12) computerized routine incorporates the dead loads in the analysis, and was used in all the recent FHWA TS&W studies (Ref. 3-9). These studies take advantage of live load to dead load moment ratios developed in previous research (Weissmann, et al.,1994) to support a full moment analysis (live + dead load moments). However, in order to allow for comparisons between the traditional moment ratio approach and the fatigue remaining life approach, this analysis used live load moment ratios and inventory rating loads.

MOANSTR was used to calculate live load moment ratios as defined by a modification of Equation 3.1 to account for live load moment ratios. The MOANSTR program's core is a finite differences routine that calculates the live load moment envelopes generated by the proposed truck configurations and the NBI/BRINSAP rating loads. The MOANSTR routine incorporates previous research work developed by Matlock (Ref. 3-4). MOANSTR calculates moment envelopes and identifies the maximum live load bending moments (positive and negative) induced by the proposed configuration and the inventory rating load.

Figure 3.5 summarizes the live load moment ratio approach. In the first step, data relevant to the previously defined case study routes were mined from BRINSAP with a combination of SAS programs using the latitude longitude and route number from BRINSAP and GIS. Figure 3.6 depicts the retrieved BRINSAP bridge data on a GIS map to confirm that the BRINSAP data mining was successful in identifying the case study routes target bridges. The next step in the analysis was to evaluate the case study bridges for the operation of the current legal traffic. In order to represent the existing traffic, the fully loaded legal CS5 configuration depicted in Figure 3.7 was simulated using MOANSTR over the case study bridges. This CS5 configuration is representative of the prevalent five-axle semi-trailer tractor-trailer configuration on Texas' highways. Bridges that are already deficient to the CS5 configuration using a 1.1 live load moment overstress criteria were then discarded from further analysis so as not to include bridges that are already deficient for other reasons in the LCV impact analysis. Researchers then submitted bridges not deficient for current traffic to a live load moment ratio analysis using the three case study LCV configurations previously discussed in this chapter. For each bridge in the selected case study routes, MOANSTR calculated live load moment envelopes. Bridges that exceeded the established moment ratio threshold were screened and their deck area and Average Daily Traffic (ADT) recorded for further analysis of cost impacts.

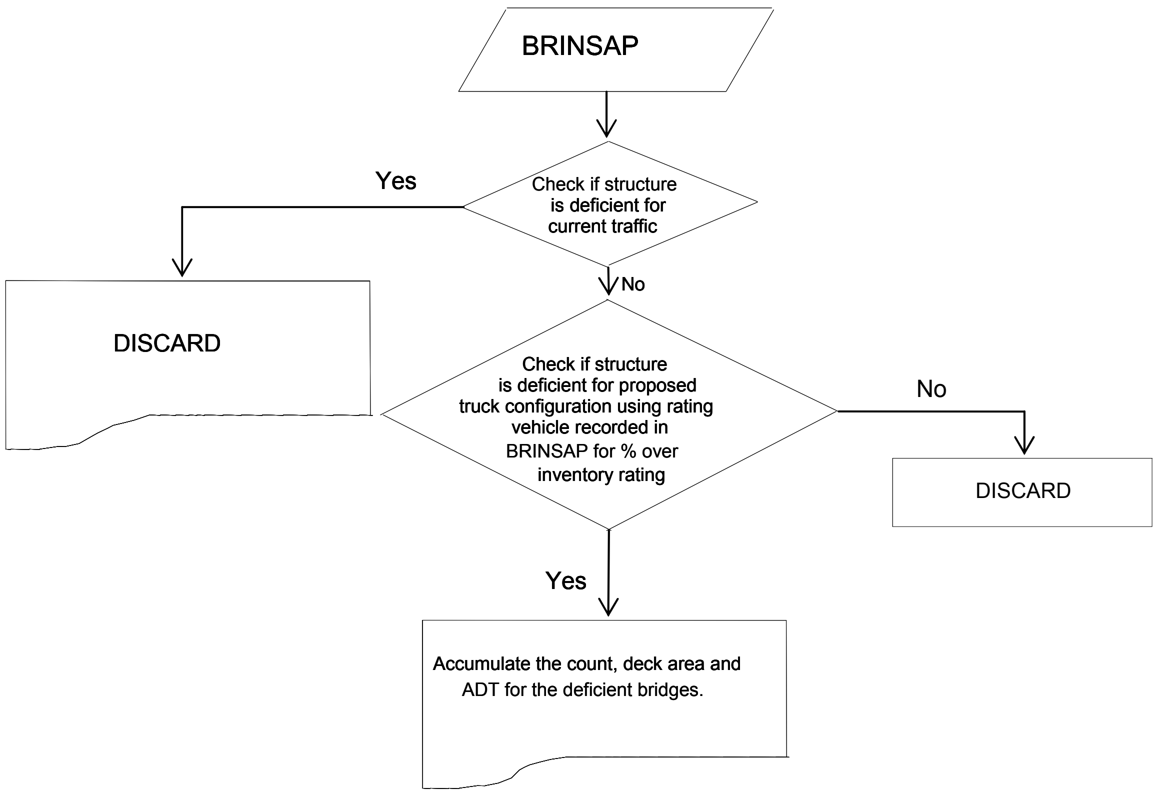


Figure 3.5: Flowchart of the Bridge Analysis Methodology

3.2.6 Fatigue Moment Analysis Methodology

Fatigue is a cumulative process in which repetitive stress cycles accumulate damage until failure occurs. The basic concept of the fatigue design and assessment for bridges relates to the fact that each cycle of truck passage causes some damage. The damage due to a population of trucks accumulates until failure (usually cracking) occurs. The damage caused by each truck depends on the vehicle weight, the bridge span length, and member section dimensions. Considering that vehicle weight and span are directly used to calculate bending moments and member section and bending moment are used in calculating stresses, stresses are directly proportional to live load bending moments.

