Comprehensive Truck Size and Weight Limits Study

June 2015

Pavement Comparative Analysis Technical Report

U.S. Department of Transportation

Federal Highway Administration

EXECUTIVE SUMMARY

Background

This report documents analyses conducted as part of the U.S. Department of Transportation (USDOT) *2014 Comprehensive Truck Size and Weight Limits Study* (2014 CTSW Study). As required by Section 32801 of MAP-21 [Moving Ahead for Progress in the 21st Century Act (P.L. 112-141)], Volumes I and II of the 2014 CTSW Study have been designed to meet the following legislative requirements:

- Subsection 32801 (a)(1): Analyze accident frequency and evaluate factors related to accident risk of vehicles to conduct a crash-based analyses, using data from states and limited data from fleets;
- Subsection 32801 (a)(2): Evaluate the impacts to the infrastructure in each State including the cost and benefits of the impacts in dollars; the percentage of trucks operating in excess of the Federal size and weight limits; and the ability of each state to recover impact costs;
- Subsection 32801 (a)(3): Evaluate the frequency of violations in excess of the Federal size and weight law and regulations, the cost of the enforcement of the law and regulations, and the effectiveness of the enforcement methods; Delivery of effective enforcement programs;
- Subsection 32801 (a)(4): Assess the impacts that vehicles have on bridges, including the impacts resulting from the number of bridge loadings; and
- Subsections 32801 (a)(5) and (6): Compare and contrast the potential safety and infrastructure impacts of the current Federal law and regulations regarding truck size and weight limits in relation to six-axle and other alternative configurations of tractor-trailers; and where available, safety records of foreign nations with truck size and weight limits and tractor-trailer configurations that differ from the Federal law and regulations. As part of this component of the study, estimate:

  (A) the extent to which freight would likely be diverted from other surface transportation modes to principal arterial routes and National Highway System intermodal connectors if alternative truck configuration is allowed to operate and the effect that any such diversion would have on other modes of transportation;

  (B) the effect that any such diversion would have on public safety, infrastructure, cost responsibilities, fuel efficiency, freight transportation costs, and the environment;

  (C) the effect on the transportation network of the United States that allowing alternative truck configuration to operate would have; and

  (D) the extent to which allowing alternative truck configuration to operate would result in an increase or decrease in the total number of trucks operating on principal arterial routes and National Highway System intermodal connectors.

To conduct the study, the USDOT, in conjunction with a group of independent stakeholders, identified six different vehicle configurations involving six-axle and other alternative configurations of tractor-trailer as specified in Subsection 32801 (a)(5), to assess the likely results of allowing widespread alternative truck configurations to operate on different highway networks. The six vehicle configurations were then used to develop the analytical scenarios for each of the five comparative analyses mandated by MAP-21. The use of these scenarios for each of the analyses in turn enabled the consistent comparison of analytical results for each of the six vehicle configurations identified for the overall study.

The results of this *2014 Comprehensive Truck Size and Weight Limits Study* (2014 CTSW Study) study are presented in a series of technical reports. These include:

- *Volume I: Comprehensive Truck Size and Weight Limits Study – Technical Summary Report.* This document gives an overview of the legislation and the study project itself, provides background on the scenarios selected, explains the scope and general methodology used to obtain the results, and gives a summary of the findings.
- *Volume II: Comprehensive Truck Size and Weight Limits Study.* This volume comprises a set of the five comparative assessment documents that meet the technical requirements of the legislation as noted:
    - *Modal Shift Comparative Analysis* (Subsections 32801 (a)(5) and (6)).
    - *Pavement Comparative Analysis* (Section 32801 (a)(2)).
    - *Highway Safety and Truck Crash Comparative Analysis* (Subsection 32801 (a)(1)).
    - *Compliance Comparative Analysis* (Subsection 32801 (a)3)).
    - *Bridge Structure Comparative Analysis* (Subsection 32801 (a)(4)).

<u>Purpose of Pavement Technical Report</u>

The purpose of the *Volume II: Pavement Comparative Analysis* is to use the six vehicle configuration scenarios identified by USDOT to address two major items:

1) How the full spectrum of axle weights and types may change as a result of modal and configuration shifts in each scenario, and
2) How these changes affect pavement performance and expected pavement costs.

The first three scenarios asses the impacts of heavier tractor semitrailers than are generally allowed under current Federal law. Scenario 1 would allow five-axle (3-S2) tractor semitrailer to operate at a maximum gross vehicle weight (GVW) of 88,000 lb. while Scenarios 2 and 3 would allow six-axle (3-S3) semitrailers to operate at maximum GVWs of 91,000 lb. and 97,000 lb. respectively.

Scenarios 4, 5 and 6 examine vehicles that would serve primarily lower density cargoes commonly associated with those trucks that carry cargo from more than one shipper (known as less-than-truckload traffic or LTL). Scenario 4 examines twin trailer combination with 33-foot trailers (2-S1-2) with a maximum GVW of 80,000 lbs. Scenarios 5 and 6 examine triple trailer combinations with 28 or 28.5-foot trailers having maximum GVWs of 105,500 lb. (2-S1-2-2) and 129,000 lb. (3-S2-2-2), respectively.

At this point it is important to note that while the control double has an approved GVW of 80,000 pounds, the GVW used for the control double in the study is 71,700 pounds based on data collected from weigh-in motion (WIM)-equipped weight and inspection facilities and is a more accurate representation of actual vehicle weights than the STAA authorized GVW. Using the WIM-derived GVW also allows for a more accurate representation of the impacts generated through the six scenarios.

Table ES-1 on the following page depicts the vehicles assessed under each scenario as well as the current vehicle configurations from which most traffic would likely shift (the control vehicles).

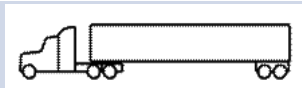
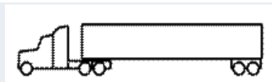
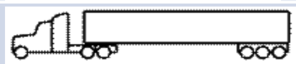
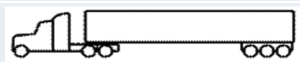
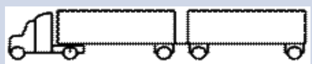
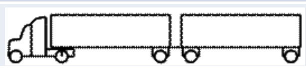
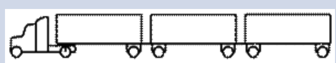
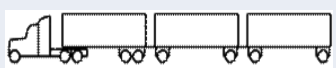
Approach and Methodology

The comparative analysis for the study consisted of a step-wise approach to determining the effects of the various truck traffic configuration scenarios, also accounting for potential freight traffic shifts to or from the rail mode as a result of each scenario, on performance and life-cycle costs. The process started with the selection of representative pavement sections (flexible and rigid along with their local materials and design inputs) within each of the four primary geographic locations in the United States.

The USDOT study team used the AASHTOWare *Pavement ME Design*® software to evaluate each of these sections to determine a base case of the expected pavement life prior to any needed rehabilitation under representative base case traffic conditions (e.g., representative of the mix of vehicle types and operating weights that might be expected). The *Pavement ME Design*® software tool uses a procedure that directly applies axle load spectra to calculate the amount of damage produced by the estimated range of traffic loads. The axle load spectra data were obtained from processing weigh-in-motion (WIM) data and include axle-load distributions (e.g., single, tandem, tridem, quads) and axle-load configurations (e.g., axle spacing and wheelbase). The study team performed an initial analysis of climatic variability within the vicinity of each geographic location to ensure that the sites selected were representative of typical weather effects and inclusive of typical subgrade soils in that area. The data from Long-Term Pavement Performance (LTPP) sections were used as a starting point for each sample section.

The analysis then considered the four pavement types (new flexible pavement, flexible overlay of existing flexible pavement, new jointed plain concrete pavement (JPCP), and composite (flexible overlay of existing JPCP) pavement) which represent the overwhelming majority of pavements used on the Interstate and National Highway System (NHS) in the United States. The Study does not attempt to evaluate the impact of the scenarios on the performance of overlaid pavements.

Table ES-1. Truck Configuration and Weight Scenarios Analyzed in the 2014 CTSW Study

Scenario	Configuration	Depiction of Vehicle	# Trailers or Semi-trailers	# Axles	Gross Vehicle Weight (pounds)	Roadway Networks
Control Single	5-axle vehicle tractor, 53 foot semitrailer (3-S2)		1	5	80,000	STAA[1] vehicle; has broad mobility rights on entire Interstate System and National Network including a significant portion of the NHS
1	5-axle vehicle tractor, 53 foot semitrailer (3-S2)		1	5	88,000	Same as Above
2	6-axle vehicle tractor, 53 foot semitrailer (3-S3)		1	6	91,000	Same as Above
3	6-axle vehicle tractor, 53 foot semitrailer (3-S3)		1	6	97,000	Same as Above
Control Double	Tractor plus two 28 or 28 ½ foot trailers (2-S1-2)		2	5	80,000 maximum allowable weight 71,700 actual weight used for analysis[2]	Same as Above
4	Tractor plus twin 33 foot trailers (2-1S-2)		2	5	80,000	Same as Above
5	Tractor plus three 28 or 28 ½ foot trailers (2-S1-2-2)		3	7	105,500	74,500 mile roadway system made up of the Interstate System, approved routes in 17 western states allowing triples under ISTEA Freeze and certain four-lane PAS roads on east coast[3]
6	Tractor plus three 28 or 28 ½ foot trailers (3-S2-2-2)		3	9	129,000	Same as Scenario 5[3]

[1] The STAA network is the National Network (NN) for the 3S-2 semitrailer (53') with an 80,000-lb. maximum GVW and the 2-S1-2 semitrailer/trailer (28.5') also with an 80,000 lbs. maximum GVW vehicles. The alternative truck configurations have the same access off the network as its control vehicle.

[2] The 80,000 pound weight reflects the applicable Federal gross vehicle weight limit; a 71,700 gross vehicle weight was used in the study based on empirical findings generated through an inspection of the weigh-in-motion data used in the study.

[3] The triple network is 74,454 miles, which includes the Interstate System, current Western States' triple network, and some four-lane highways (non-Interstate System) in the East. This network starts with the 2000 CTSW Study Triple Network and overlays the 2004 Western Uniformity Scenario Analysis, Triple Network in the Western States. There had been substantial stakeholder input on networks used in these previous USDOT studies and use of those provides a degree of consistency with the earlier studies. The triple configurations would have very limited access off this 74,454 mile network to reach terminals that are immediately adjacent to the triple network. It is assumed that the triple configurations would be used in LTL line-haul operations (terminal to terminal). The triple configurations would not have the same off network access as its control vehicle–2S-1-2, semitrailer/trailer (28.5'), 80,000 lbs. GVW. The 74,454 mile triple network includes: 23,993 mile network in the Western States (per the 2004 Western Uniformity Scenario Analysis, Triple Network), 50,461 miles in the Eastern States, and mileage in Western States that was not on the 2004 Western Uniformity Scenario Analysis, Triple Network but was in the 2000 CTSW Study, Triple Network (per the 2000 CTSW Study, Triple Network).

The data used in the pavement analyses of this Study came from several FHWA sources, namely the Highway Performance Monitoring System (HPMS), vehicle classification and weight data reported by the States to FHWA, the Long-Term Pavement Performance (LTPP) database, and from MEPDG calibration data from four State departments of transportation (DOT). The models used for the analysis are those that are in Version 2.0 of the *Pavement ME Design*® software.

After compiling the input data required for each of the sections, the USDOT study team analyzed the base case traffic volumes were analyzed for each geographic location and pavement type and ran a set of analyses for each of the six modal shift scenarios in order to estimate the change in initial service interval.

A number of key assumptions and limitations apply to this study. The main limitation is that this Study considers only initial service lives predicted by *Pavement ME Design*® software, version 2.0, and only for the distresses and pavement types that the software could suitably model. Deterioration caused by the interaction of loads and construction deficiencies or decreased materials durability (e.g., deterioration of HMA transverse cracks caused by low temperatures, deterioration of concrete pavement “D” cracking) are outside the scope of this study, although it should be recognized that these elements can significantly impact the performance of pavements. In addition, the impacts of truck tire types (e.g., wide-based radial) and tire-pavement interaction (e.g., braking, torqueing, and other physical responses) are not considered.

Pavement distress levels and ride quality were reported based on average predicted performance.

Truck Size and Weight Scenarios

The first three scenarios assess tractor semitrailers that are heavier than generally allowed under currently Federal law. Scenario 1 assesses a five-axle (3-S2) tractor-semitrailer operating at a GVW of 88,000 pound, while Scenarios 2 and 3 assess six-axle (3-S3) tractor semitrailers operating at GVWs of 91,000 and 97,000 pounds, respectively. The control vehicle for these scenario vehicles is the five-axle tractor-semitrailer with a maximum GVW of 80,000 pounds. This is the most common vehicle configuration used in long-haul over-the-road operations and carries the same kinds of commodities expected to be carried in the scenario vehicles.

Scenarios 4, 5, and 6 examine vehicles that would serve primarily less-than-truckload (LTL) traffic that currently is carried predominantly in five-axle (3-S2) tractor-semitrailers and five-axle (2-S1-2) twin trailer combinations with 28 or 28.5-foot trailers and a maximum GVW of 80,000 pounds. Scenario 4 examines a five-axle (2-S1-2) double trailer combination with 33-foot trailers with a maximum GVW of 80,000 pounds. Scenarios 5 and 6 examine triple trailer combinations with 28.5-foot trailer lengths and maximum GVWs of 105,500 (2-S1-2-2) and 129,000 (3-S2-2-2) pounds, respectively. The five-axle twin trailer with 28.5-foot trailers is the control vehicle for Scenarios 4, 5, and 6 since it operates in much the same way as the scenario vehicles are expected to operate.

At this point it is important to note that while the applicable Federal gross vehicle weight limit is 80,000 lbs., a GVW of 71,700 lbs. was used for the control double configuration in the study based on empirical findings generated through an inspection of the weigh-in-motion data used in the study.

Analysis of the relative impacts of one group of vehicles compared with another at the national system level requires some simplification of assumptions about the vehicles themselves. For each scenario in this study, freight was shifted either from one vehicle to another, or to a vehicle of the same type but with a different weight. The approach used in the study assumed that both the before and after vehicles in each scenario had the same temporal use patterns, the same tire and suspension characteristics, were traveling at the same speeds, and behaved in ways that were similar; thus, the only variable considered for pavement analysis was the change in axle weights and types.

The scenarios assessed in the Study used the Interstate and National Highway System (NHS) roadways. More than 80% of total annual truck miles travelled occurs on the NHS. There are more than four million center line miles of public roadways in the United States with most of those miles located off of the NHS. There is generally little quantitative information available regarding travel, by facility, occurring on this non-NHS roadway network and on how pavements on the local road system are designed, built, and maintained. Except in rare cases, there is minimal to no history of travel or pavement characteristic data on local roads. These data limitations have made it prohibitive to perform an accurate and representative study on the impacts of loading scenarios on local roads at this time. The lack of pavement structure characteristics, pavement surface type and typical travel levels for local system roadways yields it impossible to develop sampling based approaches that would produce results supported with adequate statistical confidence. A review of the low-volume NHS sample section results very generally point in the direction of impacts that scenario configurations may have on local roads but, it must be understood, local roads are, overall, built to lower design standards than roadways on the higher functionally classified roadway networks. It is also understood that daily travel demand levels and daily truck travel on local roads are typically low, hence the lower design standards they were built to.

Desk Scan

The purpose of the *Volume II: Pavement Comparative Analysis* desk scan was to review the most relevant previous studies comparing pavement impacts from vehicle use. These studies included international, national and state cost allocation and truck size and weight studies as well as any other studies that included estimates of vehicle-induced pavement costs on either an absolute or relative basis. The USDOT study team also searched for pavement analyses or design studies intended to assist in the selection of an appropriate analytical tool and support application of the selected tool – the *Pavement ME Design*® software.

The principal objective of the search was to gain a thorough understanding of the current state of research and practice concerning pavement performance and cost analysis related to heavy vehicle use. The literature search included a variety of information sources: (1) engineering and scientific periodicals and journals; (2) conference proceedings; (3) Federal, State, international, and university reports that show up in library search engines (such as Compendex) based on key word searches; and (4) studies identified during the May 29, 2013, stakeholder public hearing for the study or by USDOT officials.

The results of the Desk Scan can be found in **Appendix A**.

Summary of Results

This *Volume II: Pavement Comparative Analysis* analyzed the effect of overweight axles in current operations, defining overweight as single axles weighing more than 20,500 lbs. and tandem axles weighing more than 35,000 lbs. to be consistent with the axle weight group boundaries used in the vehicle weight analysis. Initial service intervals were found to increase significantly for both flexible and rigid pavement sections (except in the case of one rigid pavement section that did not reach the end of its initial service interval during the analysis period). Flexible pavement initial service intervals increased by between and 19 percent and 34 percent, and rigid pavement initial service intervals increased by between 0 percent and 10 percent when overweight axles were removed from the traffic mix.

The estimated impact of the truck size and weight scenarios varies among the scenarios as well as the pavement type and service conditions considered in the analysis. Scenario 1 (allowing 3-S2 tractor semitrailers to operate at a GVW of 88,000 lbs.) results in a heavier array of tandem axle loads, while by contrast, Scenarios 2 and 3 (allowing 3-S3 semitrailers to operate at GVWs of 91,000 and 97,000 lbs., respectively) transferred loads from some of the heavier tandem axles to tridem axles. Scenario 4 (allowing 2-S1-2 twin trailer combinations with 33-foot trailers) results in an increase in the weight distributions of single-load axles. Scenario 5 (allowing 2-S1-2-2 triple trailer combinations with GVWs of 105,500 lbs.) transfers some tandem axle loads to lighter single axle vehicles. Scenario 6 (allowing 3-S2-2-2 triple trailer combinations with two extra axles with GVWs of 129,000 lbs.) results in lower tandem axle weights as well as a similar shift in freight movements from tandems to lighter single-axle vehicles.

Average impacts of each scenario in terms of both time to first rehabilitation and life cycle cost are summarized in **Table ES-2**. Notably, flexible pavements exhibited more accelerated deterioration with Scenarios 1 and 4, whereas rigid pavements were more negatively impacted by Scenarios 4, 5, and 6.

The more significant impacts are predicted to occur on lower volume facilities, specifically on low-volume other NHS arterials, which are typically constructed with thinner cross-sections. The estimated impacts of the scenarios are relatively minor for those thicker pavement sections that were built to handle higher truck volumes. The range of impacts for each scenario results from varying pavement conditions, climatic conditions, and highway types.

The life cycle cost (LCC) implications of the scenarios also varied. **Table ES-2** summarizes these differences averaged over all pavement types, geographic locations, and types of facilities under two alternative discount rates, a rate for estimating conservative or lower bound values and a typical rate to estimate upper bound values. The 1.9% discount rate was provided in guidance to federal agencies issued in 2014 by **OMB** in the annual update to **Circular No. A-94** and 7.0% provided in Circular A-94 that was used in the FHWA **2013 Status of the Nation's Highways, Bridges, and Transit: Conditions & Performance Report**. Note that the table shows the range of the results of applying each discount rate. On average, Scenario 4 resulted in the largest LCC overall increase of 1.8% to 2.7% from the base case, whereas Scenario 2 and 3 resulted in 2.4% to 4.2% decreases in predicted LCC from the base case. Scenarios 1, 5, and 6 showed only slight increases in LCC. LCC is defined herein as agency cost for pavement rehabilitation (e.g., overlays, retexturing) over a 50-year analysis period, not including initial construction costs or user costs.

Study results, being national in nature, must be reviewed with the understanding that truck travel demand across the country is not evenly distributed. Impacts on pavements will vary by region, not only due to stress applied to pavement structures due to differences in geotechnical and climatic conditions and situations, but due to the type of truck travel demand occurring in various regions of the country. Recently, the emergence of a strategic change in the US energy model saw new and significant pressures put on transportation modes in very specific regions of the country and on very specific travel corridors. The travel demand and impacts assessed for each of the scenarios in this Study must be reviewed in this light.

Table ES-2. Impacts of Study Scenario (Compared to Base Case) on Pavement Performance and Costs

Scenario		Weighted Average Change in Service Intervals	Weighted Average Change in Life Cycle Costs
1	88,000-lb 5-Axle Single-Semitrailer Combinations	- 0.3%	+0.4% to +0.7%
2	91,000-lb 6-Axle Single-Semitrailer Combinations	+2.7%	-2.4% to -4.2%
3	97,000-lb 6-Axle Single-Semitrailer Combinations	+2.7%	-2.6% to -4.1%
4	5-Axle Double-Trailer Combinations with 33-Foot Trailers	-1.6%	+1.8% to +2.7%
5	105,500-lb 7-Axle Triple-Trailer Combinations	0.0%	+0.1% to +0.2%
6	129,000-lb 9-Axle Triple-Trailer Combinations	-0.1%	+0.1% to +0.2%

Note: Individual pavement sections were weighted based on the number of lane-miles of pavement of each type, thickness range, and highway type.