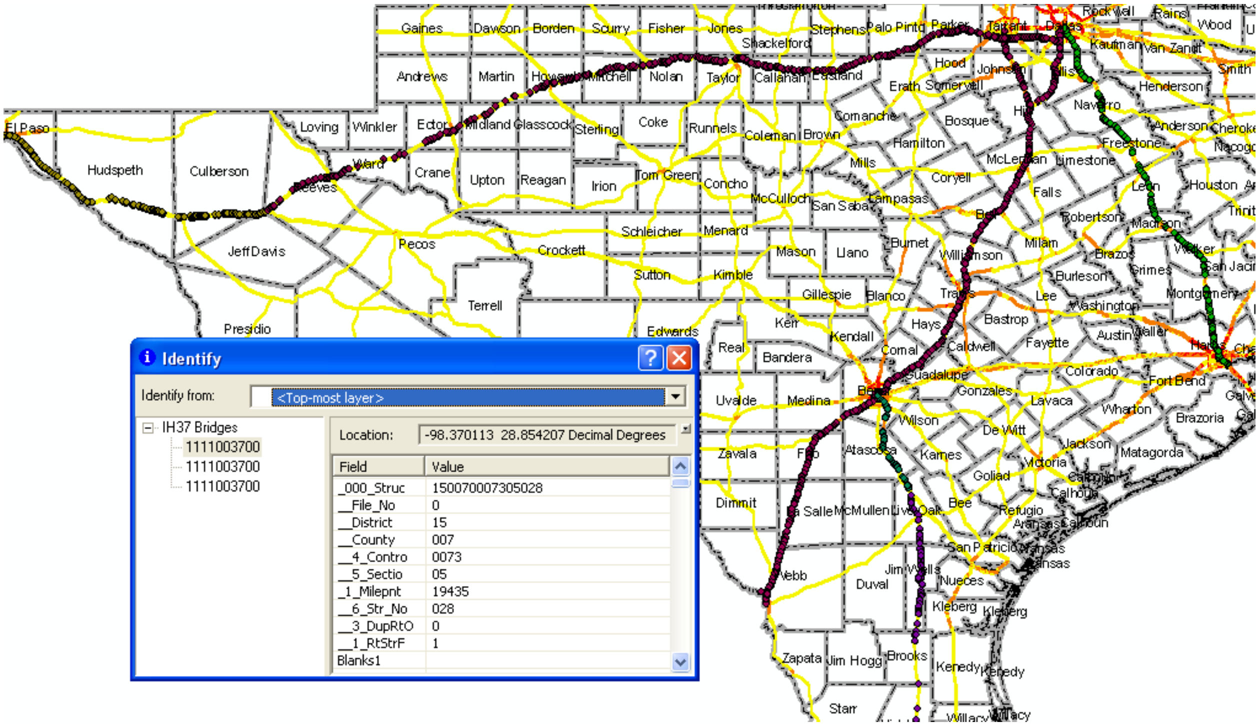


Figure 3.6: GIS Map Depicting Retrieved Bridges on Case Study Routes

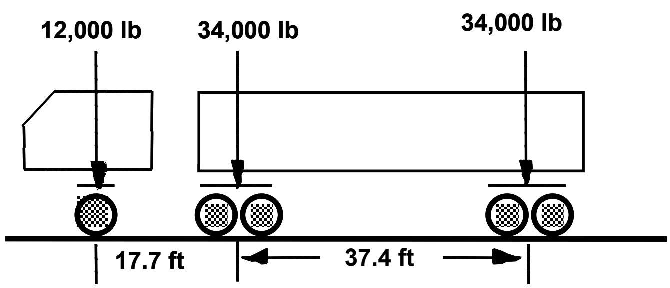


Figure 3.7: CS5 Existing Traffic Configuration

Experimental data and fracture mechanics principles indicate that fatigue damage is proportional to the third power of the Stress Range. Stress range is the difference between the maximum and minimum stress caused by a vehicle passage at the location of concern. The exponent of 3.0 for the welded steel attachments is an important parameter in comparing influences of variable stress amplitudes. It means that if the stress amplitude is doubled, the fatigue damage will increase by a factor of eight. To account for different stress ranges due to various truck weights, a linear damage accumulation law is usually assumed. The damage of one stress cycle is inversely proportional to the life that would exist if that stress of constant amplitude was cyclically repeated. The life for constant stress amplitude is predicted using the stress-life (S-N) curve for that type of bridge detail, developed based on physical testing.

AASHTO bridge specifications (Ref. 3-1) include fatigue curves that assume a 75-year design life for different details in a bridge. These curves have the general format represented by Equation 3.2, and assume stress ranges compatible with inventory rating loads.

$$NS^m = C \qquad \text{or} \qquad \text{Log } N = \log C - m \text{ Log } S \tag{3.2}$$

Where:
N—Number of cycles
S—Stress Range
m—Constant dependent on material and bridge detail
C—constant

A typical AASHTO fatigue curve is presented in Figure 3.8 for different steel bridge details.

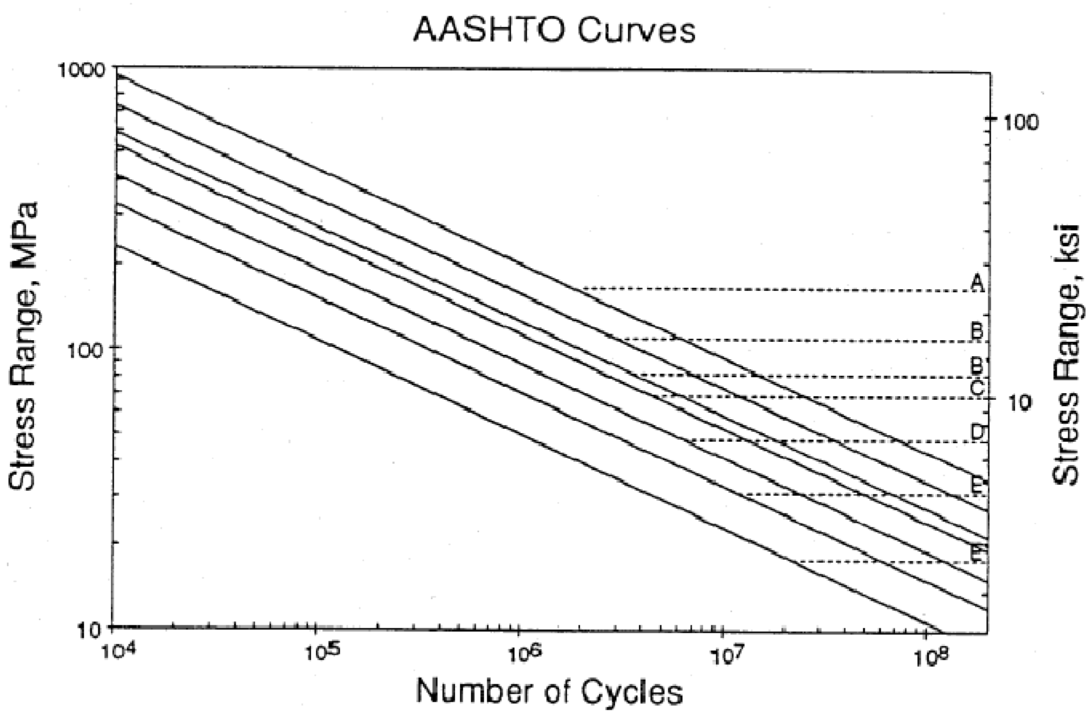


Figure 3.8: AASHTO Fatigue Curves

A simplified approach based on Equation 3.2 can be used to estimate the impact on the remaining life of a given bridge. This approach assumes that live load moment ratios are acceptable surrogates to measure stress ranges. This simplified approach is represented by Equations 3.3 and 3.4.

$$\frac{N_{existing}}{N_{LCV}} = \frac{S_{LCV}^m}{S_{existing}^m} \tag{3.3}$$

Where:
$N_{existing}$, N_{LCV}—number of cycles for existing traffic and LCV respectively
$S_{existing}$, S_{LCV}—Stress ranges for existing traffic and LCV respectively
m—Constant dependent on material and bridge detail

Assuming that the stress ranges are proportional to the live load bending moments and that the load spectra for the existing scenario and the LCV scenario are similar, the stress ratio can be replaced by the moment ratios between the LCV configuration and the inventory rating load. Assuming a 75 year design life, the effect on the remaining life of a bridge can be summarized by Equation 3.4.

$$R_{life} = (75 - Age)\left(\frac{M_{inventory}}{M_{LCV}}\right)^{m} \tag{3.4}$$

Where:

$M_{inventory}$, M_{LCV}—Live load moments for the Inventory Rating load and LCV configuration respectively

R_{life}—Calculated remaining life for a given bridge due to LCV operations

m—Constant dependent on material and bridge detail

Age—Current age of the bridge calculated using the construction date in the BRINSAP record.

Values of "m" were established based on the literature (Refs. 3-6, 3-2, 3-1) and are summarized in Table 3.1 for most of the bridges in the selected case study routes. In addition, the table includes the frequency distribution of the bridge types by NBI/BRINSAP coding, showing that about 47% of the bridges on the case study routes are simply supported prestressed concrete girder bridges.

Table 3.1: Fatigue Constant m for Bridges in the Case Study Routes

Structure Type	Frequency	m value
Other 000	1.9%	3.5
Concrete Slab 101	2.7%	4.1
Concrete Girders 102	8.2%	3.5
Concrete T Beam 104	1.0%	4.1
Concrete Box Beam 105	0.2%	4.1
Concrete Continuous Slab 201	13.6%	4.1
Concrete Continuous T Beam 204	1.6%	4.1
Steel Girder 302	1.8%	3.0
Steel Continuous Girders 402	17.2%	3.0
Steel Continuous Girder 403	0.2%	3.2
Steel Continuous Box Beam 405	0.5%	3.2
Steel Continuous Box Beam 406	0.2%	3.2
Prestressed Concrete 500	0.4%	3.5
Prestressed Concrete Slab 501	0.1%	3.5
Prestressed Concrete Girder 502	46.8%	3.5
Prestressed Concrete Box Beam 505	3.2%	3.5
Prestressed Concrete Continuous 601	0.1%	3.5
Prestressed Concrete Continuous 602	0.4%	3.5

The following quote from a recent report on the effect of overstress on Minnesota bridges supports analysis assumptions later in this chapter regarding moment ratio thresholds:

> "Essentially all prestressed girders, modern steel girders, and most bridge decks could tolerate a 20% increase in truck weight with no reduction in life. Unfortunately, most Minnesota steel girder bridges were designed before fatigue-design specifications were improved in the 1970's and 1980's. Typically, an increase in truck weight of 20% would lead to a reduction in the remaining life in these older steel bridges of up to 42% (a 10% increase would lead to a 25% reduction in fatigue life" (Ref. 3-2).

Based on these statements, this project evaluated the remaining life of bridges with a moment ratio greater than 20% based on inventory rating levels. Not all bridges would have to be replaced immediately depending on the overstress level. To incorporate this concept in the analysis, the project developed a new fatigue approach with the assumption of a 75-year fatigue design life for a bridge. If the moment ratio is between 1.2 and 1.4, the bridge is assumed to have its life shortened by fatigue effects and, depending on its age, trigger an earlier replacement than the assumed 75-year life. Bridges with a moment ratio greater than 1.4 would have to be replaced immediately. In summary, bridges with moment ratios between 1.2 and 1.4 (20% to 40% overstress) had their remaining lives calculated based on Equation 3.4. In addition, the present value of the delayed replacement was calculated using Equation 3.5 using a 5% discount rate and a cost of replacement of $192/sq ft estimated in a previous study (Ref. 3-8).

$$PV = \frac{D_{Area} Cost}{(1+i)^{R_{life}}} \tag{3.5}$$

Where:

D_{Area}—Deck area calculated from BRINSAP records.
Cost—Cost of replacement in dollars/square foot of deck.
i—Discount rate.
R_{life}—Calculated remaining life for a given bridge due to LCV operations

The fatigue methodology can be summarized by the following steps:

1. Calculate live load moment ratios using the MOANSTR routine previously described in the traditional moment analysis paragraph. Moment ratios are calculated for the CS5 configuration—representative of the existing traffic—and the proposed LCV configurations for the case study routes using Equation 3.1.
2. Screen the bridges deficient for the existing traffic, using the traditional moment analysis described previously. A moment ratio greater than 1.1 for the CS5 configuration is the assumed threshold for deficiencies.
3. Screen the bridges with live load moment ratios greater than 1.4 for the LCV configurations. Target these bridges for immediate replacement. Calculate replacement costs based on $192/sq ft of deck area.
4. Screen the bridges with live load moment ratios greater or equal to 1.1 or less or equal to 1.4 for the LCV configurations. Apply the fatigue methodology summarized by Equations 3.4 and 3.5 and record the present value for each of the screened bridges.

3.3 97-kip Tridem Results

3.3.1 Traditional Moment Analysis

Moment ratio distribution for the 97-kip tridem configuration is presented in Figure 3.9. About 40% of the bridges in the case study routes have a live load moment ratio as defined by Equation 3.1 greater than 1.2, leading to an expectation that bridge impacts for this configuration to be quite significant.

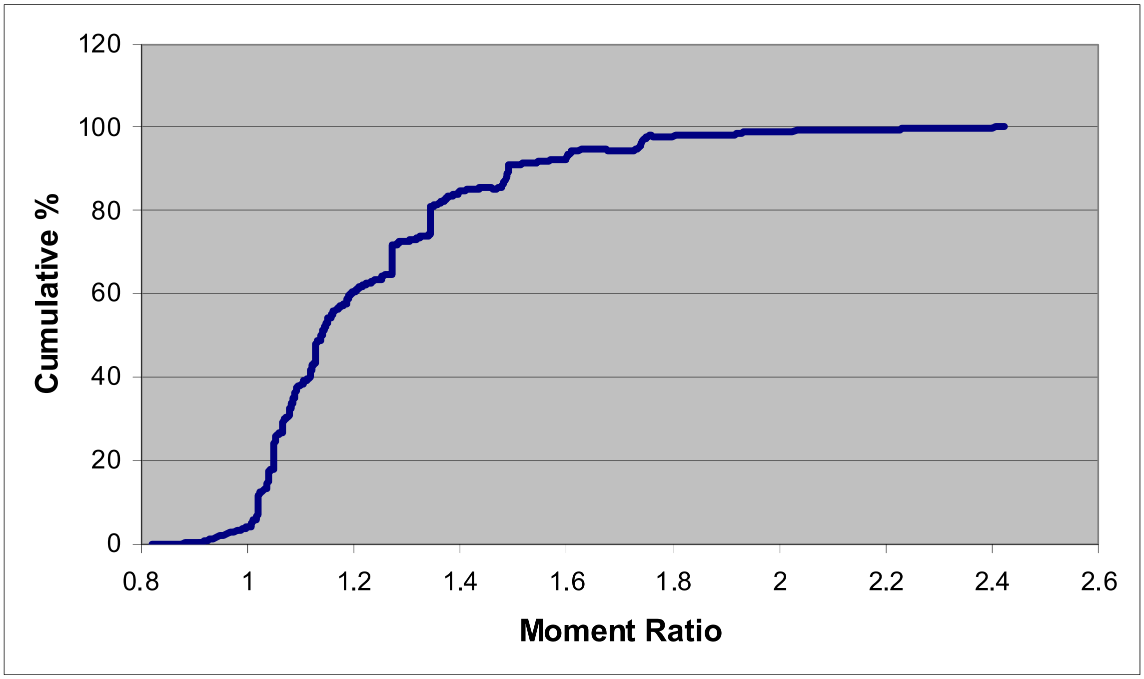


Figure 3.9: Live load Moment Ratio Cumulative Distribution for the 97-kip Tridem

Tables 3.2 and 3.3 summarize the results of the traditional moment analysis for moment ratio thresholds of 1.1 and 1.2 respectively. Results are summarized by highway corridor and include number of deficient bridges, area of deficient bridges, and ADT of the deficient bridges to provide an estimate of impacted traffic in case of bridge replacement or strengthening and finally cost, based on a replacement cost of $192/sq ft. Replacement costs are about $2.8 billion for the 1.1 moment ratio threshold and $1.1 billion for the 1.2 moment ratio threshold.

Table 3.2: Impacts of the 97-kip Tridem Using a 1.1 Moment Ratio Threshold

Highway	# Bridges	Area (sq ft)	ADT	Cost $
IH 10	145	1,092,520	3,822,520	207,578,743
IH 20	255	2,694,798	3,657,001	512,011,639
IH 35	289	7,097,868	10,091,459	1,348,594,958
IH 45	89	1,954,679	4,962,040	371,388,953
IH 37	42	793,428	1,258,670	150,751,358
US 281	60	999,060	617,330	189,821,457
Total	**880**	**14,632,353**	**24,409,020**	**2,780,147,108**

Table 3.3: Impacts of the 97-kip Tridem Using a 1.2 Moment Ratio Threshold

Highway	# Bridges	Area (sq ft)	ADT	Cost $
IH 10	126	836,570	3,300,400	158,948,357
IH 20	189	1,274,125	1,886,420	242,083,712
IH 35	183	2,938,770	6,130,009	558,366,357
IH 45	47	643,122	1,324,970	122,193,237
IH 37	14	137,679	433,460	26,158,972
US 281	23	164,369	113,730	31,230,015
Total	**582**	**5,994,635**	**13,188,989**	**1,138,980,650**

3.3.2 Fatigue-Based Moment Analysis

Table 3.4 summarizes the results of the fatigue moment analysis with the moment ratio thresholds discussed in the methodology paragraph. Results are summarized by highway corridor and include number of deficient bridges and associated costs, based on a replacement cost of $192/sq ft.