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LIST OF ACRONYMS

Acronym	Definition
AASHTO	Association of State Highway and Transportation Officials
AC	asphalt concrete
ADT	average daily traffic
ADTT	average daily truck traffic
CTSW	comprehensive truck size and weight limits
ESALs	equivalent single axle loads
FAF	Freight Analysis Framework
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
GCW	gross combined weight
GVW	gross vehicle weight
HCA	highway cost allocation
HMA	hot mix asphalt
HPMS	Highway Performance Monitoring System
ITIC	Intermodal Transportation and Inventory Cost Model
IRI	International Roughness Index
JPCP	jointed plain concrete pavement
LCC	life cycle cost
LCV	longer combination vehicles
LEF	load equivalence factors
LTL	less than truckload
LTPP	Long-Term Pavement Performance
MEPDG	Mechanistic Empirical Pavement Design Guide
NAS	National Academy of Sciences
NATS	National Truck Stop Survey
NHTSA	National Highway Traffic Safety Administration
OGW	operating gross weight
ORNL	Oak Ridge National Laboratories
PCC	Portland Cement Concrete
PCE	passenger car equivalents
STB	Surface Transportation Board
STCC	Standard Transportation Commodity Code
USDOT	US Department of Transportation
VMT	vehicle miles traveled
VIUS	Vehicle Inventory and Use Survey
WIM	weigh-in-motion

CHAPTER 1 – PAVEMENT COMPARATIVE ANALYSIS

1.1 Background

MAP-21 directs the Secretary of Transportation, in consultation with State and other Federal agencies, to conduct a series of analyses assessing the impacts from trucks operating at or within current Federal size and weight regulations as compared to the impacts from trucks operating above those limits with a particular focus on impacts to:

- Highway safety and truck crash rates;
- Infrastructure (pavement) service life;
- Highway bridge performance; and
- Delivery of effective enforcement programs.

The United Stated Department of Transportation (USDOT), in conjunction with a group of independent stakeholders, identified six different vehicle configuration scenarios, each involving one of the alternative truck configurations, to assess the likely results of allowing widespread alternative truck configurations to operate on different highway networks.

The results of this *2014 Comprehensive Truck Size and Weight Limits Study* (2014 CTSW Study) study are presented in a series of technical reports. These include:

- *Volume I: Comprehensive Truck Size and Weight Limits Study – Technical Summary Report.* This document gives an overview of the legislation and the study project iteself, provides background on the scenarios selected, explains the scope and general methodology used to obtain the results, and gives a summary of the findings.
- *Volume II: Comprehensive Truck Size and Weight Limits Stud.* This volume comprises a set of five comparative assessment documents that meet the technical requirements of the legislation:
  - *Modal Shift Comparative Analysis*
  - *Pavement Comparative Analysis,*
  - *Highway Safety and Truck Crash Comparative Analysis,*
  - *Compliance Comparative Analysis, and*
  - *Bridge Structure Comparative Analysis*

This *Volume II: Pavement Comparative Analysis* presents the analysis of the six alternative truck size and weight configurations (scenarios) selected for study and describes in detail the approach, data, models, limitations, and assumptions underlying estimates of potential pavement impacts associated with the six scenarios.

1.2 Introduction

The pavement comparative analysis for the 2014 CTSW Study consisted of a step-wise approach to determining the effects of the various truck traffic configuration scenarios on performance and life-cycle costs. The process started with the selection of representative pavement sections (flexible and rigid along with their local materials and design inputs) within each of the four primary geographic locations in the United States. The AASHTOWare *Pavement ME Design®* software (or MEPDG) was used to evaluate each of these sections to determine a base case of the expected pavement life prior to any needed rehabilitation under representative base case traffic conditions (e.g., representative of the mix of vehicle types and operating weights that might be expected based on compilation and analysis of large quantities of WIM data). An initial analysis of climatic variability within the vicinity of each geographic location was performed to ensure that the sites selected were representative of typical weather effects and inclusive of typical subgrade soils in that area. The data from Long-Term Pavement Performance (LTPP) sections were used as a starting point for each sample section.

The analysis then considered the four pavement types (new flexible pavement, flexible overlay of existing flexible pavement, new jointed plain concrete pavement (JPCP), and composite (flexible overlay of existing JPCP) pavement) that represent the overwhelming majority of pavements used on the Interstate and National Highway System (NHS) in the United States. The basic premise was that the analysis should isolate the impacts of traffic shifts while holding other parameters constant. In order to achieve this goal, the baseline pavement sections were based on the following criteria: 1) use actual current traffic characteristics; 2) use sections with modern designs and materials as were constructed over the past two or three decades (as close to actual site sections as possible); and 3) use the subgrade properties on site. The flexible and rigid pavement surface layer thicknesses were selected from the ranges reported in the 2012 HPMS database for Interstates and other NHS arterial roadways in each of the four geographic locations.

One limitation of the current version of *Pavement ME Design®* software is that it cannot suitably evaluate the impact of traffic loadings on the predicted service life of either asphalt overlays of existing flexible pavements or asphalt overlays of existing JPCP pavements because the current reflection cracking models are totally empirical, cannot predict level of severity, and are not related to traffic loadings similar to other distresses in the *Pavement ME Design®* software. For this reason, this study does not attempt to evaluate the impact of the scenarios on the performance of overlaid pavements.

After compiling the input data required for each of the sections, the USDOT study team analyzed base case traffic volumes for each geographic location and pavement type. Another set of analyses were then run for each of the six modal shift scenarios in order to estimate the change in initial service interval.

The multiple runs for each sample section enabled the study team to evaluate changes in the initial pavement service interval (defined as one or more condition measures at a level that would trigger rehabilitation) as a result of changes in truck travel associated with each of the six scenarios.

A life cycle cost analysis was then performed for the base case and each scenario on each sample section.

The computation of expected cost impacts included only the present value of a typical highway agency pavement rehabilitation strategy that represents the agency costs over a 50-year analysis period. These costs included HMA overlays for the rehabilitation of flexible pavement and diamond grinding and HMA overlays of rigid pavement. User costs related to pavement roughness, work zones and lane closures, and traffic delays were not considered in this effort out of concern that the large number of assumptions required to include them would confound that analysis. These costs, however, can be expected to have a significant impact on the traveling public and freight users of the Nation's transportation system.

1.3 Key Assumptions

The models used in the study for the pavement analysis considered an array of factors in the prediction of pavement service lives, but did not consider all possible factors. One key assumption is that a substantial amount of confidence can be placed in the relative initial service intervals predicted for each traffic scenario, while all other factors are held constant. It is important to recognize that impacts related to both materials- and post-construction durability are assumed to impact the initial service intervals of pavements under each size and weight scenario equally.

Since pavement overlays were not modeled, it was implicitly assumed that modeling new pavements adequately covers the effects of changing vehicle usage patterns on rehabilitated pavements. At the same time, however, the cost analysis assumes that relative levels of use have no effect on the performance of rehabilitated pavements. The inconsistency in these two assumptions results from the current inability of the pavement models to analyze the performance of pavement overlays.

Similarly, it was assumed that temporal travel distributions, tire pressures, vehicle widths, or power-unit wheelbases (vehicle-related inputs to *Pavement ME Design*® software) will not change under any of the scenarios. While vehicle speeds may change incrementally on some highway segments, it was assumed that the changes are not significant enough to impact rutting or any other pavement distresses. It was further assumed that damage (such as scuffing) caused by increases in lateral pavement friction from tridems and other multiple-axle sets will not significantly impact pavement costs.

As in all other parts of this study, the pavement analysis assumed that the limited available data on vehicle characteristics and travel patterns, and the predicted changes in those characteristics and travel patterns as estimated in the *Volume II: Modal Shift Comparative Analysis*, adequately describe the before-and-after conditions associated with each size and weight scenario.

CHAPTER 2 – SELECTION OF PAVEMENT SECTIONS

2.1 Key Data and Models Used in the Analysis

Performance Distress Target Inputs

The pavement distress prediction targets were initially selected as the threshold values presented in the *AASHTO MEPDG-1 Manual of Practice* (2008). A 90 percent reliability level was used in the preliminary analyses in this phase of the study to compare distress predictions in the four geographic locations as an initial check on prediction data quality. The initial IRI of 65 in/mile and terminal International Roughness Index (IRI) of 160 in/mile were used for both pavement types. The damage thresholds for both the flexible and rigid pavements considered in the study are presented in **Table 1**.

Table 1: Summary of Distress Type Target Values

<table>
<tr><th>Flexible Pavement</th><th>Rigid Pavement</th></tr>
<tr><td>Total Rutting = 0.75 inch</td><td>Mean Joint Faulting = 0.12 inch</td></tr>
<tr><td>Fatigue Cracking = 25%</td><td rowspan="2">JPCP transverse cracking (percent slabs) = 10%</td></tr>
<tr><td>Thermal Cracking = 1000 ft./mile</td></tr>
<tr><td colspan="2">Initial IRI = 65 in/mile
Terminal IRI = 160 in/mile</td></tr>
</table>

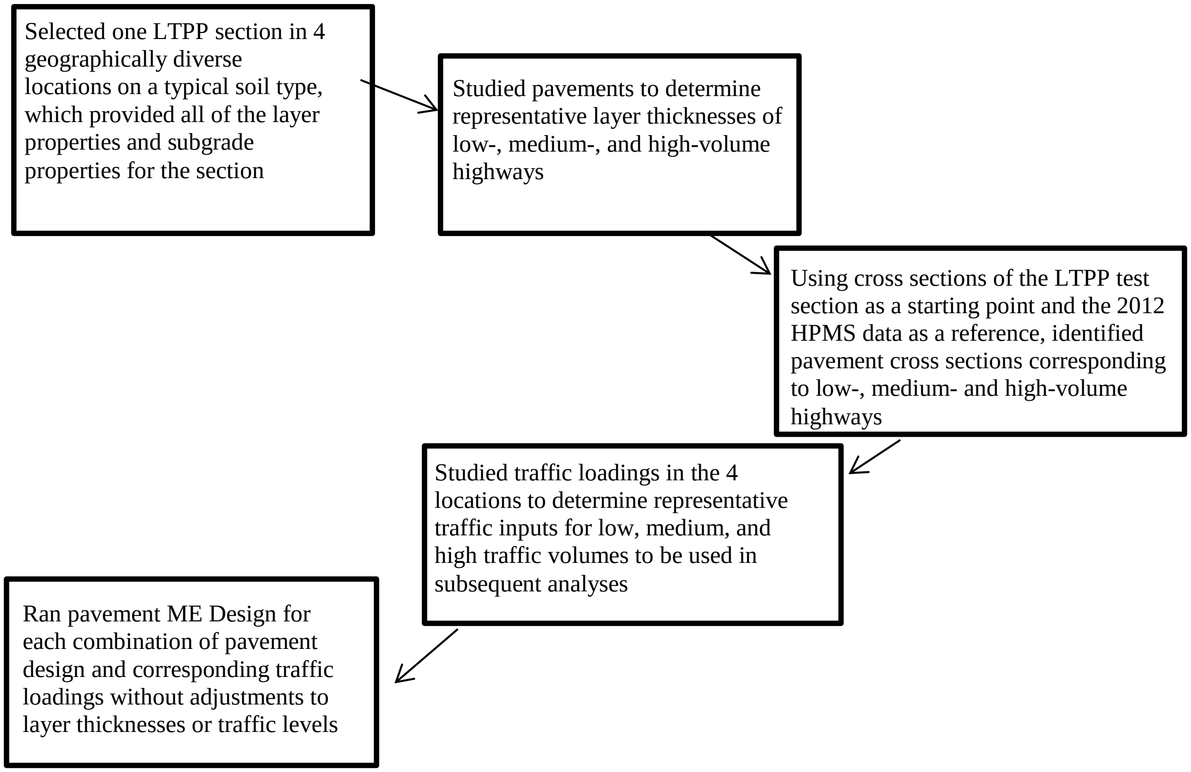
Note: All of the analyses were conducted with a pavement construction date of June 2015.

The **Figure 1** provides an overview of the general approach used in each geographic location for each pavement type:

Selection of Pavement Sections

The approach presented in **Figure 1** is intended to allow for reasonable extrapolation of these representative sections to the Nation’s roadways with increased confidence that the representative traffic and representative pavement designs were matched independently. The study used representative pavement designs for low, medium, and high volumes on highways upon which the majority of truck traffic will be traveling.

Figure 1: Overview of Sample Pavement Section Selection Approach



The LTPP database, along with the sites included in the field calibration studies used in the four States representing the geographically diverse locations in the United States, were reviewed to identify flexible and rigid pavement sections. These sections were identified as being in the general areas of Columbus, Ohio; Jackson, Mississippi; Phoenix, Arizona; and both Wasatch County and Nephi, Utah. A listing of the original sections that formed the baseline for all further analysis for the full matrix is summarized in **Table 2**. These sites were selected only to serve as a starting point in determining the impact of different truck traffic levels.

Table 2: Sample Flexible and Rigid Pavement Sections from LTPP Database

Location Nearest to Site	Roadway Designation	Pavement Type	LTPP Site Designation
Phoenix, Arizona	Interstate I-10	Flexible	04-0260
Panola County, Mississippi	Interstate I-55	Flexible	28-0501, control section (virgin mix, no overlay)
Columbus, Ohio	State Route 23	Flexible	39-0160
Wasatch County, Utah	State Route 35	Flexible	49-0804
Phoenix, Arizona	Interstate I-10	Rigid	04-0265
Pickens County, Georgia	State Route 5	Rigid	13-3007
Columbus, Ohio	State Route 23	Rigid	39-0263
Nephi, Utah	Interstate I-15	Rigid	49-3011

Using the LTPP test section cross sections as a starting point and the FHWA 2012 HPMS data as a reference, the pavement cross sections corresponding to low-, medium-, and high-volume highways were identified. In doing so, a number of alternative cross sections were explored in order to ensure that the selected cross sections encompass the range of pavements on the National Highway System (NHS) from a standpoint of both layer thicknesses and expected performance.

All of the *Pavement ME Design®* software inputs for these sections were provided by Applied Research Associates (ARA), including the materials information from the LTPP database and local State calibration factors derived during implementation of the *Pavement ME Design®* software in those States. As noted above, the inputs from these LTPP sites included in **Table 2** were only used as a starting point for the detailed analyses.

Analysis of Truck Traffic Volume Adjustments

To estimate truck traffic levels, the States were grouped by their geographic location. The average daily truck traffic (ADTT) estimates were calculated for both Interstate and Other NHS arterial routes for each State as well as for the collection of all States geographically grouped near one of the four analysis locations. These estimates were based on a combination of travel estimates developed for this study and estimates of highway miles by State and functional class published in the FHWA 2011 HM-20 table.

Interstate Truck Traffic Volumes

The Interstate truck traffic values (in trucks per day) were defined by the following procedure:

a. Medium-volume (MV) Interstate ADTT levels were derived from the average of all Interstate highways within a grouping of States near each of the four locations; and,

b. High-volume (HV) Interstate ADTT levels were derived from the highest observed State average within a grouping of States near each of the four locations.

Clearly, using State averages results in high-volume sections that together do not cover the full range of truck travel variation within a given State or grouping of States near each of the four locations, but the medium-volume ranges represent the mean values of truck travel, given their derivation. A better approach might be to gather truck volume data from a large number of highway sections, but such data were not available for this study, so the State average variation approach served as a plausible substitute.

Other NHS Arterial Truck Traffic Volumes

The USDOT study team derived the low-volume other NHS arterial ADTT values from the average of the combined Other NHS arterial highways within a grouping of States near each of the four locations.

Other traffic inputs were determined by averaging travel data within a grouping of States near each of the four locations, including the vehicle class, axle load weight distributions, axles per truck, etc. A more comprehensive table of the average truck traffic levels organized by the geographic grouping and State, from which **Table 3** was derived, is included in **Appendix G**.

Table 3: Truck Traffic Levels (Average Daily Truck Travel) by Geographic Grouping and Pavement Section

Geographic Location	High Volume Interstate (ADTT)	Medium Volume Interstate (ADTT)	Low Volume Other NHS (ADTT)
Location #1	11,338	7,206	782
Location #2	13,562	7,419	895
Location #3	9,824	3,838	481
Location #4	9,159	7,489	1,391

Vehicle class and axle weight distributions used in the pavement analysis and throughout the study were derived from weigh-in-motion (WIM) and vehicle classification data from two sources: FHWA's Office of Highway Policy Information and the LTPP WIM data provided by FHWA's Office of Infrastructure Research and Development as described in section 1.3 of the *Task III: Data Acquisition and Technical Analysis Report*. The Study's VMT estimates by vehicle class, operating weight distributions by vehicle class, and axle weight distributions by operating weight group and vehicle class for each geographic location were condensed into the format required for direct use by *Pavement ME Design*® software.

Establishment of Distress Prediction Levels for Sample Pavement Sections

In selecting the performance criteria for use in the *Pavement ME Design*® software, all predictions were analyzed at the mean predicted level over a 50-year analysis period. **Table 4** presents the levels of distress prediction defined as threshold criteria for the assessment of all pavement types.

Table 4: Summary of Distress Prediction Levels Identified in Study for the Triggering of Pavement Repairs

Pavement Type	Distress Type and Prediction Level		
New Flexible	0.4 inch total rutting	7.5% bottom-up fatigue cracking	IRI = 160 in/mile
New Rigid (JPCP)	0.15 inch faulting	7.5% slabs transverse cracked	IRI = 160 in/mile

These are similar to those values used by highway agencies for determining when major rehabilitation (e.g., overlays, retexturing, reconstruction, etc.) is needed. Agencies also use criteria that are structure-related or use a combination of criteria related to safety, comfort, or ride-ability.

In order to conduct a pavement structural analysis of this breadth, a number of engineering assumptions and an analytical scope were defined, and these are presented in **Table 5**. Other inputs such as the depth to groundwater table and soil type and properties are included in **Appendix D**.

Table 5: List of Assumptions for Expanding Pavement Structural Analysis

Analysis Factor	Assumed Value or Criteria	Justification
Analysis Period	50 years	Set at 50 to accommodate a wide range of pavement service lives and rehabilitation
Traffic Growth Rate	2.0% linear	Literature review indicated that 2.0% is on the low end of observed truck growth on Interstates and intent was to limit bias in results by use of too high a growth rate
Unstabilized aggregate base thickness	Varied by each geographic location	Based on LTPP database, extracted the range of thickness values for granular bases under flexible and rigid pavements for a group of States within each geographic location. Selected the mean thickness and/or varied per highest percentile value and lowest percentile value
Percent trucks in design lane Percent trucks in design direction	80% (HV); 85% (MV, LV) 50% (all)	Varied based on roadway functional class per the *MEPDG Manual of Practice* (2008)
Number of lanes	3 per direction (HV) 2 per direction (LV, MV)	Based on typical Interstate sections on high volume and other highway segments
Operational speed	55 mph	Based on the default speed in the AASHTO *Pavement ME Design*® software

LV = low-volume Other NHS. MV = medium-volume Interstate. HV = high-volume Interstate.

Selection of New Flexible Pavement Sections

Each of the new flexible pavement sections selected for this study were analyzed at the design targets recommended in the *MEPDG Manual of Practice*, as described above. The mixture properties and other climatic and material properties (e.g., for the base and subgrade) were based on the original LTPP calibration data files for each of the four States. The distress and IRI calibration coefficients were locally calibrated in the States from which the initial structures were selected. The truck volumes, vehicle class distributions, and axle weight distributions were applied per the method previously described.

Data from the 2012 Highway Performance Monitoring System (HPMS) were consulted to determine the appropriate pavement HMA and PCC surface thickness based on roadway type (Interstate vs. other NHS arterial) for each geographic location. The flexible pavement cross-sections to be used in the structural analyses are presented in **Table 6**. The full dataset from the HPMS is presented in **Appendix H**.

Table 6: New Flexible Pavement Structure Characteristics

Geographic Location	Roadway Section	Flexible Pavement Layer, Type, and Thickness	Base Layer Type and Thickness	Subgrade Type
Location #1	Low volume	Layer 1: 2.0-inch PG64-28 Layer 2: 2.0-inch PG64-22 Layer 3: 2.0-inch PG64-22	Unstabilized A-1-a aggregate base 8.0-inch	A-6
	Medium volume	Layer 1: 2.0-inch PG64-28 Layer 2: 2.5-inch PG64-22 Layer 3: 4.0-inch PG64-22	Unstabilized A-1-a aggregate base 10.0-inch	
	High volume	Layer 1: 2.0-inch PG64-28 Layer 2: 4.0-inch PG64-22 Layer 3: 6.0-inch PG64-22	Unstabilized A-1-a aggregate base 12.0-inch	
Location #2	Low volume	Layer 1: 1.0-inch PG70-22 Layer 2: 4.0-inch PG70-22	Unstabilized A-1-b aggregate base 9.0-inch	A-6
	Medium volume	Layer 1: 3.0-inch PG70-22 Layer 2: 4.0-inch PG70-22		
	High volume	Layer 1: 4.0-inch PG70-22 Layer 2: 7.0-inch PG70-22		
Location #3	Low volume	Layer 1: 5.0-inch PG58-34	Unstabilized A-1-b aggregate base 12.0-inch	A-2-4
	Medium volume	Layer 1: 7.0-inch PG58-34		
	High volume	Layer 1: 9.0-inch PG58-34		
Location #4	Low volume	Layer 1: 1.0-inch PG76-16 Layer 2: 5.0-inch PG70-22	Unstabilized A-1-a aggregate base 8.0-inch	A-4
	Medium volume	Layer 1: 1.0-inch PG76-16 Layer 2: 7.0-inch PG70-22		
	High volume	Layer 1: 1.0-inch PG76-16 Layer 2: 10.0-inch PG70-22		

Schematic diagrams that present the information on flexible pavement structural sections in **Table 6** are included in **Appendix I** of the report.