Results are split by moment ratios. Bridges with moment ratios greater than 1.4 were targeted for immediate replacement. Replacement costs of bridges with moment ratios less or equal to 1.4 were calculated using the remaining life concepts previously discussed, and using a 5% discount rate.

Replacement costs are about $1 billion dollars for the fatigue approach with 690 bridges in the case study corridors screened with the fatigue approach using a remaining life life-cycle approach.

Table 3.4: Fatigue-Based 97-kip Tridem Bridge Impacts

Highway	# Bridges ratio > 1.4	# Bridges ratio ≤ 1.4	PV Cost Bridges ratio>1.4	PV Cost Bridges ratio ≤1.4	Total PV
IH 10	93	52	95,652,327	49,340,578	144,992,905
IH 20	29	225	29,743,816	188,538,999	218,282,815
IH 35	39	250	73,419,591	369,055,608	442,475,199
IH 45	8	81	7,345,514	109,020,049	116,365,563
IH 37	0	42	-	51,468,909	51,468,909
US 281	18	40	23,967,778	29,241,925	53,209,703
Totals	**187**	**690**	**230,129,026**	**796,666,067**	**1,026,795,093**

3.4 138-kip Double 53 Results

3.4.1 Traditional Moment Analysis

Moment ratio distribution for the 138-kip double 53 configuration is presented in Figure 3.10. About 15% of the bridges in the case study routes have a live load moment ratio as defined by Equation 3.1 greater than 1.2, leading to an expectation that bridge impacts of this configuration would be less significant than those of the 97-kip tridem configuration (summarized earlier).

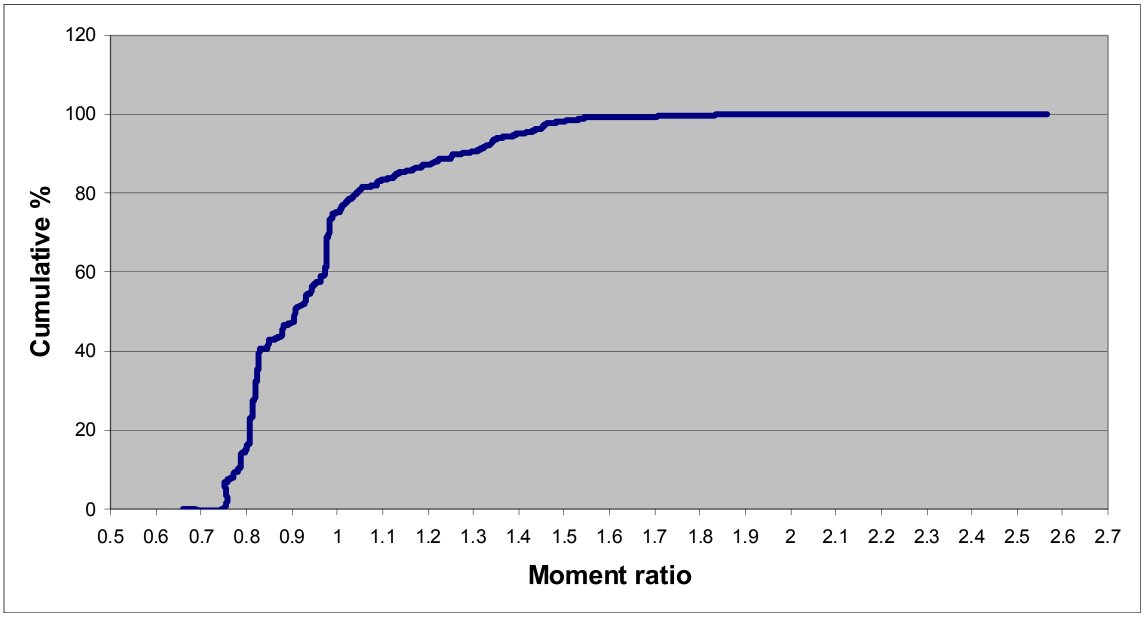


Figure 3.10: Live Load Moment Ratio Cumulative Distribution for the 138-kip Double 53

Tables 3.5 and 3.6 summarize the results of the traditional moment analysis for moment ratio thresholds of 1.1 and 1.2 respectively. Results are summarized by highway corridor and include number and area of deficient bridges ADT of the deficient bridges (to provide an estimate of impacted traffic in case of bridge replacement or strengthening) and, finally, cost, based on a replacement cost of $192/sq ft.

Replacement costs are about $1.2 billion for the 1.1 moment ratio threshold and $1 billion for the 1.2 moment ratio threshold. The US 281 route did not result in any deficient bridges for both moment ratios possibly due to the fact that it includes short-span simply supported bridges. As expected, impacts for the 138-kip double 53 configuration are lower than for the 97-kip tridem configuration.

Table 3.5: Impacts of the 138-kip Double 53 Using a 1.1 Moment Ratio Threshold

Highway	# Bridges	Area (sq ft)	ADT	Cost $
IH 10	7	148,468	427,660	28,208,844
IH 20	35	487,417	728,060	92,609,192
IH 35	116	4,632,500	4,904,869	880,174,981
IH 37	30	642,587	1,053,110	122,091,568
IH 45	13	312,459	989,500	59,367,134
Total	**201**	**6,223,430**	**8,103,199**	**1,182,451,719**

Table 3.6: Impacts of the 138-kip Double 53 Using a 1.2 Moment Ratio Threshold

Highway	# Bridges	Area (sq ft)	ADT	Cost $
IH 10	7	148,468	427,660	28,208,844
IH 20	27	370,349	419,150	70,366,348
IH 35	95	3,772,940	4,812,269	716,858,562
IH 37	30	644,855	1,091,850	122,522,488
IH 45	14	316,820	1,015,160	60,195,876
Total	**173**	**5,253,432**	**7,766,089**	**998,152,118**

3.4.2 Fatigue-Based Moment Analysis

Table 3.7 summarizes the results of the fatigue moment analysis with the moment ratio thresholds discussed in the methodology section. Results are summarized by highway corridor and include number of deficient bridges and associated costs, based on a replacement cost of $192/sq ft.

Results are split by moment ratios. Bridges with moment ratios greater than 1.4 were targeted for immediate replacement. Replacement costs of bridges with moment ratios less or equal to 1.4 were calculated using the remaining life concepts previously discussed, and using a 5% discount rate.

Replacement costs are about $0.8 billion for the fatigue approach with 150 bridges in the case study corridors screened with the fatigue approach using a remaining life life-cycle approach.