Selection of New Rigid Pavement Sections

The four new rigid pavement sections were analyzed at the design targets recommended in the *MEPDG Manual of Practice* (2008). The mixture properties and other climatic and material properties (e.g., for the base and subgrade) were carried forward from the original LTPP calibration files for each of the four States.

The same process of consulting the HPMS data was applied for the selection of rigid pavement surface thicknesses, presented in **Table 7**.

Table 7: Rigid Pavement Structure Characteristics

Geographic Location	Roadway Section	Rigid Pavement Thickness and Design Features	Base Layer Type and Thickness	Subgrade Type
Location #1	Low volume	Layer 1: 8.0-inch PCC 15-ft joint spacing 1.25-inch dowel diameter	Unstabilized A-1-a aggregate base 6.0-inch	A-6
	Medium volume	Layer 1: 10.0-inch PCC 15-ft joint spacing 1.5-inch dowel diameter		
	High volume	Layer 1: 12.0-inch PCC 15-ft joint spacing 1.5-inch dowel diameter		
Location #2	Low volume	Layer 1: 8.0-inch PCC 15-ft joint spacing 1.0-inch dowel diameter	Unstabilized A-1-b aggregate base 9.0-inch	A-6
	Medium volume	Layer 1: 10.0-inch PCC 15-ft joint spacing 1.25-inch dowel diameter		
	High volume	Layer 1: 12.0-inch PCC 15-ft joint spacing 1.5-inch dowel diameter		
Location #3	Low volume	Layer 1: 8.0-inch PCC 15-ft joint spacing 1.25-inch dowel diameter	Unstabilized A-1-b aggregate base 6.0-inch *placed over an* Unstabilized A-1-b aggregate subbase 6.0-inch	A-2-4
	Medium volume	Layer 1: 10.0-inch PCC 15-ft joint spacing 1.5-inch dowel diameter		
	High volume	Layer 1: 12.0-inch PCC 15-ft joint spacing 1.5-inch dowel diameter		
Location #4	Low volume	Layer 1: 8.0-inch PCC 15-ft joint spacing 1.25-inch dowel diameter	Unstabilized A-2-4 aggregate base 6.5-inch	A-4
	Medium volume	Layer 1: 10.0-inch PCC 15-ft joint spacing 1.5-inch dowel diameter		
	High volume	Layer 1: 13.0-inch PCC 15-ft joint spacing 1.5-inch dowel diameter		

Schematic diagrams that present the information on rigid pavement structural sections in **Table 7** are included in **Appendix J** of the report.

2.2 Summary and Conclusions

The structural profiles for each of the sample pavement sections for each geographic location were selected by considering the range of asphalt and concrete pavement thicknesses presented by roadway functional classification (for both the Interstate and Other NHS categories) in the FHWA HPMS database. The average base case truck traffic levels for each geographic location and pavement section were generated for three different highway conditions (high-volume Interstate, medium-volume Interstate, and low-volume Other NHS) that are representative of the National Network.

In conclusion, the outlined approach is intended to provide for a reasonable structural analysis of the most common types of existing pavements using appropriate representative baseline truck traffic volumes.

2.3 References

American Association of State and Highway Transportation Officials, *Mechanistic-Empirical Pavement Design Guide, Interim Edition: A Manual of Practice,* Washington, D.C., July 2008.

Applied Research Associates, *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, Final Report - Part 2 Design Inputs, Chapter 2 Materials Characterization, March 2004, available online: http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf (accessed November 6, 2013).

CHAPTER 3 – RESULTS OF PAVEMENT ANALYSIS TO PREDICT IMPACT OF SCENARIO TRAFFIC

3.1 Impacts of Scenarios on Pavement Performance

The impacts of the six Study scenarios on the performance of the sample pavement sections were assessed by comparison to the base case traffic. **Table 8** shows the vehicles that would be allowed under each scenario as well as the current vehicle configuration from which most traffic would likely shift (the control vehicle).

Analysis of Base Case

The pavement analysis averaged the base case (2011) detailed VMT estimates for the States in the vicinity of each geographic location—including estimates of travel by operating gross weight for each vehicle configuration—using the master VMT arrays developed for the study and described the *Task III: Data Acquisition and Technical Analysis* report. As described in greater detail in that report, separate axle weight and type distributions were developed for each geographic location and for each detailed vehicle class and operating weight group based on data made available by FHWA. Note that a large quantity of WIM data was available, but the available vehicle classification data had significantly less coverage than in previous studies. As a result, estimates of axle weights and types have a higher level of reliability than estimates of VMT by truck configuration, especially for non-Interstate highways.

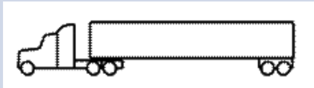
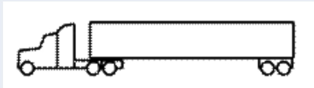
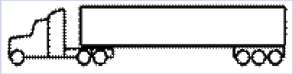
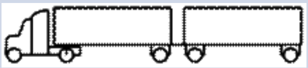
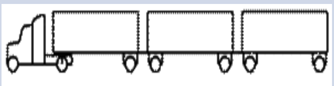
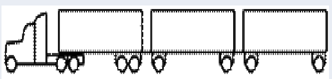
The USDOT study team compiled and condensed the VMT, operating weight, and axle weight distribution data into the formats needed for input into *Pavement ME Design®* software (i.e., percentage of truck travel by each summary configuration, number of axles of each type for each summary configuration, and percentage of axles in each of 40 axle weight categories for each type of axle for each summary configuration). Note that the travel and axle distributions vary by geographic location and type of highway (Interstate and other arterial), but not by traffic volume range (HV and MV). **Table 3** above depicts trucks per day for each of the volume ranges and highway types in each geographic location.

The default *Pavement ME Design®* software lane distributions, axle spacings characteristic of each axle type, tire pressures, and truck wheel bases were used. The *Pavement ME Design®* software was run for each base case traffic level to assess the performance of the pavement sections before any changes in truck size and weight regulations. The results from the base case analyses were found to be reasonable for all of the sample pavement sections. The detailed results for the base case analyses can be found in **Appendix K**.

Comparison of Base Case and Study Scenario Impacts on Pavement Performance

Each scenario shifted traffic from a base case set of operating weight groups of a small number of vehicle classes to a set of higher operating weight groups. Sometimes the higher operating weight groups were in the same vehicle class and sometimes they were in another vehicle class. Full descriptions of the procedures used in estimating these shifts and the resulting shifts in freight are contained in the *Volume II: Modal Shift Comparative Analysis* report.

Table 8: Truck Configuration and Weight Scenarios Analyzed in the 2014 CTSW Study

Scenario	Configuration	Depiction of Vehicle	# Trailers or Semi-trailers	# Axles	Gross Vehicle Weight (pounds)	Roadway Networks
Control Single	5-axle vehicle tractor, 53 foot semitrailer (3-S2)		1	5	80,000	STAA[1] vehicle; has broad mobility rights on entire Interstate System and National Network including a significant portion of the NHS
1	5-axle vehicle tractor, 53 foot semitrailer (3-S2)		1	5	88,000	Same as Above
2	6-axle vehicle tractor, 53 foot semitrailer (3-S3)		1	6	91,000	Same as Above
3	6-axle vehicle tractor, 53 foot semitrailer (3-S3)		1	6	97,000	Same as Above
Control Double	Tractor plus two 28 or 28 ½ foot trailers (2-S1-2)		2	5	80,000 maximum allowable weight 71,700 actual weight used for analysis[2]	Same as Above
4	Tractor plus twin 33 foot trailers (2-S1-2)		2	5	80,000	Same as Above
5	Tractor plus three 28 or 28 ½ foot trailers (2-S1-2-2)		3	7	105,500	74,500 mile roadway system made up of the Interstate System, approved routes in 17 western states allowing triples under ISTEA Freeze and certain four-lane PAS roads on east coast[3]
6	Tractor plus three 28 or 28 ½ foot trailers (3-S2-2-2)		3	9	129,000	Same as Scenario 5[3]

[1] The STAA network is the National Network (NN) for the 3S-2 semitrailer (53') with an 80,000-lb. maximum GVW and the 2-S1-2 semitrailer/trailer (28.5') also with an 80,000 lbs. maximum GVW vehicles. The alternative truck configurations have the same access off the network as its control vehicle.

[2] The 80,000 pound weight reflects the applicable Federal gross vehicle weight limit; a 71,700 gross vehicle weight was used in the study based on empirical findings generated through an inspection of the weigh-in-motion data used in the study.

[3] The triple network is 74,454 miles, which includes the Interstate System, current Western States' triple network, and some four-lane highways (non-Interstate System) in the East. This network starts with the 2000 CTSW Study Triple Network and overlays the 2004 Western Uniformity Scenario Analysis, Triple Network in the Western States. There had been substantial stakeholder input on networks used in these previous USDOT studies and use of those provides a degree of consistency with the earlier studies. The triple configurations would have very limited access off this 74,454 mile network to reach terminals that are immediately adjacent to the triple network. It is assumed that the triple configurations would be used in LTL line-haul operations (terminal to terminal). The triple configurations would not have the same off network access as its control vehicle–2S-1-2, semitrailer/trailer (28.5'), 80,000 lbs. GVW. The 74,454 mile triple network includes: 23,993 mile network in the Western States (per the 2004 Western Uniformity Scenario Analysis, Triple Network), 50,461 miles in the Eastern States, and mileage in Western States that was not on the 2004 Western Uniformity Scenario Analysis, Triple Network but was in the 2000 CTSW Study, Triple Network (per the 2000 CTSW Study, Triple Network).

The characteristic axle weight and type distributions for each vehicle class and operating weight group were applied to the new distribution of vehicle weights and types to derive the traffic input changes needed to apply *Pavement ME Design*® software to each scenario. Daily traffic volumes were decreased to reflect the smaller number of trucks needed to carry the same amount of freight and adjusted upward to account for the freight diverted from other modes of transportation.

Scenario 1 allows five-axle semitrailer (3-S2) trucks to increase from 80,000 lb. to 88,000 lb. and allows tandem axle weights to increase from 34,000 lb. to 38,000 lb. Since most of these vehicles operate below the current legal maximum weights, the modal shift estimates for this scenario show fairly small upward shifts in operating weights. They also do not show any shifts from other vehicle types to five-axle single-semitrailer combination vehicles. Overall, there is a small upward shift in the distribution of tandem axle weights, both for five-axle conventional semitrailer combinations (3-S2) and for all trucks (see **Figure 2**, below). Overall the scenario shows a net decrease of 0.6 percent in tandem axles on Interstate highways and a decrease of 0.5 percent on Other NHS highways, (0.2 percent to 0.4 percent) in the number of axle loads.

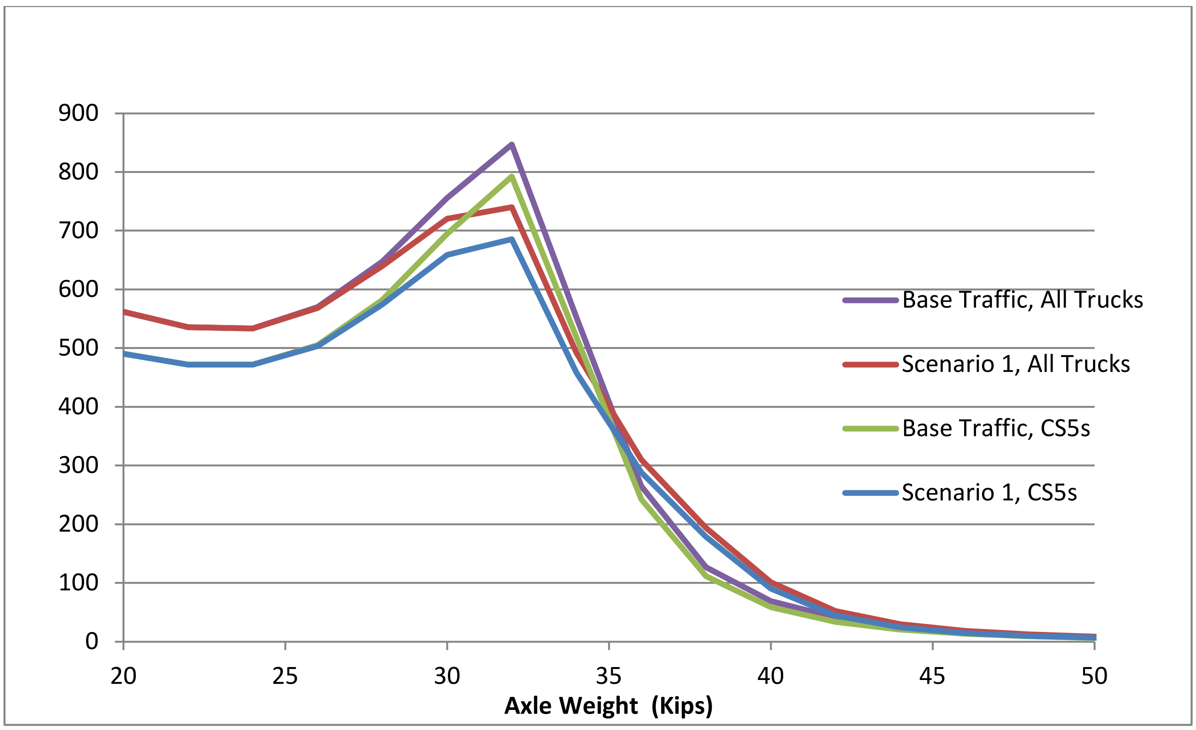


Figure 2: Scenario 1 Changes in Interstate Tandem Axle Loads

Scenario 2 allows six-axle semitrailer (3-S3) trucks with a tandem drive axle and tridem trailer axle to operate at weights up to 91,000 lb. The scenario also allows tridem axle weights up to 45,000 lb. The modal shift estimates resulted from shifting a portion of five-axle single-semitrailer traffic (3-S2) for operating weights between 76,000 lb. and 90,000 lb. to six-axle vehicles (3-S3) weighing between 80,000 and 91,000 lb., resulting in shifts away from some of the heaviest tandem axle sets to tridem axle sets, as shown in **Figure 3** and **Figure 4**, respectively. Overall, this scenario resulted in reductions in the average number of tandem axles of 4.9 percent on Interstate highways and 4.1 percent on other NHS highways. Single axles decreased by 1.7 percent on Interstate highways and by 1.0 percent on other NHS highways.

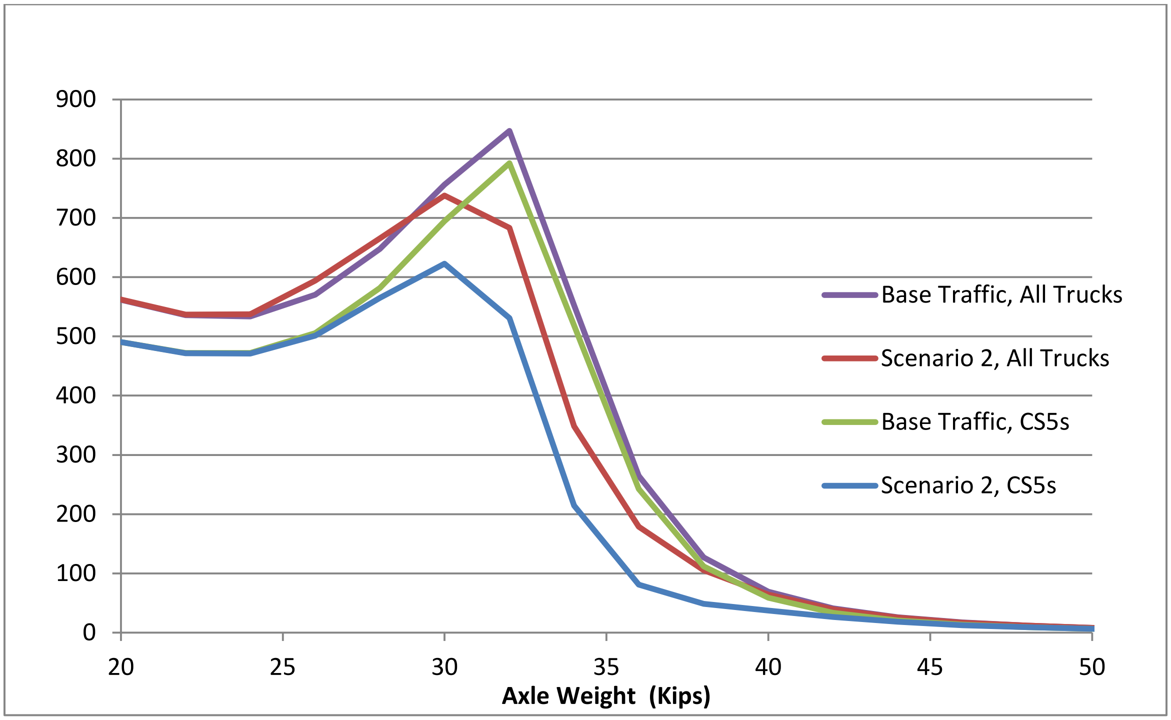


Figure 3: Scenario 2 Changes in Interstate Tandem Axle Loads

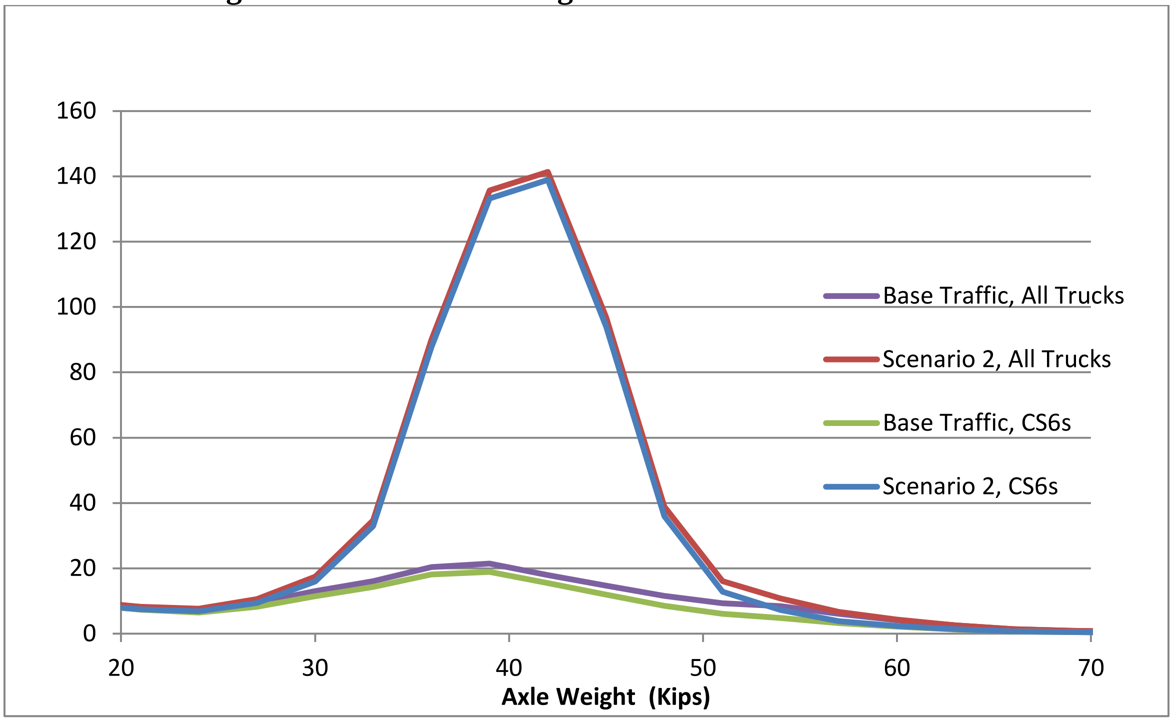


Figure 4: Scenario 2 Changes in Interstate Tridem Axle Loads

Scenario 3 allows six-axle semitrailer (3-S3) combination trucks with a tandem drive axle and tridem trailer axle to operate at weights up to 97,000 lb. It also allows tridem axle weights up to 51,000 lb. As with Scenario 2, the modal shift estimates considered shifts only from five-axle semitrailer to six-axle semitrailer; in this case, however, there is shifting from five -axle semitrailer traffic (3-S2) for operating weights between 76,000 lb. and 96,000 lb. to six-axle vehicles (3-S3) weighing between 80,000 and 97,000 lb., resulting in even more pronounced shifts away from some of the heaviest tandem axle sets to tridem axle sets, as shown in **Figure 5** and **Figure 6**, respectively. Overall, this scenario resulted in reductions in the average number of tandem axles of 6.7 percent on Interstate highways and 5.6 percent on other NHS highways. Single axles decreased by 2.6 percent on Interstate highways and by 1.5 percent on other NHS highways.

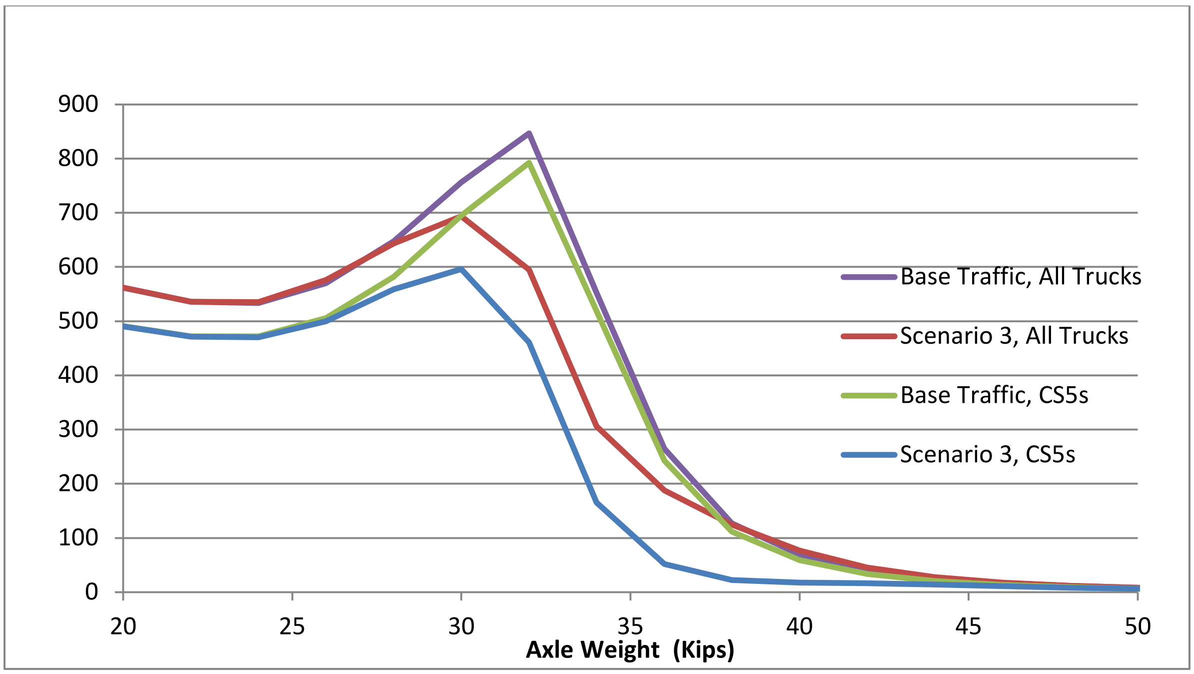


Figure 5: Scenario 3 Changes in Interstate Tandem Axle Loads

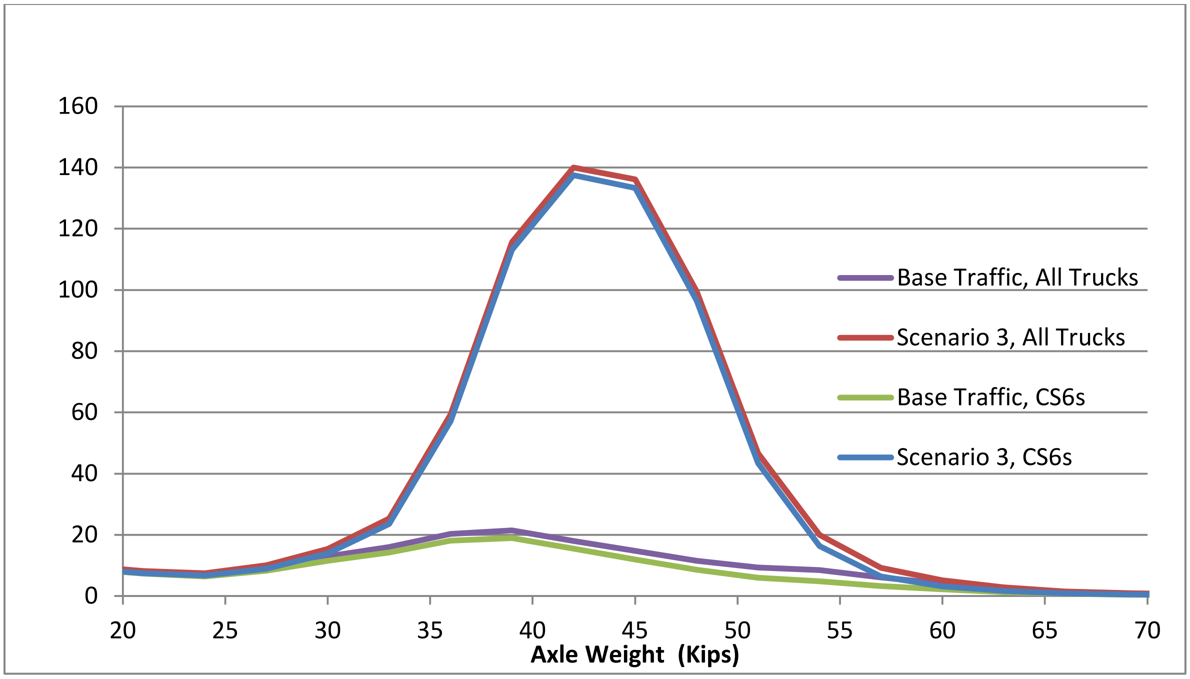


Figure 6: Scenario 3 Changes in Interstate Tridem Axle Loads

Scenario 4 allows trailer lengths on five-axle twin-trailer (2-S1-2) trucks to increase from 28 feet to 33 feet, resulting in significantly higher available cubic capacity. Unlike with Scenarios 1 to 3, which allowed higher operating weights but no increase in trailer volumes, Scenario 4 caters to the large proportion of truck traffic whose capacity is limited by size rather than by weight. The modal shift analysis shows significant shifts from configurations with five-axle single-semitrailer trucks with weights between 40,000 lb. and 70,000 lb. and shorter five-axle twins weighing between 42,000 and 79,000 lb. to configurations featuring longer five-axle twin-trailers weighing between 44,000 and 80,000 lb.

Overall, 10 percent of five-axle single-semitrailer trucks and 72 percent of shorter five-axle twins are projected to shift to heavier and longer five-axle twin-trailers under this scenario, resulting in shifts away from light- and moderate-weight tandem axle sets to higher-range single axles, as shown in **Figure 7** and **Figure 8**, respectively. Overall, this scenario resulted in reductions in the average number of tandem axles of 8.7 percent on Interstate highways and 7.5 percent on other NHS highways. Single axle vehicles increased by 10.2 percent on Interstate highways and by 6.0 percent on other NHS highways.

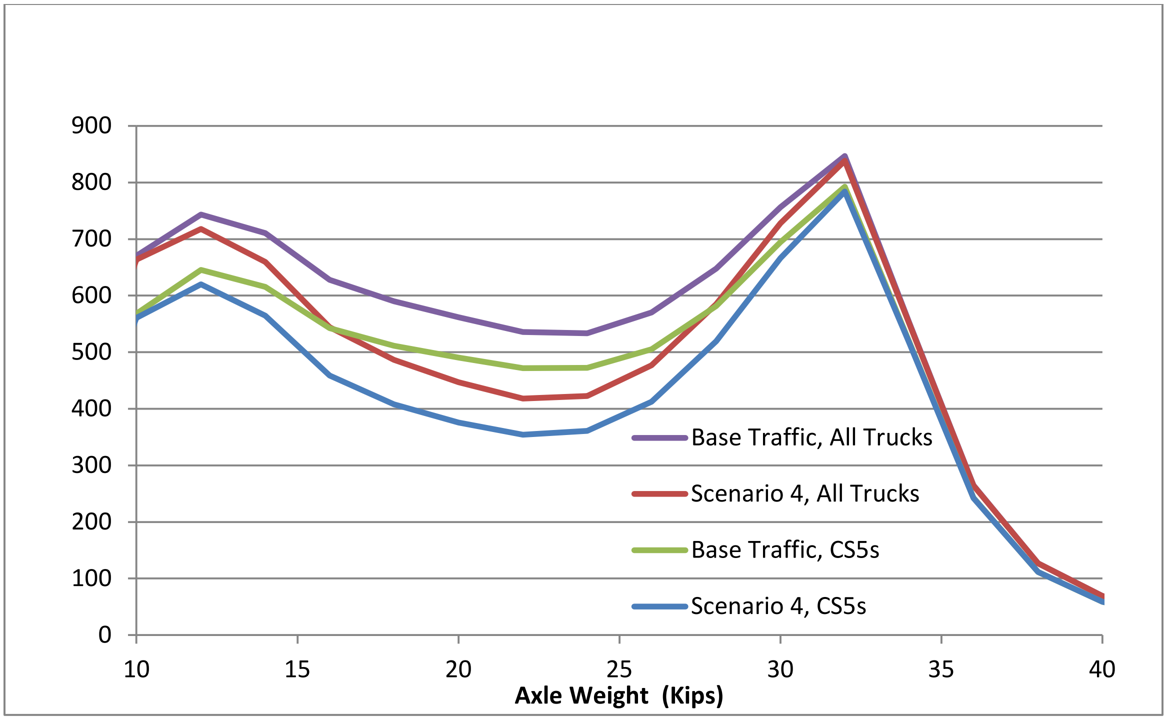


Figure 7: Scenario 4 Changes in Interstate Tandem Axle Loads

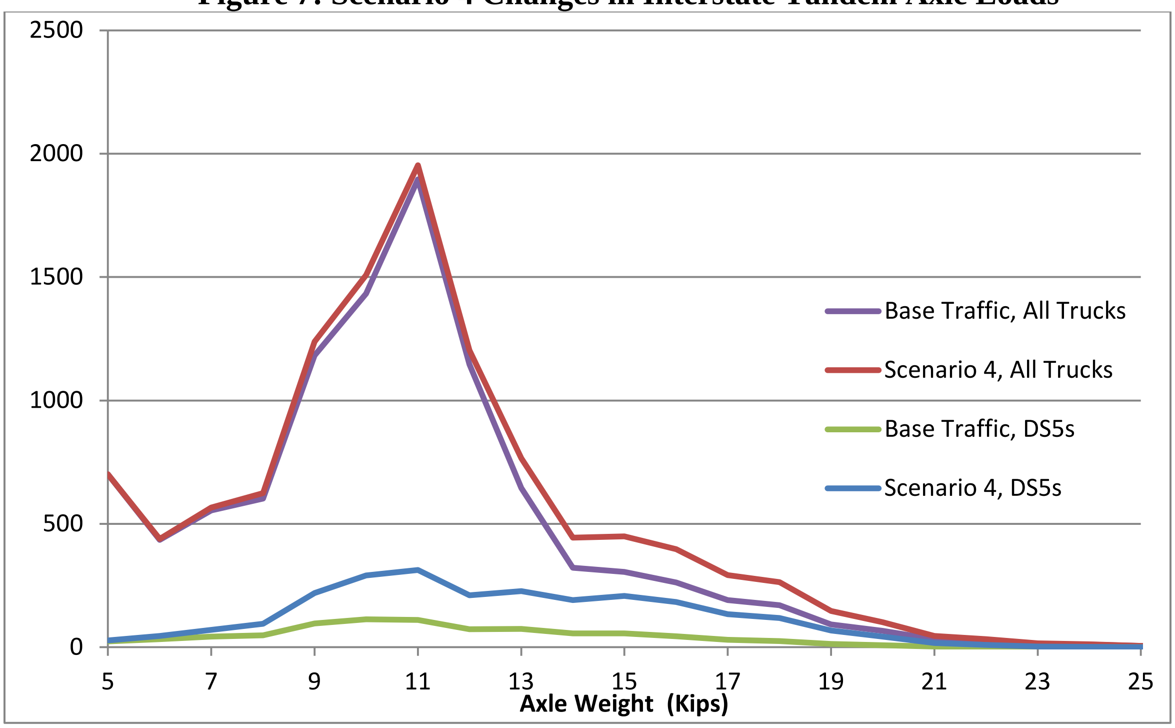


Figure 8: Scenario 4 Changes in Interstate Single Axle Loads

Scenario 5 allows seven-axle triple trailers (2-S1-2-2) that weigh up to 105,000 lb. on a designated highway network that consists of the Interstate system and selected other principal arterials. Since a number of freight destinations are not on the designated network, the modal shift analysis assumed that some portion of most trips would require conversion of two triple-trailer combinations into three twin-trailer combinations when the triple-trailer combination left the designated network. The modal shift analysis shows shifts from five-axle semitrailer trucks with weights between 38,000 lb. and 80,000 lb., and from five-axle twins weighing between 40,000 and 80,000 lb. to seven-axle triple trailers weighing between 54,000 and 105,000 lb. on the designated network and to five-axle twins weighing between 40,000 and 74,000 lb. off the designated network.

The net effect of the scenario results in fewer lighter and moderate weight tandem axle sets, as shown in **Figure 9**, with little change to the distribution of single axles on the Interstate system, as shown in **Figure 10**, but larger shifts to heavier single axles on other NHS highways, as shown in **Figure 11**. Overall, this scenario resulted in reductions in the average number of tandem axles of 3.4 percent on Interstate highways and 5.8 percent on other NHS highways. Single axles decreased by 2.4 percent on Interstate highways and increased by 2.7 percent on other NHS highways.

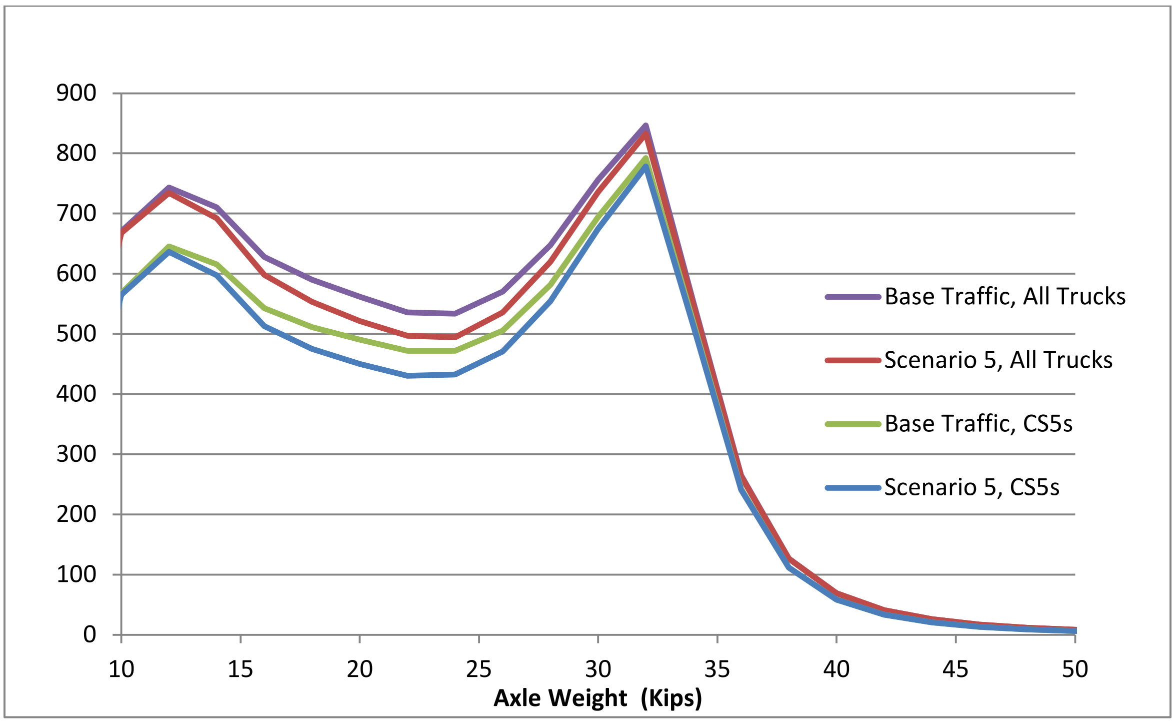


Figure 9: Scenario 5 Changes in Interstate Tandem Axle Loads

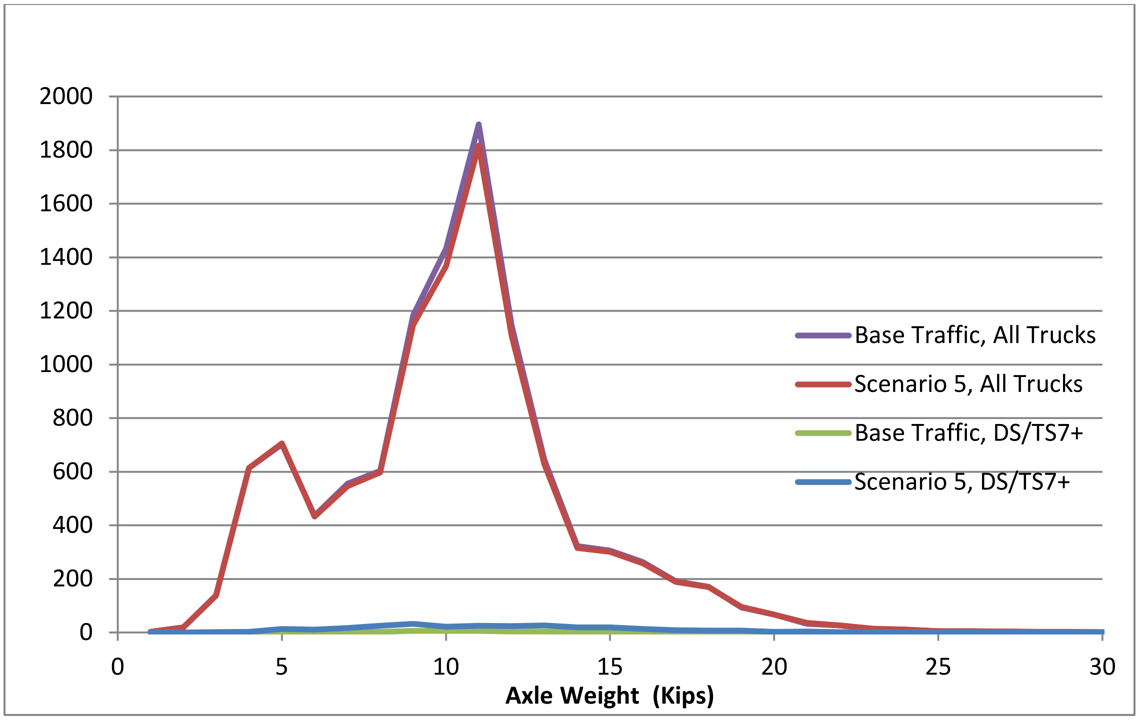


Figure 10: Scenario 5 Changes in Interstate Single Axle Loads

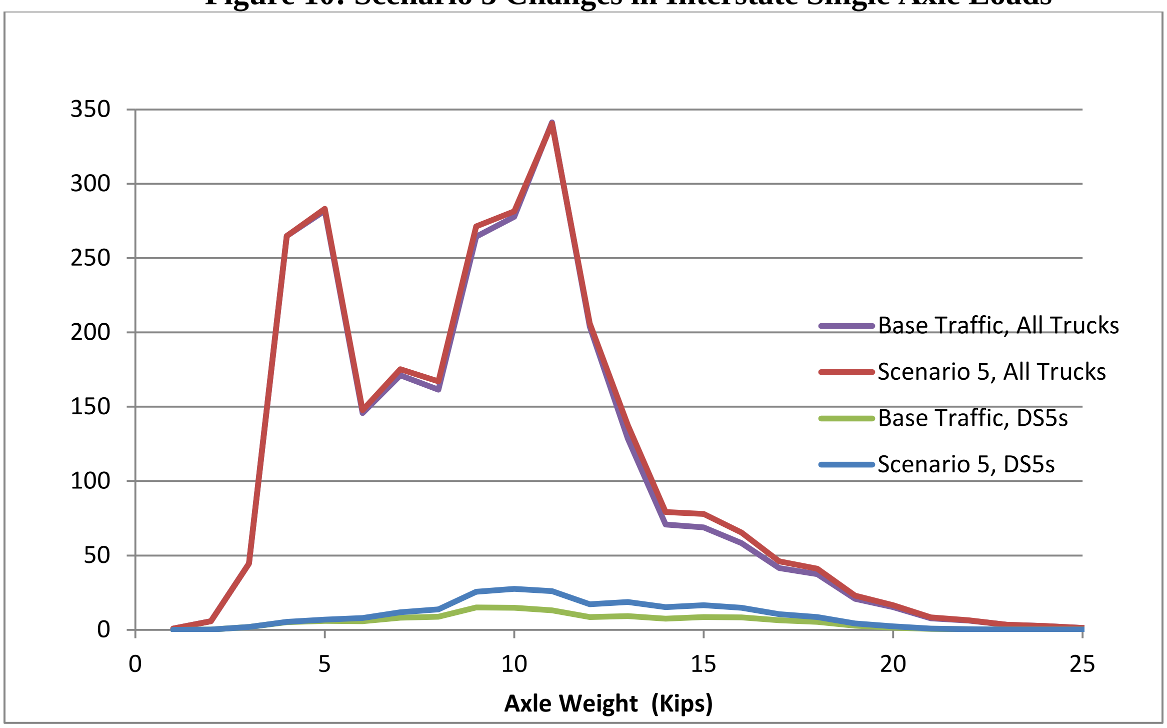


Figure 11: Scenario 5 Changes in Other NHS Single Axle Loads

Scenario 6 is very similar to Scenario 5 except that the Scenario 6 triple trailer combination has nine axles (3-S2-2-2) opposed to seven axles for Scenario 5 (2-S1-2-2) and can weigh up to 129,000 lb. as opposed to Scenario 5's 105,500 gross vehicle weight. Each of these configurations was limited in their travel to a designated network consisting of the Interstate system and access controlled principal arterials. The modal shift analysis assumed that some portion of most trips would require conversion of two triple-trailer combinations into three twin-trailer combinations when the triple-trailer combination left the designated network. The modal shift analysis shows modest shifts from five-axle semitrailer trucks with weights between 38,000 lb. and 80,000 lb. and from five-axle twins weighing between 40,000 and 80,000 lb. to nine-axle triple trailers weighing between 56,000 and 129,000 lb. on the designated network. The shift can also occur to five-axle twins weighing between 40,000 and 86,000 lb. off of the designated the network.