Table 3.7: Fatigue-Based 138-kip Double 53 Bridge Impacts

Highway	# Bridges ratio > 1.4	# Bridges ratio≤ 1.4	PV Cost Bridges ratio>1.4	PV Cost Bridges ratio ≤1.4	Total PV
IH 10	2	5	4,089,408	12,510,994	16,600,402
IH 20	5	30	19,798,798	31,217,358	51,016,156
IH 35	33	83	477,864,573	139,248,361	617,112,934
IH 37	5	25	15,094,987	51,400,797	66,495,784
IH 45	6	7	19,991,515	22,200,123	42,191,638
Totals	**51**	**150**	**536,839,281**	**256,577,633**	**793,416,914**

3.5 90-kip Double 53 Results

Results of the traditional moment analysis for the 90-kip double 53 configuration are presented in Figure 3.11. Practically none of the bridges in the case study routes have a live load moment ratio as defined by Equation 3.1 greater than one, leading to an expectation that bridge impacts for this configuration would be negligible.

As expected, and based on Figure 3.11 moment ratio cumulative frequency distribution, the analysis indicated no impacts for both the traditional and fatigue based analysis.

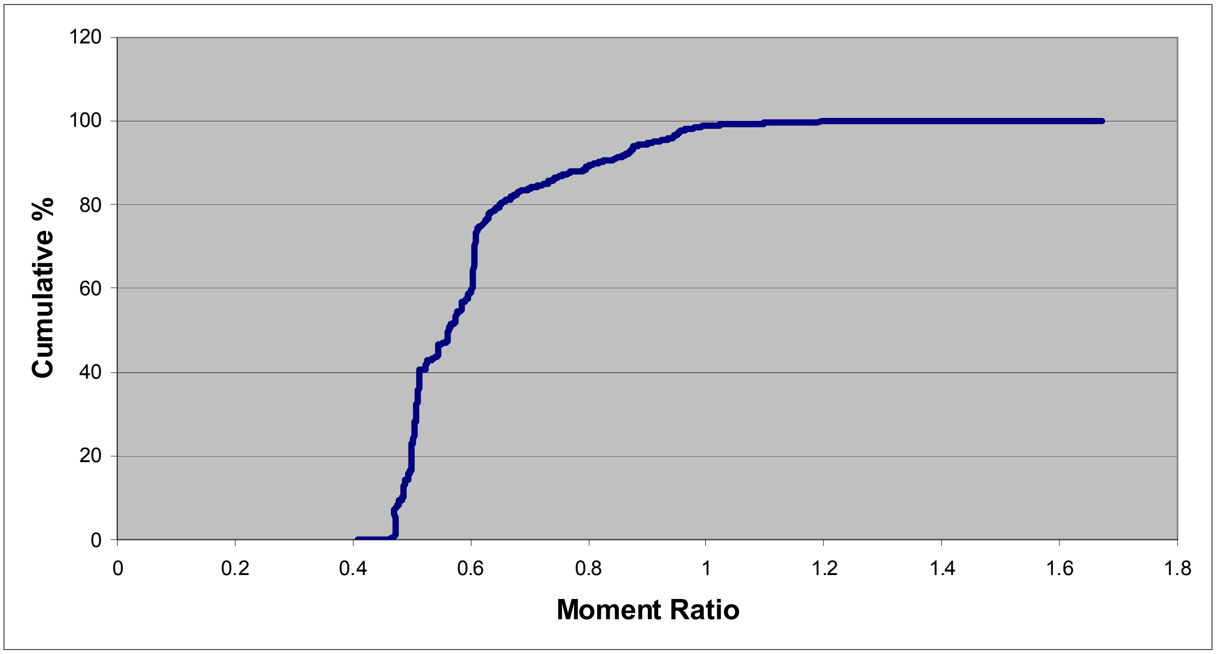


Figure 3.11: Live Load Moment Ratio Cumulative Distribution for the 90-kip Double 53

3.6 Combined Results

Impacts calculated with the traditional moment analysis were also summarized by running the three proposed configurations concurrently, a scenario consistent with the pavement analysis approach reported in Chapter 2. Results are summarized in Tables 3.8 and 3.9 for moment ratio thresholds of 1.1 and 1.2 respectively.

Results in Tables 3.8 and 3.9 were determined by using a procedure that avoided double-counting and a scenario wherein a bridge deficiency was triggered by the most damaging configuration. The results in Tables 3.8 and 3.9 show combined proposed configuration deficiencies are probably being controlled primarily by the 97-kip tridem configuration. Impacts are around \$2.8 billion and \$1.9 billion for the 1.1 and 1.2 moment ratios respectively. Bridges targeted for replacement by the analysis affect about 25 million vehicles per day, leading to potentially high user delay costs if replacement or strengthening was to be implemented.

Table 3.8: Impacts of the Combined Configurations Using a 1.1 Moment Ratio Threshold

Highway	# Bridges	Area (sq ft)	ADT	Cost $
IH 10	145	1,092,520	3,822,520	207,578,743
IH 20	257	2,709,810	3,678,501	514,863,919
IH 35	293	7,169,479	10,346,609	1,362,201,010
IH 45	89	1,954,679	4,962,040	371,388,953
IH 37	42	793,428	1,258,670	150,751,358
US 281	60	999,060	617,330	189,821,457
Total	**886**	**14,718,976**	**24,685,670**	**2,796,605,440**

Table 3.9: Impacts of the Combined Configurations Using a 1.2 Moment Ratio Threshold

Highway	# Bridges	Area (sq ft)	ADT	Cost $
IH 10	130	914,899	3,403,570	173,830,753
IH 20	202	1,449,063	2,090,690	275,322,008
IH 35	246	6,023,241	8,770,989	1,144,415,828
IH 37	36	694,103	1,225,590	131,879,608
IH 45	53	837,670	1,879,790	159,157,262
US 281	23	164,369	113,730	31,230,015
Total	**690**	**10,083,345**	**17,484,359**	**1,915,835,474**

3.7 Summary of Findings, Recommendations, and Discussion

Table 3.10 summarizes the results of the bridge analysis by vehicle, moment ratio threshold, and fatigue methodology. Principal findings are:

- The 97-kip tridem configuration has the greatest impacts on the bridges of the case study routes, also rendering the largest number of bridges deficient.
- The 90-kip double 53 configuration has no impact on the bridges on the case study routes and is recommended for immediate implementation with no associated bridge costs.
- The combined scenario—97-kip tridem and 138-kip double 53—is most likely being controlled by the 97-kip tridem configuration and presents significant bridge impacts.
- The fatigue approach screens some of the bridges for delayed replacement or strengthening and reduces the bridge impacts as expected.
- The IH 35 corridor selected for the case study presents the largest contribution to the bridge impacts and should be studied separately on future pilot implementation of the proposed configurations.
- Implementation of the proposed configurations should be combined with a detailed plan for cost recovery of the bridge impacts.

Table 3.10: Summary of the Bridge Analysis for the Different Approaches

Scenario	# Bridges	Cost $ (billions)
97-kip Tridem 1.1 Moment Ratio	880	2.78
97-kip Tridem 1.2 Moment Ratio	582	1.14
97-kip Tridem Fatigue Approach	877	1.03
138-kip Double 53 1.1 Moment Ratio	201	1.18
138-kip Double 53 1.2 Moment Ratio	173	1.00
138-kip Double 53 Fatigue Approach	201	0.79
Combined LCV Configurations 1.1 Moment Ratio	886	2.80
Combined LCV Configurations 1.2 Moment Ratio	690	1.92

Chapter 4. Findings and Recommendations

4.1 Findings/Recommendations

4.1.1 Users

In debates[15] surrounding the introduction of more productive trucks, whether they are single vehicles such as the 97-kip tridem semi-trailer or various types of combination trucks, a common concern is that of cost allocation and the payment for the additional (marginal) damage that such vehicles create on the highway network. The reasoning for such thinking is not irrational. A number of cost allocation studies undertaken in the U.S.[16] since the late 1980s have found that heavy trucks—like those in Class 9—are frequently subsidized by lighter vehicles[17]. Accordingly, those opposing the introduction of larger, more productive vehicles seize on allocated costs as an issue of significance in public policy debates. And they have found that legislators, mindful of general public anxiety over sharing highways with growing numbers of large trucks, are reluctant to introduce changes to a user segment that does not fully pay its equitable share of highway costs. The solution is simple albeit apparently unconvincing to most highway planners; the marginal cost on that part of the network used by the vehicles needs to be calculated, regularly updated, and then allocated to the users benefiting from their introduction. This issue—that the beneficiaries pay—is now broadly accepted by the U.S. trucking sector. The study team found, through transportation literature, company interviews, and discussions with the study operator panel that the industry is willing to cover the marginal costs created by operating larger trucks. The reason for this willingness is simple—the ability to derive any further benefit in cost per ton-mile units from improving current truck designs is severely limited. In fact, trucks may get heavier and face reduced future payload limits[18]. That the industry is now willing to pay the marginal costs for a range of products on key corridors should be a defining moment in the long discussion over how to allow truckers to benefit from economies of scale, as other modes have done since 1990.