As with Scenario 5, the net effect of the scenario results in fewer light- and moderate- weight tandem axle sets, as shown in **Figure 12**, with little change to the distribution of single axles on the Interstate system, as shown in **Figure 13**, but larger shifts to heavier single axles on other NHS highways, as shown in **Figure 14**. Overall, this scenario resulted in reductions in the average number of tandem axles of 0.4 percent on Interstate highways and 3.7 percent on other NHS highways. Single axles increased by 2.1 percent on Interstate highways and by 4.6 percent on other NHS highways.

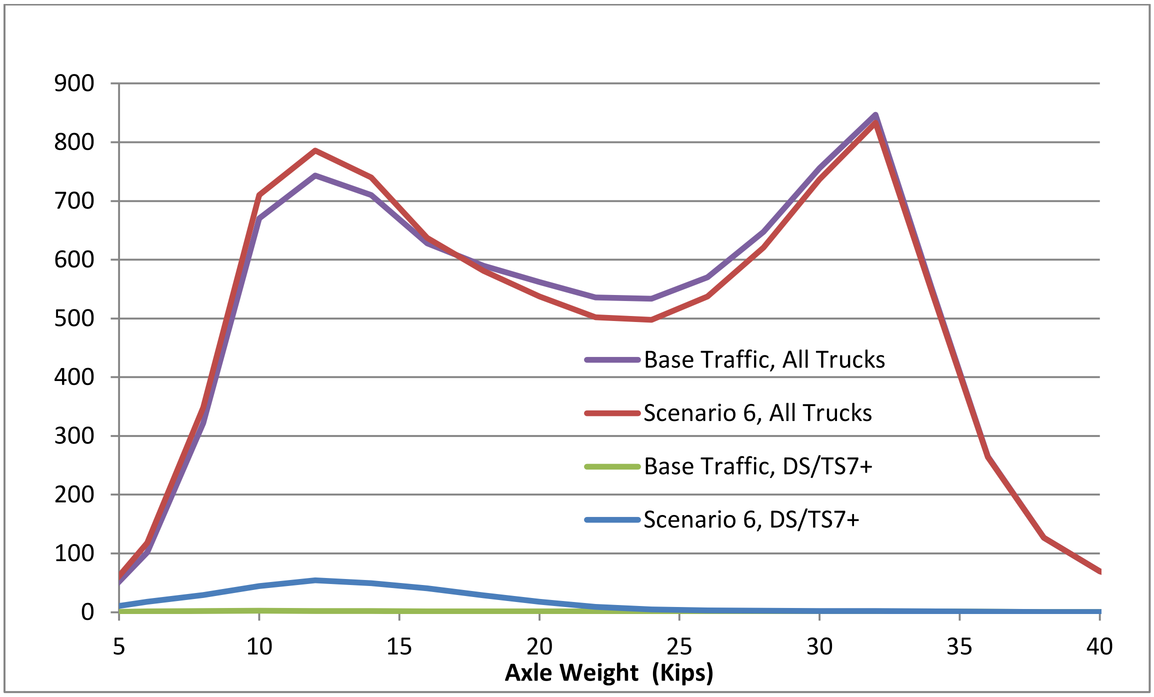


Figure 12: Scenario 6 Interstate Tandem Axle Weight Loads

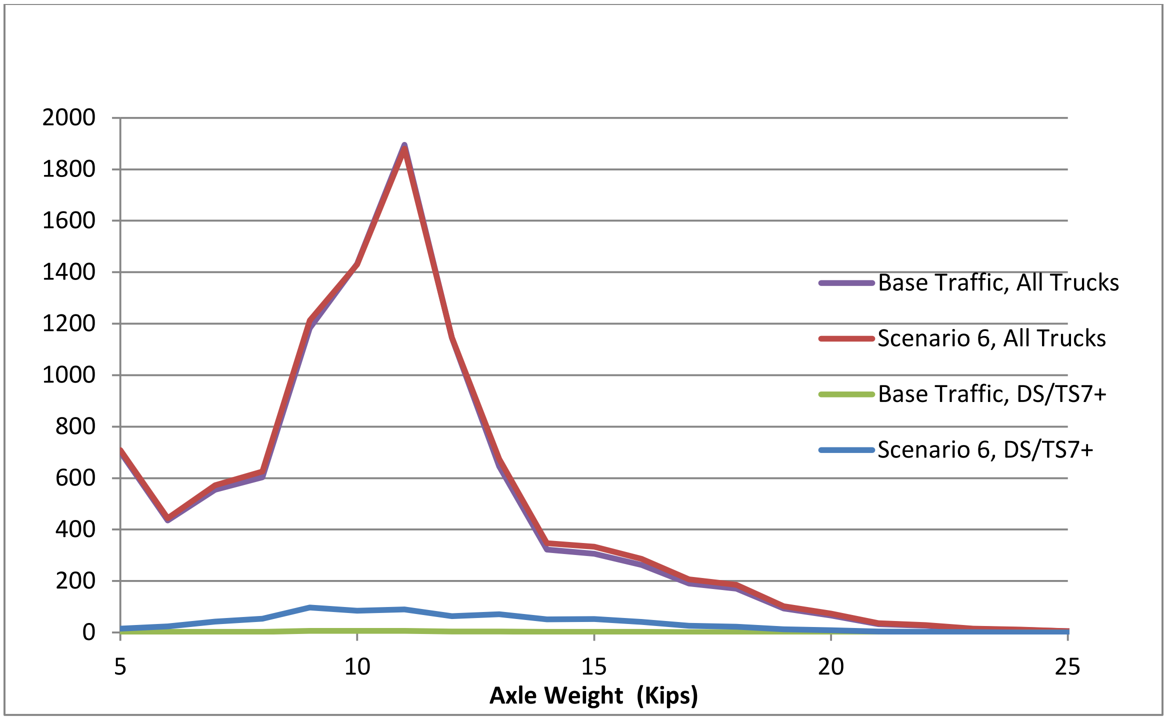


Figure 13: Scenario 6 Interstate Single Axle Weight Loads

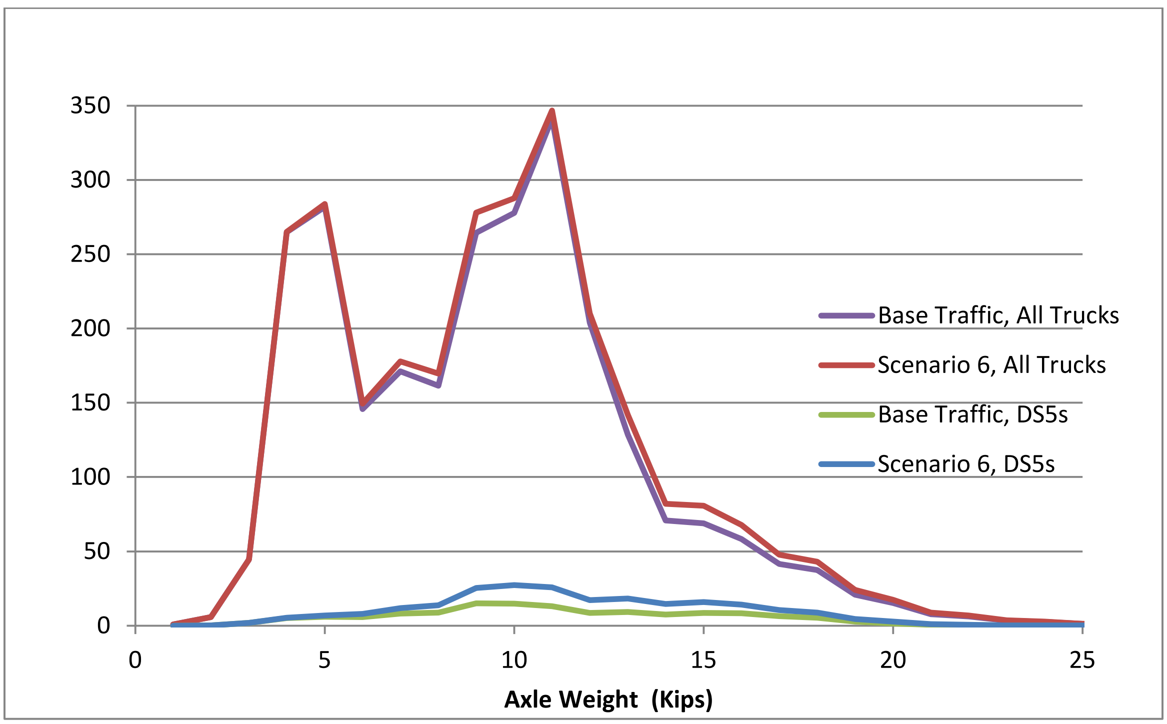


Figure 14: Scenario 6 Changes in Other NHS Single Axle Loads

The impacts of the six scenarios on the structural performance of the sample pavement sections were assessed by comparing the changes in predicted initial service intervals at the specified reliability level. The approach used the *Pavement ME Design*® software to provide performance predictions for each scenario as compared with the base case predicted performance. The initial service interval for each case was identified based on the distress thresholds presented in **Table 4**. The detailed result tables are presented in **Appendix L** for all of the sample pavement sections. A predicted pavement life factor, defined as the time to first rehabilitation for the scenario divided by the time to first rehabilitation for the base case, was computed for each pavement type in each geographic location to facilitate assessing the impacts of each scenario on pavement costs. Since the base case was considered the baseline for each pavement type in each geographic location, it was assigned a predicted pavement life factor of 1.0. The performance associated with the six scenarios was then compared to the performance of the baseline to compute predicted life factors for each.

<u>Rehabilitation Treatment Type and Frequency of Application</u>

The most common type of rehabilitation treatment for cracking in both flexible and rigid pavements is the placement of an asphalt overlay. The asphalt overlay thicknesses used for application to existing flexible and rigid pavement types for the pavement life cycle cost analysis are presented in **Table 9**.

Table 9: Asphalt Overlay Thickness to be Applied for Existing Pavement Repairs

Pavement Type and Facility		Asphalt Overlay Thickness
Flexible Pavement	Interstate (HV,MV)	3 inches (If fails from fatigue cracking, then place 4.5 inches of structural overlay)
	Other NHS (LV Arterial)	3 inches
Rigid Pavement	Interstate (HV,MV)	4.5 inches
	Other NHS (LV Arterial)	2 inches

The rehabilitation treatment types selected for the next step in the analysis were based on the failure mechanisms predicted in the pavement performance analysis. The failures predicted for rigid pavements were either by transversely cracked slabs, faulting, or roughness (IRI), while the failures predicted for flexible pavements were by rutting, IRI, or fatigue cracking. The issues with faulting or IRI in concrete pavements can be addressed by diamond grinding the surface for a number of years, but ultimately may require a structural overlay.

The existing pavement rehabilitation treatment survival analyses was used as a basis to determine the appropriate treatment frequency of diamond grinding and asphalt overlay placement for this study. The mean life for rigid pavement diamond grinding was reported to range from 14.0 years (Darter and Hall, 1990) to 16.8 years (California Department of Transportation, 2007). A survival analysis study on both flexible and rigid pavements in Utah reported that the mean life for asphalt overlay treatments was 12 years (Hoerner et al., 1999). Therefore, the treatment type and frequency of application over a 50-year performance period were selected and are presented in **Table 10** for both flexible and rigid pavements. Note that the thickness of the asphalt overlay varies depending on the facility type (Interstate versus other NHS, but the timing of application was consistent).

Within the frame of a fifty year life cycle, the algorithm applied for flexible pavement rehabilitation was to first mill and apply a structural AC overlay assumed to last 12 years. After 12 years, another mill and slightly thinner AC overlay would be placed and again assumed to last for 12 years. The process would be repeated a third time, and then, after the third 12 year overlay life span ended, the road would be fully reconstructed.

Again, within the frame of a fifty year life cycle, the algorithm applied for rigid pavement rehabilitation depended on the primary distress type. If the pavement failed in slab cracking, a structural AC overlay was placed over the existing concrete and assumed to last 12 years. After 12 years, a thinner AC overlay would be placed and again assumed to last for 12 years. The process would be repeated a third time, but after the third overlay's 12 year life span, a full reconstruction of the entire overlaid concrete pavement would be implemented.

If the rigid pavement failed in faulting or IRI, the pavement surface was diamond ground and the resulting ride quality was assumed to last for 15 years. The same process and timing would be repeated three times, followed by a structural AC overlay.

Table 10: Rehabilitation Treatment Type and Frequency over 50-Year Pavement Performance Period

<table>
<tr><th rowspan="2">Pavement Type</th><th colspan="3">Failure Condition</th></tr>
<tr><th>Fatigue (alligator) cracking</th><th>Total Rutting</th><th>IRI</th></tr>
<tr><td>Flexible – Interstate (HV, MV)</td><td>4.5-inch structural AC mill/overlay that lasts 12 years. Another 3-inch AC mill/overlay at next 12 years. Repeat every 12 years.</td><td>3-inch structural AC mill/overlay that lasts 12 years. Another 3-inch AC mill/overlay at next 12 years. Repeat every 12 years.</td><td>3-inch structural AC mill/overlay that lasts 12 years. Another 3-inch AC mill/overlay at next 12 years. Repeat every 12 years.</td></tr>
<tr><td>Flexible – Other NHS arterial (LV)</td><td>3-inch structural AC mill/overlay that lasts 12 years. Another 2-inch AC mill/overlay at next 12 years. Repeat every 12 years.</td><td>2-inch structural AC mill/overlay that lasts 12 years. Another 2-inch AC mill/overlay at next 12 years. Repeat every 12 years.</td><td>2-inch structural AC mill/overlay that lasts 12 years. Another 2-inch AC mill/overlay at next 12 years. Repeat every 12 years.</td></tr>
<tr><th rowspan="2">Pavement Type</th><th colspan="3">Failure Condition</th></tr>
<tr><th>Slabs transverse cracked</th><th>Faulting</th><th>IRI</th></tr>
<tr><td>Rigid – Interstate (HV, MV)</td><td>4.5-inch structural AC overlay that lasts 12 years. Another 3-inch AC mill/overlay at next 12 years. Repeat every 12 years.</td><td rowspan="2">Diamond grind joints. Repeat grinding every 15 years.</td><td rowspan="2">Diamond grind pavement. Grind surface another two times (at 15 year intervals), then place structural AC overlay.</td></tr>
<tr><td>Rigid – Other NHS arterial (LV)</td><td>2-inch structural AC overlay that lasts 12 years. Another 2-inch AC mill/overlay at next 12 years. Repeat every 12 years.</td></tr>
</table>

The treatment strategies presented in **Table 10** were carried forward to assess the impacts of the scenarios on pavement costs.

3.2 References

California Department of Transportation, “Chapter 5 Diamond Grinding and Grooving,” MTAG Volume II - Rigid Pavement Preservation 2nd Edition, California Division of Maintenance, 2007, http://www.dot.ca.gov/hq/maint/RPMTAGChapter5-DiamondGrindingandGrooving.pdf.

Darter, M., and K. Hall, “Performance of Diamond Grinding,” Transportation Research Record No. 1268, Transportation Research Board, National Research Council, Washington, D.C., 1990.

Hoerner, T., Darter, M., Gharaibeh, N., and T. Crow, “Comparative Performance and Costs of In-service Highway Pavements, I-15 Utah”, Technical Report, ERES Consultants, Inc., 1999.

CHAPTER 4 – SCENARIO IMPACTS

4.1 Impacts of Scenarios on Pavement Costs

The next step in the study was to explore the impacts of the six scenarios for the pavement life cycle costs. Life cycle costs considered include only those related to highway agency costs for rehabilitation following the rehabilitation strategy and schedule presented in **Table 10**. The five-year nominal rate to be used for discounting the nominal flows that are often encountered in lease-purchase analysis is 1.9 percent. This figure is based on the Office of Management and Budget 2015 budget figures (OMB, 2013). In terms of unit costs for rehabilitation treatments, the American Concrete Pavement Association was consulted leading to a recommendation of an average cost of $3.43 per SY over the last 5 years (2009 to 2013) for concrete diamond grinding. Recent asphalt overlay construction data was gathered from the National Center on Asphalt Technology and State asphalt pavement association representatives. The total cost of milling and overlay (including milling, tack, overlay placement, and traffic control) for a 1.5-inch thick overlay is approximately $8.00 per square yard and between $11 and $12 per square yard for a 2-inch or 2.5-inch thick overlay.

It should be recognized that there are many variables involved with pavement rehabilitation that will greatly affect the cost; for example, the quantity of existing asphalt that is milled, new asphalt that is overlaid, section length, pavement thickness and geotechnical and hydraulic conditions. The asphalt concrete overlay and diamond grinding estimates are based on large-scale projects; however, smaller quantity projects could significantly increase the estimated values used in this analysis.

The rehabilitation strategies described in **Table 10** were applied beginning with the time of initial failure for the base case and each scenario using unit costs of $68,400 per lane mile for a 2-inch asphalt overlay, $74,800 for a 3-inch asphalt overlay, $87,400 for a 4.5-inch asphalt overlay, $3.43 per square yard for diamond grinding, $1.65 per square yard for milling before asphalt overlays, and a discount rate of 1.9 percent.

An example of the calculations applied for the base case and two scenarios for the high-volume flexible pavement section in the Geographic Location #1 is presented in **Table 11** to illustrate the cost comparison approach.

Table 11: Pavement Cost Comparison of Base and Scenarios 3 and 4 for High Volume Flexible Pavement in Geographic Location #1

	Base Case	Scenario 3	Scenario 4
Failure Mode	**Rutting**	**Rutting**	**Rutting**
Timing of Rehab Activity 1 (years)	31.98	32.07	31.22
Cost 1	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 46,781	$ 46,698	$ 47,467
Timing of Rehab Activity 2 (years)	43.98	44.07	43.22
Cost 2	$ 43,324	$ 42,662	$ 42,662
Net Present Cost 2	$ 18,634	$ 18,317	$ 18,317
Total Net Present Costs	***$ 65,415***	***$ 65,015***	***$ 68,462***

The example in **Table 11** shows that the initial flexible pavement rehabilitation (triggered by rutting) occurred at 32.0 years for the base case, at 32.1 years for Scenario 3, and at 31.2 years for Scenario 4. The pavements in each scenario were given a 3-inch asphalt overlay (Rehab Activity 1) at each of those times at a total cost of $86,400 per lane mile (including milling), with present value costs as shown on the third line. Additional 3-inch asphalt overlays were placed after a 12-year interval, and the present value costs of each were accumulated. Since the second overlay was near the end of the 50-year life of each scenario, the cost was prorated based on its 12-year anticipated life. The base case pavement, for example, was charged for only 6.0 years' worth of the overlay.

It should be recognized that in practice, the rehabilitation strategies may not be applied at the exact time that a failure is predicted for reasons such as funding availability, rehabilitation programming methods, weather conditions conducive to proper construction, and other priorities. For a fair, direct comparison between the different traffic scenarios, however, the service interval predictions presented in this Study represent the best estimate based on model prediction (for initial pavement service interval) and survival analysis of when the subsequent rehabilitation timing will occur.

Note that each of the scenarios included in the example above showed initial pavement service intervals at ages between 32 and 33 years, and a total of two rehabilitation activities were required during the 50-year life cycle. **Appendix L** compiles the changes in service intervals for each pavement section and **Appendix N** contains the tabulations and results of the life-cycle cost analysis on each pavement section. The shoulder pavement costs and all other costs were not included in the cost analysis.

Table 12 shows the weighted average percentage changes in initial service intervals and life cycle costs for each Scenario. Life cycle costs are shown as ranges as a result of using two discount rates applied to modeled treatment costs over the fifty year life of the pavement sections. To derive the weighted averages, **s**ample pavement sections were weighted based on the number of lane-miles of pavement of each type, thickness range, and highway type. Scenario 4 increased LCC the most, by approximately 1.8 percent more than the base case. Scenario 3 decreased LCC the most by 2.6 percent below the base case.

Table 12: Service Interval and Life Cycle Cost Percent Changes by Scenario

Scenario	1	2	3	4	5	6
% Change in Service Interval	-0.3	+2.7	+2.7	-1.6	-0.0	-0.1
% Change in LCC	+0.4 to +0.7%	-2.4 to -4.2%	-2.6 to -4.1%	+1.8 to +2.7%	+0.1 to +0.2%	+0.1 to +0.2%

Conclusions on Percent Life Cycle Cost Changes for Scenario 1

Scenario 1 (allows 3-S2 trucks to increase from an 80,000 lb. to an 88,000 lb. GVW and allows tandem axle weights to increase from 34,000 lb. to 38,000 lb.) was found to shift tandem axle weights modestly upwards but slightly decrease overall truck travel. (See the *Volume II: Modal Shift Comparative Analysis* for an in-depth discussion of the anticipated shifts between truck and rail modes and between vehicles and operating weights within the truck mode as a result of each Scenario.) Cumulative life cycle cost estimates were found to vary somewhat among the sample pavement sections for Scenario 1, with a few sections showing slight decreases and most showing slight increases in expected life cycle pavement costs.

Overall, Scenario 1 showed a small overall increase in LCC over the base traffic scenario.

Conclusions on Percent Life Cycle Cost Changes for Scenario 2

Scenario 2 (allows 3-S3 trucks to operate at weights up to 91,000 lb. and allows tridem axle weights up to 45,000 lb.) was found to result in fairly significant reductions in average loads per axle. The long-term pavement costs tended to decrease under this scenario.