4.1.2 Pavement

Many previous LCV studies examined the impacts of LCV operations on federal or state networks. This study recognized that such vehicles are used only on freight arteries and would not be part of general truck operations across Texas. It was therefore designed to focus on specific truck corridors in the state where freight movement played a critical impact on state and regional economies. Furthermore, this report reduced the types of LCVs to those recommended by the operator panel. TxDOT engineers are fully aware of the impact that heavy truck axles can impose on highway pavements, although many corridors in the state now have segments of extremely thick pavement, designed for long life[19]. The research team therefore placed great importance on addressing those concerns by undertaking a thorough preliminary analysis of the

[15] These debates go back at least two decades, peaking in the period 1992–1997.
[16] TxDOT has sponsored several cost allocation studies over the past two decades.
[17] In Texas, the last study found that pickup trucks cross-subsidized Class 9 trucks.
[18] The 2010 EPA SCR equipment adds over 300 lb to a tractor and if DOT funding continues to fall, rougher roads may mean stronger vehicles and higher tare weight.
[19] In Houston, segments of the interstate built in the late 1970s had a 20-year life; now those are being replaced with designs of 40 years.

estimated impact that the three types of LCV had on pavements. The scenario assumed the following impacts decreased:

- Tandem & single weight limits,
- Single and tandem axle repetitions,
- Number of overweight single axles,
- Number of overweight tandem axles.

Another assumption was that tridem axle weights and heavy tridem repetitions increased.

Changes in axle weights for each scenario were compared to the base case of not implementing LCVs. Finally, the report provides the differences in costs for various routes if LCVs were allowed. A fundamental result of this analysis was that much of the system is already capable of carrying the three types of LCVs chosen by study. The result for one type—the 90-kip double 53—was expected because the weight carried by each axle is substantially less than the fully loaded 80,000 lb vehicle currently permitted to operate over the federal system. The evidence is that very little of the pavement system would need to be strengthened and that a modest increase in permit or registration fees would cover the marginal cost for pavements impacted by the vehicles. The results of the pavement analysis are shown through the estimated changes in the annualized costs of periodically overlaying the pavements if LCVs were allowed on these routes. The cost is defined as the difference between the annual cost of pavement with LCVs and the annual cost of pavement without LCVs. Depending on the price of construction, allowing LCVs on the Dallas–El Paso and Dallas–Laredo routes could result in a savings of $17.4 to $53.1 million per year. The Dallas–Houston route is rigid pavement and the LCV scenario has no impact on rigid pavement life. The San Antonio–McAllen route exhibited a slight increase in cost of $0.14 to $.044 million per year. The researchers recommend a more detailed study encompassing several scenarios to accurately ascertain pavement impacts. Note also that the operator panel stated that some LCVs on some route segments would be running empty, and this scenario would need to be factored into the more detailed work that could be undertaken during a pilot study.

4.1.3 Bridges

The research team used the same LCV and heavy semi-trailer types, dimensions, gross weights, and axle weights for the bridge impact section as for the pavement analysis. The latest BRINSAP database was provided by TxDOT Bridge Division and the bridges already deficient for the existing truck loads were excluded to avoid double-counting. Two sets of results were calculated using the traditional bridge analysis approach with one set based on the 10% overstress criteria and a second set based on the 20% overstress criteria. The 90-kip double 53 configuration showed no impact on the bridges of the selected case study routes for both overstress ratios. This result is one of the few occasions that an LCV operation has had no impact on bridges while an amount of truck VMT is registered by trucks that *cube out*[20] because of commodity characteristics. The results for the remaining types when loaded with commodities that *weigh out* vary by type; impacts for a 20% overstress over Inventory Rating amount to $1.1

[20] Approximately one-third of the commodities carried cube out, and some LCV VMT involves empty running.

billion for the 97-kip tridem and $1.0 billion for the 138-kip double 53. Impacts for a 10% overstress over Inventory Rating are higher and amount to $2.8 billion and $1.2 billion for the 97-kip tridem and 138-kip double 53 respectively. Another set of results were calculated using the newer bridge analysis based on the fatigue concept. This analysis showed the impacts amount to $1.0 billion and $0.8 billion for the 97-kip tridem and 138-kip double 53 respectively, with no impacts for the 90-kip double 53 configuration. BRINSAP data are extremely useful for policy analysis but those bridges that failed as part of the impact-estimating process would need to be examined in the field by a registered bridge engineer to determine, more precisely, the recommendations. Bridge replacement can be avoided in some cases by "retrofit" activities that can be undertaken under traffic—avoiding the cost of replacement and the associated user costs while the construction is taking place[21]. If Texas were to permit LCV operations, bridge impacts would have to be addressed and remedies undertaken. But other large networks have faced this situation—the EU, for example—and the magnitude of the study bridge cost estimates on the key state corridors can be covered by fees collected over the life of the assets.

4.2 Pilot Study

The second study workshop took place in the summer of 2010 when the researchers presented their results to the TxDOT and industry panels. Participants agreed at the conclusion of that meeting to recommend a pilot study using a sample of operators willing to provide equipment and share data on operating over a selection of corridor routes. The precise routes would be determined as part of this study. The pilot study would analyze the adequacy of the geometric characteristics of the routes based on the dimensions of the LCVs and heavy single semi-trailer trucks selected for testing. Performance-based standards would be specified—including acceleration, braking, time of day, weather, and experience of driver—for LCV operations. The empirical data collected on actual operations would feed back into regulations and a fee-based cost allocation procedure. The study could collect cost information on trucking, pavements, and bridges to determine an equitable marginal cost for LCV operations. This step would allow industry representatives, highway engineers, and researchers to gain data on LCV operations while working closely with the trucking industry to devise a feasible, safe, and efficient way to allow LCVs to operate in Texas. The alternative, given the decreased funding for new lane-miles in Texas in conjunction with the projected increase in state population and economic activity, is definitely second-best.

[21] Caltrans reported such benefits for California, after inspecting state bridges failing theoretical seismic shocks related to the 1989 earthquake, and finding many could be strengthened rather than replaced.