Overall, Scenario 2 showed no change or decreased long-term rehabilitation LCC over the base traffic scenario for all pavement sections, with an overall weighted average LCC reduction of 2.4 percent.

Conclusions on Percent Life Cycle Cost Changes for Scenario 3

Scenario 3 (allows 3-S3 combination trucks to operate at weights up to 97,000 lb. and allows tridem axle weights up to 51,000 lb.) was also found to result in fairly significant reductions in average loads per axle. The pavement costs were observed to decrease accordingly, but slightly less under this scenario than under Scenario 2.

Overall, Scenario 3 showed decreased LCC over the base traffic scenario for nearly all sample pavement sections, with an overall weighted average 2.6 percent reduction in LCC.

Conclusions on Percent Life Cycle Cost Changes for Scenario 4

Scenario 4 (allows trailer lengths on 2-S1-2 trucks to increase from 28 feet to 33 feet) was found to result in significantly higher average loads per axle as well as significant shifts to single axles from tandem axle groups. As a result, the estimated pavement costs increased more under this scenario than any other.

Scenario 4 showed no change or increased LCC over the base traffic scenario for all sample pavement sections, with an overall weighted average 1.8 percent increase in LCC.

Conclusions on Percent Life Cycle Cost Changes for Scenario 5

Scenario 5 (allows seven-axle triple trailers up to 105,500 lb. on a designated highway network) was found to decrease truck travel and shift weight to slightly lower average axle loads on the designated network but slightly higher average axle loads on off-network highways.

Scenario 5 showed changes in LCC that varied from positive to negative when compared with the base traffic scenario for various pavement sections, with a slight (0.1 percent) overall weighted average increase in LCC.

Conclusions on Percent Life Cycle Cost Changes for Scenario 6

Scenario 6 (similar to Scenario 5, allows nine-axle triple trailer configurations up to 129,000 lb. on a designated network) was found to move significant numbers of tandem axles to lighter tandems and slightly heavier single axles.

While all the other scenarios used actual WIM observations to develop the distribution of axle weights and types for the scenario vehicles, Scenario 6 was unique in that it had no WIM observations for the target vehicle. There were many WIM observations for seven-axle and eight-axle triple trailer configurations in the full range of operating weights up to 130,000 lb., so axle weights observed on seven- and eight-axle triple trailer configurations were used to estimate a distribution of axle weights for the nine-axle triple trailer configurations at each operating gross weight. Steering axles were assumed to keep the same weight distributions, as were tandem axles on the eight-axle triple trailer configurations. Single load axles were assumed to be equally likely to have been converted to tandem load axles across the full range of observed axle weights.

Similar patterns of life cycle cost estimates and variations were observed for Scenario 6 as were observed for Scenario 5, showing changes in LCC that varied from positive to negative when compared with the base traffic scenario for various sample pavement sections, with a slight (0.1 percent) overall increase in weighted average LCC.

4.2 Impacts on Local Roads

The study focused on Interstate and NHS highways—key corridors in which trucks are delivering the highest vehicle miles traveled (VMT). More than 80% of total annual truck miles travelled occurs on the NHS. There is a more than four million center line miles of public roadway mileage in the United States with most of those miles located off of the NHS. Except in cases of very rare exception, there is little quantitative information available regarding travel, by facility, occurring on this non-NHS roadway network and on how these pavements are designed, built, and maintained. There is minimal to no history of HPMS data from local roads, and hard data on both how many and how often trucks use these facilities is in many cases not readily available. These data limitations have made it prohibitive to perform an accurate and representative study on the impacts of loading scenarios on local roads at this time. The lack of pavement structure characteristics, pavement surface type and typical travel levels for local system roadways yields it impossible to develop sampling based approaches that would produce results supported with adequate statistical confidence.

In order to better investigate the qualitative impacts of the six loading scenarios on local road pavements, a framework would need to start by gathering information on existing pavement sections, pavement design standards used, construction specification details, maintenance frequency and application types, materials properties of the pavements and underlying soils, and the traffic amount and distribution. It would be particularly beneficial to quantify the truck traffic volumes, truck type distributions (classes), and truck traffic variations (seasonally, hourly, monthly, etc.) as a basis for comparing the effects of introducing any of the six loading scenarios in this study.

4.3 References

California Department of Transportation, “Chapter 5 Diamond Grinding and Grooving,” MTAG Volume II - Rigid Pavement Preservation 2nd Edition, California Division of Maintenance, 2007, http://www.dot.ca.gov/hq/maint/RPMTAGChapter5-DiamondGrindingandGrooving.pdf.

Darter, M., and K. Hall, “Performance of Diamond Grinding,” Transportation Research Record No. 1268, Transportation Research Board, National Research Council, Washington, D.C., 1990.

Hoerner, T., Darter, M., Gharaibeh, N., and T. Crow, “Comparative Performance and Costs of In-service Highway Pavements, I-15 Utah”, Technical Report, ERES Consultants, Inc., 1999.

Office of Management and Budget, “Discount Rates for Cost-Effectiveness, Lease Purchase, and Related Analyses,” Circular A-94 Appendix C, OMB Circular No. A-94, Washington, D.C., 2013, http://www.whitehouse.gov/omb/circulars_a094/a94_appx-c.

Oman Systems, Inc., BidTabs, 2013, http://omanco.com/index.php/product/bidtabs/

CHAPTER 5 – CURRENT IMPACTS OF OVERWEIGHT VEHICLES

5.1 Impacts of Overweight Vehicles on Pavement Costs

Vehicle-related pavement costs are incurred based on the axle loads that vehicles impose on a roadway. Current regulations allow single axle weights of 20,000 lbs. and tandem axle weights of 34,000 lbs. on the Interstate system, but many vehicles have axle weights in excess of those limits because of special permits, lack of enforcement, or other factors.

The study analyzed the effect of overweight axles in current operations, defining overweight as single axles weighing more than 20,500 lbs. and tandem axles weighing more than 35,000 lbs. to be consistent with the axle weight group boundaries used in the vehicle weight analysis and in recognition of the fact that any regulation has a small margin of enforcement tolerance associated with it.

The approach used to analyze these impacts was straight-forward. For a selected subset of pavement sections (the medium-volume traffic sections on the Interstate system), *Pavement ME Design*® software was applied twice—once with the base case of traffic, and once with all axles over the specified limit removed from the traffic mix. Comparing the results of the runs would allow estimation of how many years it would take under each traffic mix for a pavement to reach the end of its initial service interval—the time when some sort of rehabilitation action would be needed,

Initial service intervals were found to increase significantly for both flexible and rigid pavement sections (except in the case of one rigid pavement section that did not reach the end of its initial service interval during the analysis period).

Flexible pavement initial service intervals increased by between and 19 percent and 34 percent and rigid pavement initial service intervals increased by between 0 percent and 10 percent when overweight axles were removed from the traffic mix. The initial service interval changes for each of the MV Interstate pavement sections are contained in **Appendix O**. LCCs would decrease correspondingly.

APPENDIX A: REVISED PAVEMENT DESK SCAN REPORT

TABLE OF CONTENTS

CHAPTER 1 - INTRODUCTION

This report presents a revised version of the Desk Scan (Subtask V.B.2) developed to support the Pavement Comparative Analysis (Task V.B.) of the 2014 *Comprehensive Truck Size and Weight Limits Study (2014 CTSW Study)*. This revised Desk Scan addresses the recommendations made by the National Academy of Science (NAS) Peer Review Panel concerning the originally submitted version of this scan.

1.1 Purpose

The purpose of the revised Desk Scan is to:

- Reorganize and enhance the original Desk Scan; and
- Add any additional, relevant content that may have been identified since the submission of the original Desk Scan.

Specifically, the NAS Peer Review Panel recommended that the original Desk Scan be reorganized to address four issues:

- Survey of analysis methods and a synthesis of the state of the art in modeling impacts
- Identification of data needs and a critique of available data sources
- Assessment of the current state of understanding of the impacts and needs for future research, data collection and evaluation
- Synthesis of quantitative results of past studies including reasonable ranges of values for impact estimates.

The team reviewed the most relevant previous studies comparing pavement costs of vehicle use, including state, national, and international cost allocation and truck size and weight studies, as well as any other studies that include estimates of vehicle-induced pavement costs on either an absolute or relative basis. They also include pavement analysis or design studies that will help in the application of *AASHTOWare Pavement ME Design*® or in the compilation of data required for that model. The principal objective of the search was to gain a thorough understanding of the current state of research and practice concerning pavement cost analysis related to heavy vehicle use. The literature search included a variety of information sources: (1) engineering and scientific periodicals and journals; (2) conference proceedings; (3) federal, state, international, and university reports that show up in library search engines, such as Compendex, based on key words; and (4) studies identified during the May 29, 2013 public hearing for the 2014 CTSW Study or by USDOT officials.

Cost allocation studies develop detailed estimates of costs related to vehicle weights and other characteristics in somewhat more detail than a typical Truck Size and Weight (TSW) study, so this desk scan included both highway cost allocation (HCA) and TSW studies at the federal and state levels and in other countries. No studies with new methodologies were found, but several will help in the pavement cost analysis for this 2014 CTSW Study, and several more lend support or perspective to the proposed approach. Section 1.6 includes a list and brief synopsis of all publicly-available reports reviewed as part of this desk scan.

1.2 Overview of Alternative Approaches to Analyzing Pavement Costs

The pavement team's review of previous studies and techniques for analyzing pavement costs associated with changes in traffic loads reveals approaches that fit into three broad categories: (1) using traditional "equivalent single axle loads" (ESALs) derived from the half-century-old AASHO Road Test as a measure of pavement damage, and therefore pavement damage costs, (2) applying pavement deterioration models to a representative group of pavement sections with a large number of traffic loading conditions to derive a new set of load equivalence factors (LEFs) and deterioration curves that vary by distress type, or (3) directly applying current pavement design models to a small number of sample pavement sections under scenario traffic loadings to derive estimates of changes in pavement life and therefore pavement cost changes. Each of these three alternative approaches is discussed below.

1.2.1 Using ESALs as a Measure of Pavement Damage

As used in this report, the term "ESAL" refers exclusively to the AASHO-Road-Test-based factors as calculated by formulas in the in the ***1993 AASHTO Pavement Design Guide***. All other factors are referred to by the generic term "LEF". FHWA's HCA and TSW studies stopped using unmodified AASHO-Road-Test-based ESALs in 1979, after the Congressional Budget Office (CBO) strongly criticized their continued use, based on the outdated assumptions used to derive the formula for ESALs, which was based on a short term test of a small set of pavement cross sections in a single environmental zone. It should be noted that only a limited range of axle types were included in the study, and the calculation of ESALs for tridem axles is based on extrapolating a dummy variable. Most, but not all, states followed the federal lead and discontinued use of ESALs for HCA studies, but typically continued to use them when they commissioned TSW studies.

By far the largest number of truck-size-related pavement studies in the past fit into the first category: using ESALs as an assumed determinant of pavement damage and deriving cost estimates in various derivative approaches based on that initial assumption. The most prevalent approach involves calculating the number of ESALs before and after the proposed or implemented changes in vehicle traffic loads, calculating either an average or marginal cost per ESAL through either a micro or macro approach, and simply multiplying the two factors.

The 2009 *Wisconsin Truck Size and Weight Study* provides an example of a typical ESAL-based approach. As stated in the study, the analysis used a four-step approach to estimate pavement (and, in this case, bridge deck) impacts of each size-and-weight scenario:

- *Step 1 – Estimate cost to highway agencies and other road users associated with an additional ESAL mile of travel for various types of highways and highway conditions;*

- *Step 2 – Estimate ESALs as a function of operating weight for Base Case and Scenario trucks;*

- *Step 3 – Calculate the change in ESAL miles due to freight shifting from Base Case to Scenario trucks; and*

- *Step 4 – Calculate the change in pavement and bridge deck costs as the product of 1) the change in ESAL miles and 2) cost per ESAL mile.*

One could argue that ESALs provide a reasonably credible job at describing the average effects of single and tandem axle loads under typical conditions, despite their general limitations cited above, since the Road Test did use a range of weights of each of those axle types, and since its measured variable, roughness, is a function of most of the common flexible and rigid pavement distresses. Since some of the Wisconsin scenarios, however, used tridem axle trucks, using ESALs that are based upon extrapolating a dummy vehicle for those vehicles is considerably less credible.

Recent studies in Virginia (Allen *et al.*, 2010) and Kansas (Bai *et al.*, 2010) derived statewide and corridor-specific, respectively, per-ESAL-mile estimates of variable pavement maintenance costs to use as basis for estimating the pavement cost impacts of heavier truck traffic. The Virginia study is notable in that it provides an excellent example of using current expenditures and imputed foregone maintenance costs to derive a statewide estimate of damage per truck-mile of travel. Although marred by the use of ESALs as the loading metric, rather than a more up-to-date set of LEFs, the overall approach of calibrating to actual costs has merit.

Since using ESALs as a basis for differentiating among trucks for national policy considerations is neither technically defensible nor politically feasible, the second and third types of approach have more potential for use in the CTSW.

1.2.2 Deriving Pavement Damage Relationships from Pavement Performance Models

Most of the studies that did not use ESALs used another form of axle load factors (ALFs) or load equivalence factors (LEFs), that were typically derived for a single particular distress to describe the relative damage by one axle weight and type compared to the damage of a standard axle. As used in this report, the term ”LEF” refers to a factor that describes the relative damage caused by one axle to a standard reference axle. ESALs, as used in this report, are a specific type of LEF. Some of the LEFs derived in previous studies were based on mechanistic primary-models, and some of these were calibrated to a small amount of observed empirical data. Some approaches based their LEFs on the MEPDG model in its various versions. Some of the studies used reduction in time-to-failure as the variable that determined LEFs, and some used target distress levels.

The 1997 *Federal Highway Cost Allocation Study (HCAS)* Final Report and the 2000 *U.S. Department of Transportation Comprehensive Truck Size and Weight Study* both used the same version of the National Pavement Cost Model (NAPCOM) for estimating the relative shares of pavement damage caused by each vehicle of a given type and operating weight.

NAPCOM was originally developed to enhance the 1982 HCAS approach of assigning costs to vehicles based on their estimated contribution to each pavement distress weighted by the importance of each distress to the need to repair or replace a pavement. Its earliest version used newly-developed mechanistic-empirical pavement models to derive a set of pavement damage equations for six of the most important distresses observed on each type of pavement (flexible and rigid). Each distress equation used axle types and weights as primary variables with the weight exponent independent of ESALs, so each the equation for each distress produced an LEF that varied not only by weight and type of axle, but could also vary by pavement, base, and climate characteristics as well. FHWA had updated NAPCOM regularly as new and better mechanistic-empirical pavement models became available.

The NAPCOM version used in the 1997 and 2000 studies included the third set of major updates to the damage equations. Unlike in earlier versions, however, FHWA used a simplified version of estimating axle weights for each operating group and vehicle configuration, rather than using an array of all observed axle weights in an effort to speed up run times. For each vehicle class and operating gross weight (OGW) group, axle weights for each position on a vehicle were expressed as an average of all observed weights at that position on the vehicle (steering axle, drive axle, first load axle, etc.). Given the non-linear aspect of most LEFs, this produced compromised results of unknown magnitude.

FHWA's 2012 *Vermont Pilot Program Report* used a new set of LEFs that were developed in 2011 and 2012 as part of a major restructuring of NAPCOM. The LEFs had been fully incorporated into a spreadsheet version of NAPCOM designed for use at a state level. New damage models were based on running MEPDG thousands of times, systematically varying base traffic by one axle weight and type at a time to determine the relative effect that each has on pavement deterioration in a range of pavement types and climatic conditions. The LEFs were applied to the pavement sections in Vermont that were affected by the pilot project and weighted by the prevalence of each distress on these sections. The approach used all available WIM data and classification counts to determine before-and-after axle load spectra.

Oregon's 2013 *Highway Cost Allocation Study* used the same LEFs in an adapted spreadsheet version of NAPCOM to assess appropriate levels of weight-distance tax rates and other user fees. Oregon uses smaller OGW increments than have been used previously in national studies-- 2,000-pound increments rather than 5,000 pounds-- a feature that greatly improves the precision of the results. The approach uses imputed axle weights for OGW groups on each side of a WIM observation to compensate for the smaller number of observations in each smaller OGW increment, an approach that seems to have merit. As in Vermont, LEFs were weighted based on available estimates of pavement distress prevalence, with the weighted LEF-miles used to allocate load-related pavement costs. The results allow assessment of the pavement damage costs associated with any particular type of vehicle at each OGW, so could be readily applied to a truck size and weight study.

A University of Texas paper (Bannerjee *et al.*, 2013) applied a later version of MEPDG, DARWin-ME, in a similar manner to the approach used by FHWA for PaveDAT and the latest version of NAPCOM to derive what the authors termed "Equivalent Damage Factors" (EDFs). Focusing on flexible pavements and using a smaller number of distresses than the FHWA approach (combining AC rutting and other rutting and combining all cracking components), the study found that the EDFs for rutting varied with pavement thickness somewhat more than was observed in the FHWA study. A probable explanation is that the ratio of surface-to-total rutting changes as pavements get thicker; even though the AC layer and subsurface rutting LEFs stay fairly constant, the increased prevalence of AC layer rutting with thicker pavements changes the relative importance of the two LEF components.

A Michigan DOT study (Chatti, 2009) used laboratory studies and mechanistic models to determine "axle factors" (AF) for single, tandem, tridem, quadrem, and larger axle groups. AFs were defined as the ratio of damage of a full axle grouping to the average weight of each single axle in that grouping. AF values were then used to correct AASHTO-based LEF (ESAL) values for the average single-axle weight, in effect producing new LEFs that were not dependent upon the original ESAL values for multi-axle groupings. The study reinforced MEPDG-derived LEF results from other recent studies cited above: tridems and quadrems have less relative effect than tandems or singles (with the same weight per individual axle) on cracking, but a greater relative effect on rutting. The results could allow extension of mechanistic model findings to quadrems—currently outside the scope of *AASHTOWare Pavement ME Design*®.

All the previous approaches in this group of studies have relied upon damage relationships built from earlier mechanistic-empirical models or from earlier versions of *AASHTOWare Pavement ME Design*®. Although sound in concept, their use of now-superseded models may affect their credibility, which leads us to the third group of approaches that can be used for truck size and weight analysis.

1.2.3 Estimating Pavement Performance Directly from Models

Previous truck size and weight studies have not used this approach, partly because until recently there has not been a mechanistic-empirical model that has achieved such broad acceptance among pavement engineers as has *AASHTOWare Pavement ME Design*®. A 2007 FHWA-sponsored study (Timm *et al.*, 2007) used this approach by applying MEPDG to a small number of hypothetical pavement sections, all having the same 15-inch crushed stone base, the same A-6 subgrade, and using the same MEPDG default Alabama climate file. For each of four traffic levels, flexible surface layer thicknesses were selected that resulted in reaching MEPDG-predicted terminal pavement conditions in about 20 years (24 years for the lowest–traffic section). Similarly, 15-foot rigid slab thicknesses were selected that reached terminal conditions in 20 to 28 years for each traffic level. All traffic levels used the same base case truck class and axle load distributions, varying only in annual average daily truck traffic (AADTT).

Base case predicted service lives were compared to predicted lives for three different loading scenarios: (1) shifting entire weight distributions toward heavier axles, (2) adding specific heavier axles, and (3) changing the GVW from 80,000 to 97,000 lbs. while adding an axle to the rear tandem group and using an idealized weight distribution for each vehicle type. The first scenario showed very large decreases in pavement life (and increases in cost), the second showed significant cost increases when the number of added heavy axles exceeds 10% of the number of legally loaded axles, and the third showed no practical difference between the 80,000-pound 5-axle vehicle and the 97,000-pound 6-axle vehicle. Mechanistic analysis outside of MEPDG showed only slight difference in pavement response, confirming the finding of insignificant changes in pavement costs.

The authors noted that their findings represent a limited set of conditions and that results were pavement-specific-- in several of the scenarios, one or two of the pavement sections showed much higher rates of change in service life than the other sections. The authors did not report which terminal pavement condition (IRI, alligator cracking, AC rutting, or total rutting for flexible pavements; IRI, transverse cracking, or faulting for rigid) was reached first for each of the eight pavement sections included in the study. Subsequent work using the next MEPDG version to support FHWA's NAPCOM and PaveDAT models showed large variation in the relative effects of axle weights among the various pavement condition metrics, so it is perhaps not surprising that different sections could show vary different results if two different failure mechanisms were involved.

A 2010 TRB paper (Tirado *et al.*, 2010) coupled the use of a primary response model with damage predictions from mechanistic-empirical analysis to quickly estimate relative levels of distress by particular combinations of axle loads, including groups of more than three axles. Together with the Chatti 2009 paper cited earlier, this approach could perhaps extend damage analysis to axle groups with more than three axles in the group (tridems), a current limitation of the *AASHTOWare Pavement ME Design*® model. The team does not have the time or budget to apply the approach in this study, but may find it informative to tabulate the prevalence of multi-axles in our WIM analysis and apply a rudimentary approach for considering the likely effect of considering quadrem and larger axle groupings, rather than arbitrarily dividing them into tridem and tandem groupings as is current common practice.

Using models to directly estimate changes in service lives, building especially upon the lessons learned in the work by Timm *et al.* for FHWA, could potentially achieve the objectives of this study. Assuming accurate estimates of actual axle loading spectra under the base case and each scenario for this study, as well as a small number of tightly-defined scenarios, the approach would eliminate the need to estimate LEFs for the impact assessment phase of the study thereby potentially improving the accuracy of the results calculated in this study.

1.3 Data Requirements for Pavement Comparative Analysis

Each of the types of approaches outlined above requires a variety of data inputs, with some variations. The traffic data requirements are similar to the data needs of the bridge, safety, and modal shift analyses, except that the pavement and bridge phases need more detailed information on axle load distributions.

1.3.1 HPMS Section Data

Using any of three approaches to develop a valid national estimate of changes in pavement cost would rely upon detailed knowledge of the national highway system network characteristics and traffic levels. FHWA's compilation of HPMS section data provides the best available collection of traffic estimates, single-unit truck traffic estimates, combination truck traffic estimates, pavement condition, pavement design, and age data currently available, although the level of detail about many of these parameters has to be supplemented with other data sources. Average daily truck traffic, for example, does not supply nearly enough information to properly apply any of the three approaches, so must be supplemented with vehicle classification and weigh-in-motion (WIM) data. All three approaches require approximately the same level of detail regarding traffic data.

An ESAL-based approach, although not recommended for this 2014 CTSW Study, requires the least supplemental detail for most of the non-traffic factors supplied by HPMS. ESALs vary by pavement type and thickness, which are supplied quite reliably by the section data, as well as by terminal PSI value, which can be assumed to be relatively uniform for all sections on a given highway class.

Applying a derived model such as NAPCOM requires not only the pavement type and thickness information required by an ESAL-based approach, but also detailed information about pavement condition, since modern LEFs vary by distress type. If a rigid pavement section fails by faulting, for example, LEFs increase much less with axle weight than if the section fails by cracking. Unfortunately, HPMS has only recently added information about the states of pavement distress, and states have been somewhat slow in supplying the information. Also, HPMS fails to distinguish between top-down and bottom-up cracking, or between AC surface rutting and total rutting, and LEFs for each of these distresses have significantly different exponents and significantly different offsets among the axle types (single, tandem, and tridem). Thus, the HPMS data has to be supplied with ad hoc data from other sources—either special studies or information from state pavement engineers. Some of this supplemental data has been gathered from a few states, but more information is needed.

Applying *AASHTOWare Pavement ME Design*® requires much more detail about a pavement structure and its material properties than is available on HPMS, as discussed below in section 1.3.4.

1.3.2 Vehicle Classification Data

All approaches require obtaining as much vehicle classification data as possible-- whatever FHWA can provide and deems appropriate for initial estimates of truck travel for broad classes of trucks in each state on functional class. FHWA no longer publishes or compiles formerly available HPMS area wide travel counts reported by the states for the 13 HPMS vehicle classes on each highway system, but will provide VMT estimates for regional groups of states for six broad summary vehicle classes (two truck and four passenger vehicle classes).State-reported raw vehicle classification station counts are available to support further break down of FHWA's summary VMT reports, as has been done in previous cost allocation and size and weight studies.

Raw classification data contains many errors and inconsistencies, as well as a strong tendency in most states to use class 13 as a "catchall" category. The data needs to be not only reviewed and edited, but also systematically corrected using the additional information available from WIM data. Further, FHWA is able to supply slightly over 1400 stations that have a continuous year's worth of 24-hour data (a necessary criterion to avoid temporal bias) to provide a high degree of accuracy in estimating travel by detailed vehicle class. Triple this number of stations would significantly increase the accuracy of detailed truck travel estimates. The inadequate number of classification stations and the lack of adequate quality control in the reported data are perhaps the greatest data limitation in the 2014 CTSW. This data inadequacy affects all three approaches equally, since all three depend upon accurate estimates of travel by detailed vehicle class upon which to project axle weight distributions provided by WIM data.

1.3.3 Weigh-in-Motion (WIM) Data

All three approaches also require the same level of detail on operating weight and axle weight distributions, so will make use of all available WIM data compiled by FHWA for multiple purposes in this 2014 CTSW Study, as well as the most recent years of WIM data collected for LTPP. Previous compilations of national travel estimates and truck travel characteristics have frequently used the most recent consecutive 12 months of WIM data for each state in order to lessen potential seasonal bias. There may be value in using 12, 24, or 36 months of consecutive data from the WIM sites, since that many years are generally available and easily obtained.

In general, there is much more WIM data available than ever before, increasing the accuracy of estimating the distributions of operating weights and axle loads at each station. WIM data limitations stem mostly from the insufficient number of stations reported to FHWA, as well as the lane bias of the stations. Nearly all the 19 LTPP WIM sites and the 451 FHWA-compiled, state-reported WIM sites, for example, systematically erase data collected from light vehicles (which negates the opportunity to estimate the percentage of trucks in the traffic stream) and collect WIM data from only the right lane on four-or-more-lane highways (which may bias truck type and weight estimates). Further, the potentially large differences in north-south vs. east-west truck traffic cannot be accounted for in the absence of a comprehensive truck-travel network and a sufficient number of WIM stations to populate that network.

1.3.3.1 Detailed Vehicle Class Travel Estimates

Since raw WIM data reported to FHWA or under the LTPP program includes axle weights and distances between axles for each observed vehicle, vehicle classifications provided by the standard axle-spacing algorithms used by the states can be corrected based on the additional information. Also, the data provides enough detail to sub-classify the 13 standard classification vehicle classes into the more detailed classes required by the 2014 CTSW Study. In previous FHWA studies, individual WIM observations have been evaluated for validity based on the reported axle weights and spacings, and either reclassified or rejected according to explicit edit criteria. The team will work with FHWA and the pavement team to update, refine, and adjust these edit criteria for this 2014 CTSW Study based on the collective expertise.

1.3.3.2 Operating Gross Weight (OGW) Distributions for Each Vehicle Class

Based on the refined WIM-record edit criteria, the team will compile the operating weight distributions for each detailed truck class in each state and on each available highway class. Ideally, each state would report enough WIM data to FHWA to allow independent operating weight distributions for each vehicle class on each type of highway. In most cases, however, states collect WIM data on Interstate and arterial highways, especially rural arterial highways. Also, many states do not have enough use by some of the vehicle classes, since some are allowed only by special permit or not at all. Therefore, the team has to group highway types and sometimes states to develop valid OGW distributions for many vehicle classes. The team will take care to distinguish among states with varying weight regulation on Interstate and non-Interstate highways in developing the estimates of OGW distributions.

1.3.3.3 Axle Weight and Type Distributions

Axle weights and types have large effects on pavement deterioration and service life. WIM data provide an excellent source of knowledge about the actual distribution of axle weights for the weight groups in each vehicle class, so that the pavement team does not have to use unrealistic idealized axle weights to typify a weight class. For example, an 80,000-pound 3-S2 is often characterized as having a 12,000-pound steering axle and two 34,000-pound tandem load axles. If the actual distribution of axle weights is 10,000 / 37,000 / 33,000-pound, however, the vehicle will cause significantly more pavement damage than would be estimated by the standard weight distribution.

For consistency with *Pavement ME Design*® traffic input requirements, the team will tabulate axle weight frequencies in 1,000-pound weight groups for steering axles and single load axles, 2,000-pound increments for tandem axles, and 3,000-pound increments for tridem axles, and will develop separate frequency distributions for each weight group and each vehicle class.

1.3.4 Pavement Design and Materials Data

An ESAL-based approach requires only rudimentary pavement design information (pavement type, thickness, and terminal PSI), since those are the only design variables in the ESAL equations. HPMS supplies type and thickness information, and reasonable assumptions can be made about terminal PSI values as a function of highway class.

A NAPCOM-based approach requires primarily observed distress data, since LEFs used in NAPCOM vary mostly by pavement type and distress type, and much less by climate, base type, and other design details. HPMS supplies much of the information needed for these subtle variations, but is only just beginning to supply rudimentary pavement distress data. States are including distress information for an increasing number of pavement sections, thereby improving the accuracy of NAPCOM's cost-share estimates. A fundamental source of uncertainty, however, stems from the HPMS data reporting structure that fails to distinguish between (a) bottom-up and top-down cracking (for both rigid and flexible pavements), and (b) surface and total rutting (for flexible pavements). In each case, LEFs for the grouped distresses vary widely, so not knowing the relative importance of each component compromises the accuracy of the estimates.

Directly applying *AASHTOWare Pavement ME Design*® to a set of pavement sections requires a large number of pavement design details, soil data, and other materials data. The software package includes the climatic data needed for proper program operation, and includes a large quantity of nationally-derived default data for nearly everything else. To properly analyze the sample pavement sections, the team needs to carefully match materials, design, and calibration parameters to a representative set of pavement cross-sections in a representative range of climates. Fortunately, sample LTPP sections have developed all the required information, so will provide the details for the selected sample sections.

1.4 Needs for Future Research and Data Collection

The accuracy of future TSW (and HCA) studies could be improved if the most significant data deficiencies and analytic uncertainties were lessened. The following two sections describe what the study team believes are the greatest sources of uncertainty—both of them current data deficiencies.

1.4.1 Vehicle Classification Data

All three approaches depend heavily upon detailed knowledge about the types of vehicles using the national highway system—knowledge that currently depends on periodic ad hoc analysis of large quantities of WIM and classification data. Both WIM data and classification data have their deficiencies, as does an analysis system that does not continuously compile and evaluate the state-reported data so that it can be compared from year to year and better evaluated as it is submitted.

WIM data collection alone cannot provide vehicle class travel estimates by itself unless the 400 or so stations reported annually: (a) increase in number by at least 10-fold, (b) collect data in all lanes of a multi-lane roadway instead of just the right lane, (c) report data for all vehicles instead of screening out light vehicles, and (d) are located more rationally—either randomly or as part of a truck transportation network. Since most of these improvements are unlikely, vehicle classification data is likely to be a necessity far into the future.

Current classification data falls far short of what is needed for accurate estimation of travel by vehicle class. States report far too few stations, do not adequately review or edit the data, and do not report the weighted importance of each station. FHWA does not attempt to annually compile the information and report detailed truck class travel as part of the Highway Statistics series, which would go a long way to improving the quality of data that now seems to only be compiled on an ad hoc basis every five years or so.

1.4.2 Pavement Condition Data

Applying NAPCOM or a similar model requires information about pavement distresses on the national highway system, since LEFs vary substantially depending upon distress type. Current HPMS section data needs to be more complete in order to apply an approach of this type, and distinctions need to be made between rutting components (surface and total rutting) and among cracking components (bottom-up, top-down, transverse), either through statewide reporting or special studies. The other two approaches do not necessarily require detailed information about pavement distresses observed on the highway system.

1.5 Linkage with Project Plan

Based on evaluating previous studies and available current models and approaches, as described in the previous sections, the 2014 CTSW pavement comparative analysis will focus on a small number representative pavement sections covering a range of locations with varying climates, pavement types, pavement types, and surface thicknesses. The *AASHTOWare Pavement ME Design®* model will be used in this analysis and run for each of these sections to determine a base case of the expected pavement performance under traffic conditions appropriate for each thickness (mix of vehicle types and operating weights as well as truck traffic levels). Locations will be selected that avoid climate extremes and thus represent typical weather effects several groups of states. To the extent possible, Long Term Pavement Performance Program (LTPP) sections will be used as a basis for each sample section and will adjust base case parameters as required to make sure that each sample section represents the pavement performance history that would typically be expected.

For each sample section, the first step will be to perform a base traffic performance analysis. Next, traffic inputs will be varied in ways that represent traffic shifts that occur as a result of the various truck scenarios. This will require a series of runs of *Pavement ME Design®* during which all factors except traffic are held constant.

The multiple runs for each sample section will enable an evaluation of changes in pavement service life as a result of changes in truck travel associated with each modal shift scenario. These changes in pavement service life will be translated into pavement cost changes associated with size and weight scenarios using rudimentary life cycle cost analysis. The approach used in the project plan coincides with the third approach outlined in this report, "Estimating Pavement Performance Directly from Models."

The first approach, "Using ESALs as a Measure of Pavement Damage" is ruled out because it relies on ESALs-- widely discredited because (a) calculating ESALs for tridems has no empirical or theoretical validity since it requires extrapolating a dummy variable, (b) ESALs apply only to roughness, which has many components that vary in their sensitivity to magnitude of axle load, and (c) ESALs derive from the AASHO Road Test, performed long ago as a short term performance test in a single location.

The second approach, "Deriving Pavement Damage Relationships from Pavement Performance Models" is ruled out because it (a) relies upon LEFs derived from an earlier version of *AASHTOWare Pavement ME Design®* that need to be verified using the latest version, and (b) requires an inventory of distress observations that is currently incomplete.

1.6 Comparison of Results with Previous Studies

Unlike most other recent truck size and weight studies, the 2014 CTSW Study contained some scenarios that result in anticipated increases in average axle loads and some that resulted in decreases. In the 2000 CTSW Study, all scenarios resulted in significant reductions in average axle loads, as did the 2004 Western Uniformity Scenario Study and state studies in Minnesota and Wisconsin. Only the Vermont pilot study resulted in increases in average axle loads.

Table 1 contains summary results from each of these recent state, regional, and national studies. Note that, as might be expected, scenarios with lower average axle loads tended to see reduced pavement costs, while cases with higher average axle loads tended to show increased costs. Note, however, that some scenarios resulted in somewhat more subtle interactions between reduced VMT and increased average loads per axle. Average axle loads, after all, are not as important as the distribution of axle loads at the higher ends of the axle load range, given the non-linearity of pavement damage as a function of axle load.

Table 1: Summary Pavement-Related Analysis Results

Study	Vehicles and Weights Analyzed k = thousands of pounds	Change in truck VMT	Change in Pavement Costs
Nationwide Studies			
USDOT, Comprehensive Truck Size and Weight Limits Study (2014)	3S2-88k	-0.6%	+0.4%
	3S3-91k	-1.0%	-2.4%
	3S3-97k	-2.0%	-2.6%
	DS5 33s-80k	-2.2%	+1.8%
	TS7-105.5k	-1.4%	+0.1%
	TS9-129k	-1.4%	+0.1%
USDOT, Comprehensive Truck Size and Weight Study (2000)	3S3-90k; DS9 33s-124k	-10.6%	-1.6%
	3S3-97k; DS9 33s-131k	-10.6%	-1.2%
	RMD-120k; TPD-148k; Triple-132k	-23.2%	-0.2%
	Triple-132k	-20.2%	0.0%
Regional Studies			
USDOT, Western Uniformity Scenario Analysis (2004)	RMD-129k; TPD-129K; Triple-110k	-25%	-4.2%
WsDOT, Wisconsin Truck Size and Weight Study (2009)	3S3-90k	-0.4%	-$14.6 M
	3S4-97k	-1.2%	-$19.9 M
	SU7-80k	-0.5%	-$1.5 M
	DS8-108k	-0.02%	-$16.8 M
	3S3-98k	-0.4%	-$10.2 M
	SU6-98k	-0.04%	-$0.3 M
FHWA, Vermont Pilot Program Report (2011)	SU3-55k; SU4-69k; CS5-90k; 3S3-99k expanded to Interstate for one year	+1.7%, Int -1.5% Non-I	+12%, Int -0.5%, Non-I
MnDOT, Minnesota Truck Size and Weight Project (2006)	3S3-90k	Not Reported	-$1.3 M
	3S4-97k		-$2.2 M
	3S3-2-108k		-$1.3 M
	SU6/7-80k		-$0.6 M

1.7 Summary of Publicly-Available Reports Reviewed

Below are listed all studies reviewed as part of this process including comments about the utility of each study for this project. The first four groups of reports include readily-available reports that were identified through web search or prior knowledge, while group 5 includes studies suggested at the May 29, 2013, 2014 CTSW Study's Public Hearing webinar that in some cases were less easily located.

(1) Using ESALs as a Measure of Pavement Damage

Allen, Gary, Audrey Moruza, and Brian Diefenderfer, Oversize and Overweight Vehicle Studies. Virginia DOT Presentation to the Transportation Accountability Commission, August 4, 2010. http://dls.virginia.gov/GROUPS/transaccount/meetings/080410/oversize.pdf

Researchers used the array of all axle weights from Virginia WIM data, as well as historical expenditure data, to determine an average cost per ESAL-mile of travel. If overweight vehicles are charged only for the extra costs (beyond the legal axle load limit), they would be assessed 3.56 cents per ESAL mile, but that rate needs to be reviewed and updated over time as truck characteristics change. This is one of the few studies that attempted to calibrate relative pavement damage to actual expenditures and imputed costs, but the ESAL assumption requires an updated form of the analysis to reflect better current knowledge. Replacing the ESALs with updated, distress-specific load equivalence factors could overcome this limitation and make the report findings useful for truck size and weight studies.

Bai, Yong, Steven D. Schrock, and Thomas E. Mulinazzi, Estimating Highway Pavement Damage Costs Attributed to Truck Traffic. Mid-America Transportation Center, Report # MATC-KU: 262. 2010.
http://matc.unl.edu/assets/documents/matcfinal/Bai_EstimatingHighwayPavementDamageCostsAttributedtoTruckTraffic.pdf

Sponsored by the USDOT University Transportation Centers Program, this University of Kansas study collected highway data on 41.13 miles of U.S. Highway 50/400 in Kansas, and applied HERS and AASHTO methods to derive average maintenance expenditures per ESAL mile. This became the basis for estimating the additional costs that would be associated with an increase in meatpacking truck traffic. As with the study by Allen et al., the ESAL assumption makes the findings of only general interest to the current 2014 CTSW Study, since we now know that ESALs do not adequately measure the relative effects of tridems, particularly.

Fortowsky, J. Keith, and Jennifer Humphreys, "Estimating Traffic Changes and Pavement Impacts from Freight Truck Diversion Following Changes in Interstate Truck Limits," Transportation Research Record: Journal of the Transportation Research Board, No. 1966, TRB. National Research Council. Washington, D.C. 2006, p. 71.

This TRB paper assumes all pavement damage is directly related to ESALs. We will not be using the assumptions necessary to rely upon ESALs, as cited above, so the study does not help us in our current 2014 CTSW Study.

Hajek, Jerry J, Susan L. Tighe, and Bruce G. Hutchinson. “Allocation of Pavement Damage Due to Trucks Using a Marginal Cost Method.” Transportation Research Record: Journal of the Transportation Research Board, No. 1613, Paper # 98-1283. TRB. National Research Council. Washington, D.C., 2008. http://localroads.wisc.edu/sites/default/files/Allocation%20of%20%20Pavement%20Damage%20Due%20to%20Trucks%20Using%20a%20Marginal%20Cost%20Method.pdf

Ontario Ministry of Transportation determined the marginal cost of providing pavement structure for one additional passage of an ESAL on various roads, and found that a typical additional truck mile resulted in marginal costs that varied significantly across the highway system, ranging from a low of C$0.004 per km ($0.006 / mile) on a southern Ontario freeway to C$0.46 per km ($0.72 / mile) on a local road. Ontario used standard Road-Test-derived ESALs for single and tridem axles, and used elastic layer theory to extend the Road Test results to derive ESALs for other axle groupings. Unlike FHWA’s cost allocation procedures, however, which used average ESAL costs, Ontario’s method uses marginal ESAL costs for the particular heavy vehicles of interest. Thus, the overweight vehicles receive the full benefit of the existence of other heavy vehicles, which is much more significant on major highways than on lightly-traveled local roads-- hence the much higher difference in costs than usually appears in U.S. analysis. We do not suggest reviving the incremental design approach (abandoned for pavement cost analysis in this country in the 1970s), and cannot use the ESAL assumption, so the findings are of only general interest to the current 2014 CTSW Study. The wide scatter in the results, however, by type of roadway provides a cautionary tale to using only a small number of pavement sections without considering the context of a national sample of pavement sections.

Roberts, Freddy L., Aziz Saber, Abhijeet Ranadhir, and Xiang Zhou. Effects of Hauling Timber, Lignite Coal and Coke Fuel on Louisiana Highways and Bridges, LTRC Report No. 398. USDOT. March 2005. http://www.ltrc.lsu.edu/pdf/2005/fr_398.pdf.

Using the 1986 AASHTO Design Guide and standard ESALs shows that heavier tandem axles (up to 48 kip) require additional overlay thickness and reduce pavement life. The current $10 annual overweight fee for an 86 kip 3S2 timber truck in Louisiana should be raised to many times higher per year if the axles are evenly loaded, and much higher, still, if the 48 kip axle is permitted. Allowing 100 kip trucks should not be permitted because pavement overlay costs double compared with an 86 kip truck. The ESAL assumption makes the findings of only general interest to the 2014 CTSW Study, since we will not be assuming that ESALs adequately measure relative effects of axle loads, for the reasons cited above.

Saber, Aziz, Mark Morvant, and Zhongjie Zhang. “Effects of Heavy Truck Operations on Repair Costs of Low Volume Highways”. Presented at TRB 200 Annual Meeting, on CD-ROM of 2009 Meeting Proceedings. January 2009. http://www.google.com/url?sa=t&rct=j&q=&esrc=s&source=web&cd=1&ved=0CC0QFjAA&url=http%3A%2F%2Fsites.kittelson.com%2FIUHUserFee%2FDownloads%2FDownload%2F21822&ei=EMnmUcycNPSq4AO_vIGgAQ&usg=AFQjCNFBV8fRPgXBkJRE1zI8ICCQfFHovQ&sig2=IZmW61v6vQcBodLm8xthgA&bvm=bv.49405654,d.dmg

Using standard ESALs, the study analyzed two vehicle types and three gross weights and concluded that 100 kip sugarcane trucks should be paying an annual fee of many times higher *than their current annual fee if they are use the standard 3S2 configuration, but would not need an increase in that fee if they use a 3S3 configuration. The ESAL assumption makes the findings of only general interest to this 2014 CTSW Study, for the reasons cited above.*

Study of Impacts Caused by Exempting Currently Non-exempt Maine Interstate Highways from Federal Truck Weight Limits, Appendix C: Pavement Cost Impacts, Development Process for the Study Network, Wilbur Smith Associates Study Team, June 2004.