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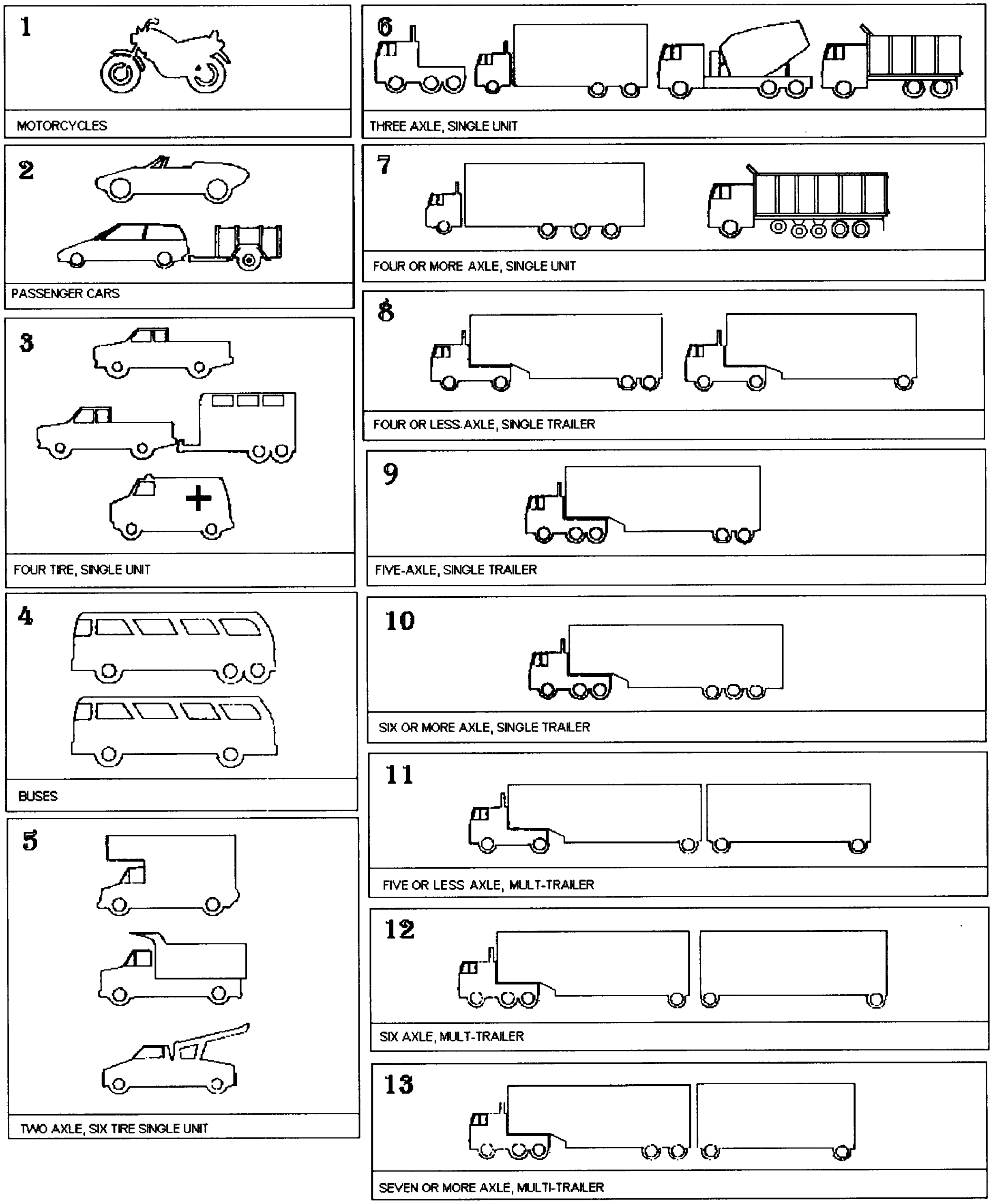
Appendix A: Federal Highway Administration Vehicle Classification System

All materials in this appendix come from the following source:
http://www.fhwa.dot.gov/policy/ohpi/vehclass.htm

Vehicle Class	*Description*
1	*Motorcycles*—All two or three-wheeled motorized vehicles. Typical vehicles in this category have saddle type seats and are steered by handlebars rather than steering wheels. This category includes motorcycles, motor scooters, mopeds, motor-powered bicycles, and three-wheel motorcycles.
2	*Passenger Cars*—All sedans, coupes, and station wagons manufactured primarily for the purpose of carrying passengers and including those passenger cars pulling recreational or other light trailers.
3	*Other Two-Axle, Four-Tire Single Unit Vehicles*—All two-axle, four-tire, vehicles, other than passenger cars. Included in this classification are pickups, panels, vans, and other vehicles such as campers, motor homes, ambulances, hearses, carryalls, and minibuses. Other two-axle, four-tire single-unit vehicles pulling recreational or other light trailers are included in this classification. Because automatic vehicle classifiers have difficulty distinguishing class 3 from class 2, these two classes may be combined into class 2.
4	*Buses*—All vehicles manufactured as traditional passenger-carrying buses with two axles and six tires or three or more axles. This category includes only traditional buses (including school buses) functioning as passenger-carrying vehicles. Modified buses should be considered to be a truck and should be appropriately classified.
5	*Two-Axle, Six-Tire, Single-Unit Trucks*—All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., with two axles and dual rear wheels.
6	*Three-Axle Single-Unit Trucks*—All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., with three axles.
7	*Four or More Axle Single-Unit Trucks*—All trucks on a single frame with four or more axles.
8	*Four or Fewer Axle Single-Trailer Trucks*—All vehicles with four or fewer axles consisting of two units, one of which is a tractor or straight truck power unit.

Vehicle Class	*Description*
9	*Five-Axle Single-Trailer Trucks*—All five-axle vehicles consisting of two units, one of which is a tractor or straight truck power unit. (Note: this type of truck is often referred to as “18-wheeler”).
10	*Six or More Axle Single-Trailer Trucks*—All vehicles with six or more axles consisting of two units, one of which is a tractor or straight truck power unit.
11	*Five or fewer Axle Multi-Trailer Trucks*—All vehicles with five or fewer axles consisting of three or more units, one of which is a tractor or straight truck power unit.
12	*Six-Axle Multi-Trailer Trucks*—All six-axle vehicles consisting of three or more units, one of which is a tractor or straight truck power unit.
13	*Seven or More Axle Multi-Trailer Trucks*—All vehicles with seven or more axles consisting of three or more units, one of which is a tractor or straight truck power unit.


1
MOTORCYCLES
2
PASSENGER CARS
3
FOUR TIRE, SINGLE UNIT
4
BUSES
5
TWO AXLE, SIX TIRE SINGLE UNIT
6
THREE AXLE, SINGLE UNIT
7
FOUR OR MORE AXLE, SINGLE UNIT
8
FOUR OR LESS AXLE, SINGLE TRAILER
9
FIVE-AXLE, SINGLE TRAILER
10
SIX OR MORE AXLE, SINGLE TRAILER
11
FIVE OR LESS AXLE, MULT-TRAILER
12
SIX AXLE, MULT-TRAILER
13
SEVEN OR MORE AXLE, MULTI-TRAILER

Appendix B: Pavement Materials' Properties in the Study Corridors

Pavement

Material	Elasticity Modulus (ksi)	Poisson's ratio
Hot mix asphalt	200	0.35
Concrete slab	3122	0.12
ACP	200	0.35
CMHB	200	0.35
Crumb Rubber	200	0.35
Superpave	200	0.35
Asphalt-stabilized Base	150	0.35
Flex Base	55	0.35
Lime Stab Flex Base	68	0.35
Crushed Limestone	30	0.3
Cement Treated Base	115	0.28
Lime treated subgrade	38	0.3
Permeable Friction Course	300	0.35
Stone Matrix Asphalt	200	0.35
Granular Base	25	0.45

Subgrade

Soil	Description	Resilient Modulus (ksi)	Poisson's ratio
CH	Clay of high plasticity, fat clay	12	0.2
MH	Silt of high plasticity, elastic silt	11.5	0.325
CL	Clay	17	0.2
ML	Silt	20	0.325
SW	Well graded sand, fine to course sand	28	0.15
SW-SC	Well graded sand, fine to course sand—clayey sand	25.5	
SW-SM	Well graded sand, fine to course sand—silty sand	28	
SP-SC	Poorly graded sand—clayey sand	25.5	
SP-SM	Poorly graded sand—silty sand	28	0.3
SC	Clayey sand	24	0.25
SM	Silty sand	32	0.325
GW	Well graded gravel, fine to course gravel	41	0.15
GP	Poorly graded gravel	38	0.15
GW-GC	Well graded gravel, fine to course gravel—clayey gravel	34.5	
GW-GM	Well graded gravel, fine to course gravel—silty gravel	38.5	
GP-GM	Poorly graded gravel—silty gravel	36	
GC	Clayey gravel	31	0.15
GM	Silty gravel	38.5	0.2
SP	Poorly graded sand	25	0.25
SM-SC	Silty sand—Clayey sand	28	0.3

Appendix C: Invited Attendees: Longer Combination Vehicles & Road Trains for Texas?

August 18, 2010
1:00–4:00 PM
Center for Transportation Research
1616 Guadalupe Street, Room 4.518

TxDOT Project Monitoring Committee:

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Peter Sweatman, Director, University of Michigan Transportation Research Institute, sweatman@umich.edu
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Invited Guests and Colleagues:

Ken Allen, Sr. Vice President, Supply Chain and Logistics, H-E-B, allen.ken@heb.com
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Angela Weissmann, transanalysis@att.net

Appendix D: Agenda: Longer Combination Vehicles & Road Trains for Texas?

August 18, 2010
1:00–4:00 PM
Center for Transportation Research
1616 Guadalupe Street, Room 4.518

AGENDA

1.	Introduction	*All*
2.	Structure of Project	*Michael Walton*
3.	Safety	*Kara Kockelman*
4.	Vehicle Types and Chosen Corridors	*Rob Harrison*
5.	Pavement Analysis: Methodology and Case Studies	*Angela Weissmann*
6.	Bridge Analysis: Methodology and Case Studies	*José Weissmann*
7.	Cost/Benefit Tradeoffs	*Rob Harrison* *José Weissmann*
8.	Recommendations • Pilot Test(s)	 *Research Team*
9.	Feedback/Final Report	*All*

Appendix E: Workshop Summary

An important aspect of this research was the interest and input from Texas trucking operators. To close out this project, a workshop was held to present the information to these operators and other interested parties for discussion. The entire research team, as well as members of the advisory, monitoring, and consulting teams, was present either at the meeting or through the webinar. The invited attendees and the workshop agenda are in Appendix C and Appendix D, respectively.

The workshop was chaired by Dr. C. M. Walton, who began with an overview of the project. He provided a background to the subject—which goes back over two decades—and then described the study objectives and the schedule of the research. Dr. Kara Kockelman presented her findings on the safety of LCVs based on the Large Truck Causation Study. An abstract of a paper[22] describing the analysis is provided in Appendix F. The results show that two-trailer LCVs appear to enjoy the lowest crash costs per mile driven. Attached to this report is a CD containing a complete set of the workshop materials. The results, as well as the literature review, suggest that LCVs can be recommended for use on limited access highways—such as interstates—for longer hauls with proper enforcement. Safety remains a paramount issue when more productive trucks are discussed and the analytical work was supplemented by calls to the FMCSA.

Robert Harrison then discussed the international perspective of LCVs. The allowable LCV types in Australia, Canada, and the European Union were shown as a guide to the LCVs chosen for the Texas analysis. The three types chosen for analysis were presented as well as the specific routes. The trio comprised two "true" LCVs and one "supersized" semi-trailer similar to the current Class 9 trucks plying the interstate system. The difference is twofold: 1) the vehicle has a third axle on the trailer (called a "tridem") and 2) it has a gross weight of 97,000 lb, which equates to 40 metric tons.[23] The increase in payload is over 20% higher than a typical 80,000 Class 9 rig. The types and routes were well received by those present as viable options for their companies.

Dr. Angela Weissmann from UTSA presented the pavement analysis and described how the data were collected and treated for use in the pavement analysis. The scenario she used assumed the following decreased:

- Tandem & single weight limits
- Single and tandem axle repetitions
- Number of overweight single axles
- Number of overweight tandem axles

The only numbers that increased were tridem axle weights and heavy tridem repetitions. Dr. Weissmann showed graphs of the changes in axle weights for each scenario compared to the

[22] A revised version of the paper entitled "Analysis of Large Truck Crash Severity Using Heteroskedastic Ordered Probit Models" by J. Lemp, K. Kockelman, and A. Unnikrishnan was submitted to the *Journal of Accident Analysis and Prevention* in June 2010.

[23] This vehicle is popular in Mexico, Canada, and the E.U. In Europe it is the "work horse" of the trucking system and is limited to 100 kph or 58 mph. The E.U. is currently sponsoring an evaluation of the costs and benefits of an LCV 60 metric ton design, as noted in the first-year report.

base case of not allowing LCVs. Finally, the differences in costs were shown for the different routes if LCVs were allowed. A fundamental result of this analysis was that much of the system is already capable of carrying the three types of LCVs chosen by study. One, the 90-kip double 53, was not a surprise because the weight carried by each axle is substantially less than a fully loaded 80,000 lb vehicle currently permitted to operate over the federal system. The evidence is that very little of the pavement system would need to be strengthened and that a modest increase in permit or registration fees would cover the marginal cost imposed by the vehicles. She acknowledged the limitations of her work and then provided some recommendations.

Dr. Jose Weissmann from UTSA presented the bridge analysis and gave an overview of Texas structures by showing the years they were built and the structural type. He pointed out that many interstate structures were completed during the 1970s and those bridges will be nearing the end of their design life—50 years—within a decade. An overview of the traditional method of bridge analysis was shown, together with an entirely new and different method that Dr. Weissmann and a colleague at UTSA created for this study. The traditional and newer method bridge analysis results for the three scenarios were then presented. The results showed the number of bridges needing replacing or strengthening, the square mileage this represented, and the total costs of the construction. He then gave his recommendations for future bridge work.

To close out, Mr. Harrison then discussed the idea of a cost/benefit tradeoff and indicated that a pilot study would be the next step in this research. Discussion was then open to the attendees of the meeting. During the time, the truck operators present all strongly recommended that a pilot test be run in the state to refine the cost allocation procedure, monitor safety, and measure the productivity of the various LCV and heavier truck types. TxDOT current funding limitations that preclude adding capacity to key freight corridors was also cited as a major reason for raising productivity and lower truck volumes. As mentioned, a copy of the workshop presentations is included in the CD attached to this report.

Appendix F: Analysis of Large Truck Crash Severity Using Heteroskedastic Ordered Probit Models

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ABSTRACT

Long-combination vehicles (LCVs) have significant potential to increase economic productivity for shippers and carriers by decreasing the number of truck trips, thus reducing costs. However, size and weight regulations, triggered by safety concerns and, in some cases, infrastructure investment concerns, have prevented large-scale adoption of such vehicles. Information on actual crash performance is needed. To this end, this work uses standard and heteroskedastic ordered probit models, along with the United States' Large Truck Crash Causation Study, General Estimates System, and Vehicle Inventory and Use Survey data sets, to study the impact of vehicle, occupant, driver, and environmental characteristics on injury outcomes for those involved in crashes with heavy-duty trucks. Results suggest that the likelihood of fatalities and severe injury is estimated to rise with the number of trailers, but fall with the truck length and gross vehicle weight rating (GVWR). While findings suggest that fatality likelihood for two-trailer LCVs is higher than that of single-trailer non-LCVs and other trucks, controlling for exposure risk suggest that total crash costs of LCVs are lower (per vehicle-mile traveled) than those of other trucks.

Keywords: Long-combination vehicles, crash severity, heavy duty truck safety, heteroskedasticity, ordered probit