This report assumed that all pavement damage is related to ESALs, so has limited information useful to this 2014 CTSW Study, for the reasons cited above.

Wisconsin Truck Size and Weight Study: Final Report. Prepared for Wisconsin Department of Transportation by Cambridge Systematics with National Center for Freight and Infrastructure, University of Wisconsin- Madison and Others. June 15, 2009. http://www.topslab.wisc.edu/workgroups/tsws/deliverables/FR1_WisDOT_TSWStudy_R1.pdf

Pavement analysis considered differential effects of traffic under various temperature and moisture conditions, and effects of load and non-load factors, but assumed that all vehicle-related damage is measured by and related to traditional ESALs and that the Road Test ESALs can be extended to tridems by extrapolating a dummy variable from a regression equation. The ESAL assumption makes the findings of only general interest to this 2014 CTSW Study, for the reasons cited above.

(2) Deriving Pavement Damage Relationships from Pavement Performance Models

Bannerjee, Ambarish, Jorge A. Prozzi, and Prasad Buddhavarapu, A Framework for Determination of load Equivalences Using DARWin-ME, Paper Number 13-1770, TRB 2013 Annual Meeting, on CD-ROM of 2013 Meeting Proceedings. January 2013.

The study used DARWin-ME to compute Equivalent Damage Factors (EDF) consisting of two partial factors: Axle Load Factor (ALF) and Group Equivalency Factor (GEF), based on pavement responses that result in the same distress level, following a procedure used earlier by an FHWA research project. The overall load equivalency for a truck is equal to the sum of the EDFs for each constituent axle group. Three AC distresses were analyzed: rutting, fatigue cracking, and roughness. After analyzing EDFs for a wide range of AC pavement designs, the authors concluded that there is little evidence that EDFs are affected by structural capacity for the latter two distress types. For rutting, however, EDFs had in inverse relationship with thickness for single axles, while EDFs of multi-axle groupings peaked for structural numbers between 3.5 and 4.0. The findings verify findings of the LEF derivations for the updated NAPCOM and PaveDAT models, and variation of thickness adds a nuance that will be useful in this base pavement section design.

Chatti, Karim. “Effect of Michigan Multi-Axle Trucks on Pavement Distress.” Michigan DOT and Michigan State University, Final Report, Executive Summary, Project RC-1504. February 2009. http://www.michigan.gov/documents/mdot/MDOT_Research_Report_RC-1504__ExecSum_272183_7.pdf

*Laboratory studies were used to determine axle factors (AF) for each tridem (and larger) grouping at each weight. AFs were defined as the ratio of damage of a tridem, for example, to a single axle weighing one-third as much. The AFs were multiplied by the ESALs for each axle grouping on a truck and subbed to derive a truck factor (TF). When combined with empirical data on selected Michigan highways with flexible pavements, the study concluded that tridems (and n-groups) had less relative effect on cracking but more relative effect on rutting than single or tandem axles of an equivalent weight per axle***.*** *The results of this study could be useful in extending study findings to quadrem and larger axle groupings.*

Ioannides, Anastasios M., and Lev Khazanovich, "Load Equivalency Concepts: A Mechanistic Reappraisal." Transportation Research Record: Journal of the Transportation Research Board, No. 1388, pp. 42-51. TRB. National Research Council. Washington, D.C., 1993.

The paper reviews the evolution of load equivalency concepts, both prior to and after the 1958 - 1960 AASHO Road Test. The Road Test's mechanistic-empirical ESAL concept varies considerably from the purely mechanistic equivalent single-wheel load (ESWL) and equivalent single-axle radius (ESAR) approaches. The latter mechanistic approach, however, appears to offer advantages over either of the other two approaches. When the paper was written, load equivalency factors (LEFs) were vital for designing pavement for mixed traffic, since they allowed the relative effects of each vehicle to be incorporated into design. To the extent that mechanistic-empirical models become prevalent for design, however, a truck size and weight study can avoid the use of LEFs if there is no need to report the relative effects of various vehicles on pavement life.

Nicholas, John, Roger Mingo, Mark Berndt, and Eulois Cleckley, Pavement Damage Analysis Tool (PaveDAT) for Overweight Truck Permit Calculation, Talking Freight Seminar Series, June 12, 2012. https://www.fhwa.dot.gov/planning/freight_planning/talking_freight/june202012.cfm

PaveDAT builds upon the National Pavement Cost Model (NAPCOM) and the improvements made to it in recent work by FHWA. New damage models were based on running MEPDG thousands of times, systematically varying traffic to determine the relative effect that each type and weight of axle has on pavement deterioration in a full range of pavement types in a full range of climatic conditions. PaveDAT is a simplified version of the complicated, nationally representative NAPCOM model, but uses the same relative damage factors. These new load equivalence factors (LEFs) are similar in concept to the traditional ESAL concept, but vary widely across the important distresses for each type of pavement. PaveDAT was applied in the District of Columbia in a recent assessment of the costs associated with overweight vehicles.

1997 Federal Highway Cost Allocation Study (HCAS) Final Report, FHWA. http://www.fhwa.dot.gov/policy/hcas/final/index.htm

New pavement costs were allocated to vehicles based on the same minimum pavement approach used in the 1982 HCAS, wherein costs of pavement thickness above a sidewalk or bikeway standard are assigned to vehicles based on traditional ESALs. Costs for pavement reconstruction, rehabilitation, and resurfacing (about 25% of all federal obligations) were allocated using the latest version of NAPCOM, following the 1982 approach of assigning costs to vehicles based on their estimated contribution to each pavement distress weighted by the *importance of each distress to the need to repair or replace a pavement. For both types of cost, FHWA developed estimates of travel by vehicle class and operating weight group. Unlike in 1982, however, FHWA used a simplified version of estimating axle weights for each operating group and vehicle configuration, rather than using an array of all observed axle weights. The team intends to use an array of axle weights for each weight group and configuration, rather than a regression equation describing the average weight of each axle.*

Highway Cost Allocation Study: 2013 - 2015 Biennium, Final Draft. Prepared for Oregon Department of Administrative Services, Office of Economic Analysis by ECONorthwest, with R.D. Mingo and Associates, Jack Faucett Associates, HDR Engineering, and Mark Ford. January 2013. http://www.oregon.gov/DAS/OEA/docs/highwaycost/2013report.pdf

Every two years, Oregon evaluates its anticipated highway program and its current highway usage patterns to determine how to adjust user fees to match highway user cost responsibilities. As in 2011, a new version of NAPCOM / PaveDAT was adapted to vehicle classes weight categories, and simplified highway classes, was updated to include the most recent Oregon WIM and pavement condition data, and was used for pavement cost allocation.

Trucks and Infrastructure Maintenance Costs. State Smart Transport Initiative. Undated http://ssti.us/wp/wp-content/uploads/2011/11/Trucks%20and%20Infrastructure%20Maintenance%20Costs.pdf

Compiles truck estimated per-mile pavement costs from a variety of cited sources, including CBO and FHWA reports. May be of general interest to this 2014 CTSW Study as a point of comparison for baseline per-mile pavement cost estimates.

U.S. Department of Transportation Comprehensive Truck Size and Weight Study, FHWA. August 31, 2000. http://www.fhwa.dot.gov/reports/tswstudy/

The study found that pavement wear is an important area of interest in conducting truck size and weight studies because rough pavement affects the cost of travel via vehicle operating costs, delay costs, and crash costs. Pavement wear increases with axle weights and the number of axle loadings applied to a pavement. To analyze the magnitude of changes in pavement wear given alternative mixes of weights and axle configurations, the study used the same version of NAPCOM that was used in the 1997 HCAS, using the same baseline estimates of travel by vehicle class and operating weight group and the same simplified version axle weight distributions. The team recommends using an array of axle weights for each weight group and configuration for this 2014 CTSW Study, but will attempt to modify the older study's approach of using axle weights and types as the primary units of analysis in favor of considering all the axle weights and types in each operating weight group as a single unit. Vermont Pilot Program Report, FHWA Report to Congress Required by P.L. 111-117, 2012. http://www.ops.fhwa.dot.gov/freight/sw/reports/vt_pilot_2012/vt_pilot.pdf

Vermont raised size and weight limits on its Interstate highways for one year beginning in December 2009. This study estimated traffic and infrastructure impacts and energy consumption and compared them to the pre-pilot (control) case. For pavements, the study team used an expanded version of the PaveDAT model, with its newly derived, distress-specific LEFs. Since *traffic shifted mostly to 4-axle single units and 6-axle combination trucks as a result of the temporary allowance of 51 kip tridems on the Interstate system, pavement damage attributable to these vehicle classes increased considerably. Pavement damage on the Vermont Interstate system increased by 12 percent, which translates to significant increases in pavement maintenance and repair costs and more frequent work zones. There was a negligible decrease (less than 0.5%) in pavement damage off the Interstate system.*

(3) Estimating Pavement Performance Directly from Models

Timm, David H., Rod E. Turochy, and Kendra D. Peters. *Correlation between Truck Weight, Highway Infrastructure Damage and Cost*. Auburn College of Engineering for FHWA, DTFH61-05-Q-00317, Subject No 70-71-5048. October 2007. http://www.eng.auburn.edu/files/centers/hrc/DTFH61-05-P-00301.pdf

Using MEPDG for a small sample of pavement sections, the study team determined the time until terminal pavement distress for a base case of traffic, then under three different loading scenarios: shifting entire weight distributions toward heavier axles, adding specific heavier axles, and changing the GVW from 80,000 to 97,000 lbs. while adding an axle to the rear tandem group. The first scenario showed very large decreases in pavement life (and increases in cost), the second showed significant cost increases when the number of added heavy axles exceeds 10% of the number of legally loaded axles, and the third showed no practical difference. Mechanistic analysis outside of MEPDG showed only slight difference in pavement response, confirming the finding of insignificant changes in pavement costs. The authors noted that their findings represent a limited set of conditions and that results were pavement-specific. They recommended that future work identify other loading scenarios for MEPDG simulation and establish a methodology to more accurately predict changes in loading spectra. FHWA followed up on these recommendations and initiated a project that systematically varied axle loadings for a larger number of pavement sections, and derived a general set of findings that could apply to any set of traffic shift scenarios (see Nichols et al., above). If the team cannot successfully use the primary approach proposed in this work plan, based on the pilot pavement section, the findings of this study and especially the follow-up study are directly applicable to the proposed back-up approach.

Timm, David, and Kendra Peters. *Effects of Increasing Truck Weight Limit on Highway Infrastructure Damage*. ICWIM 5, Proceedings of the International Conference on Heavy Vehicles: 5th International Conference on WIM of Heavy Vehicles, March 2013. http://road-transport-technology.org/HVTT10/Proceeding/Papers/Papers_WIM/paper_123.pdf

Using MEPDG, no change in pavement life was found under idealized vehicle loading conditions when the same weight of freight was carried on a 97,000-lb vehicle or an 80,000-lb. vehicle. The idealized loading assumption makes the study unusable for this analysis, since we will be using actual observed axle weights as the basis for analysis, and the results are likely to be very different.

Tirado, Cesar, Cesar Carrasco, Jose M. Mares, Nasir Gharaibeh, Soheil Nazarian, and Julian Bendaña. "Process to Estimate Permit Costs for Movement of Heavy Trucks on Flexible Pavements." Transportation Research Record: Journal of the Transportation Research Board, 2154, pp. 187-196. TRB. National Research Council. Washington, D.C., 2010. http://pustaka.pu.go.id/files/pdf/BALITBANG-03-C000066-610032011103843-process_to_estimate_permit_cost.pdf

The paper describes use of a primary-response model, coupled with damage predictions from a mechanistic-empirical analysis, to quickly estimate relative levels of distress caused by particular combinations of axle loads. It is interesting for the current study, since it does not vary traffic within the M-E model, but external to the model, thus allowing much more rapid estimation of the relative effects of axles based solely on their primary responses. The approach certainly has merit, but expanding it to this 2014 CTSW Study would require a fairly major research effort that is probably beyond the scope, since we do not have enough calendar time or staff-hour budget to substantially extend the findings of AASHTOWare Pavement ME Design
Zapata, C., and C. Cary. Integrating the National Database of Subgrade Soil-Water Characteristic Curves and Soil Index Properties with the MEPDG. National Cooperative Highway Research Program Project 9-23B, Preliminary Draft Final Report, National Research Council, Washington, D.C., 2012. http://onlinepubs.trb.org/onlinepubs/nchrp/docs/NCHRP09-23B_FR.pdf

Findings of this report, and, more importantly, the associated ASU Soil Maps software tool can be used to establish what the substructure properties will be for any of the sites analyzed in any (and within any) of the four LTPP climatic regions evaluated. The team will use this report in compiling the data necessary for each pilot section.

(4) Description of Modeling Techniques, Potential Improvements, and Inputs

Cenek, P., R. Henderson, I. McIver, and J. Patrick, Modelling of Extreme Traffic Loading Effects. Opus Central Laboratories for New Zealand Transport Agency, Research Report 499. October 2012. http://www.nzta.govt.nz/resources/research/reports/499/docs/499.pdf

The study investigated the premature failure of low-volume, low-strength roads that were sometimes associated with significant increases in heavy truck traffic on New Zealand highways, as might occur with road detours or with new mining or forestry operations. A key finding of the study was that extreme traffic loading does not immediately show increased distress or added maintenance costs. Thus, traffic deterioration models are more useful than examining historical pavement management data in assessing vehicle-related pavement costs. Although the findings are not directly applicable to this 2014 CTSW Study, the amount of scatter in the data as well as the length of time needed to observe accelerated wear serve as cautionary tales in analyzing the effects of heavy trucks on pavements via solely empirical data.

Chatti, Karim, Hassan Salama, and Chadi El Mohtar. “Effect of Heavy Trucks with Large Axle Groups on Asphalt Pavement Damage.” Presented at 8th International Symposium on Heavy Vehicle Weights and Dimensions, Johannesburg, South Africa, March 2004. http://road-transport-technology.org/Proceedings/8%20-%20ISHVWD/EFFECT%20OF%20HEAVY%20TRUCKS%20WITH%20LARGE%20AXLE%20GROUPS%20ON%20ASPHALT%20PAVEMENT%20DAMAGE%20-%20Chatti.pdf

Laboratory studies of a particular asphalt mix subject to pulse loadings representing various axle groupings (1 to 8 axles per group, 3.5-foot spacing) indicates that the normalized distress *per ton goes down as the number of axles in a group goes up. Linear regression of LTPP distress and WIM data confirms this observation. The results of this study could be useful in extending study findings to quadrem and larger axle groupings. It is likely that considering actual axle groups, rather than arbitrarily dividing large groups of axles into tridems and tandems, would increase the accuracy of pavement damage analysis, and we will attempt to partially incorporate this approach.*

Mallick, R., S. O'Brien, D. Humphrey, and L. Swett, Analysis of Pavement Response Data and Use of Nondestructive Testing for Improving Pavement Design, First Year Report 04-1A, Maine Department of Transportation, August 2006.

This report presents a description of instrumentation at the first fully instrumented flexible test pavement test section in Maine. Strain gauges were installed at the bottom of the HMA layer as well as in the subbase and subgrade, while pressure cells were installed in the subbase and the subgrade. Other instruments consist of thermocouples, moisture and thermal resistivity probes. Models relating temperature at two depths of the HMA layer with ambient temperature and solar radiation were developed. Stress/strain data were collected using a loaded truck running at different speeds at different temperatures. The response pulses at different layers were modeled with the Haversine equation and its slight variations. The effect of speed on the time of loading at the different layers was examined, to develop equations for predicting time of loading for laboratory testing, for example, for different traffic speeds for similar structures in Maine. The effect of time of loading on HMA strains, especially at higher temperatures, was well manifested in the measured data. Comparisons of predicted versus measured responses showed that the tensile strains in the HMA layer match with the predicted ones at lower temperature and lower time of loading. For subbase, the stresses were under predicted, whereas predicted strains matched quite well with the measured strains. In the case of subgrade, both the stresses and the strains were consistently higher than the predicted values - the difference increased with an increase in time of loading and temperature. The results from this ongoing study provide much needed information on response of typical reconstructed pavement in Maine, which can be used for laboratory testing and theoretical modeling, as well as in structural design using mechanistic procedures. This section could be used as one of the sites for analysis for the Northeast zone since the Maine DOT has been collecting real-time data on that site since 2006. Specifically, they have the following information available on this section (on Rt. 15) that could be used with complete data for one of the pavement analysis sites: test section cross-sectional layer features (layer thickness and material types); material properties for the subgrade, base, and HMA courses; temperature data for the mid-depth of the asphalt base and at the bottom of the asphalt base; pavement mechanical response data on the speed versus time of loading in the different pavement layers.

Oh, Jeongho, E.G. Fernando, and R.L. Lytton. "Evaluation of Damage Potential for Pavements Due to Overweight Truck Traffic, Journal of Transportation Engineering," 133(5), 308-317. DOI:10.1061/(ASCE)0733-947X(2007)133:5(308). 2007 http://www.academia.edu/937877/Evaluation_of_Damage_Potential_for_Pavements_due_to_Overweight_Truck_Traffic

Researchers installed multidepth deflectometers (MDDs) along a section of highway in Brownville, where overweight trucks were routinely allowed starting in 1998, in order to *establish a correlation between field measurements of pavement response to overweight trucks and the observed critical strains of rutting and fatigue cracking. The analysis was done in the overall framework of cross-anisotropic modeling of pavement response. The researchers found excellent correlation between damage and primary response, meaning that primary response is a good proxy for expected pavement damage. The study could be used to check consistency of their findings with the AASHTOWare Pavement ME Design® model, but we do not have enough calendar time and did not propose enough effort to second-guess the models incorporated in AASHTOWare Pavement ME Design®.*

Sadeghi, J. M., and M. Fathali. Deterioration Analysis of Flexible Pavements under Overweight Vehicles. Journal of Transportation Engineering, 133(11), 625-633. DOI:10.1061/(ASCE)0733-947X(2007)133:11(625). 2007.
http://www.nlcpr.com/Deterioration%20Analysis%20of%20Flexible%20Pavements.pdf

The authors used layer theory, following the Burmeister approach, to derive operational life reduction factors for two-axle and three-axle single unit trucks and for 3S2s. Not really credible for our purposes given alternative available models. We have opted to use the AASHTOWare Pavement ME Design® model, and do not have enough calendar time and did not propose enough effort to second-guess the incorporated damage models in AASHTOWare Pavement ME Design®.

Schwartz, Charles W., Rui Li, Sung Hwan Kim, Halil Ceylan, and Kasthurirangan Gopalakrishnan. Sensitivity Evaluation of MEPDG Performance Prediction. NCHRP Project 1-47, Final Report. TRB. December 2011.
http://onlinepubs.trb.org/onlinepubs/nchrp/docs/NCHRP01-47_FR.pdf

The study systematically varied all the user inputs for the MEPDG model to determine the sensitivity of the pavement performance predicted by the model to the variability of the input factors for five types of pavements-- new HMA, HMA over a stiff foundation, new JPCP, JPCP over a stiff foundation, and new CRCP-- and five climate types (the usual four, plus temperate). Although design inputs were varied, traffic composition was not-- only AADTT and operating speed were varied. The study derived normalized sensitivity indices (NSIs) for each distress for each input variable, expressing the percentage change in the normalized distress divided by the percentage change in the design input. Key findings were that design inputs for the surface layers were the most important; longitudinal cracking, alligator cracking, and AC rutting were substantially more sensitive to inputs than were IRI and thermal cracking; design input sensitivities for thermal cracking had little overlap with the design input sensitivities for the other distresses; and little thermal cracking occurred when binder grades were properly matched to the climate. The study will be helpful in designing base case pavement sections, but the lack of traffic variations make it less useful for the overall analysis for the current 2014 CTSW Study.

(5) Not Directly Usable but Supplying Background Information

Acimovic, Benjamin, Leela Rejaseker, and Reza Akhavan. *Forensic Investigation of Pavement Failure on Vasquez Boulevard.* Colorado DOT Research Branch, Report No. CDOT-2007-7. May 2007.
http://www.google.com/url?sa=t&rct=j&q=&esrc=s&source=web&cd=1&ved=0CC8QFjAA&url=http%3A%2F%2Fwww.coloradodot.info%2Fprograms%2Fresearch%2Fpdfs%2F2007%2Fvasquez.pdf%2Fat_download%2Ffile&ei=EVHlUa3ELob84APO6YCIBA&usg=AFQjCNGAHQ0JgEiOrUiU4nscsx7X8rXdpQ&sig2=jKCllzvuu3SZuRhv9bHIVA&bvm=bv.48705608,d.dmg

Vasquez Boulevard in Commerce City, Colorado, as part of U.S. 6, provides a main trucking route in the I-25 corridor for overweight and over-height trucks. After reconstruction in 2001, parts of the pavement showed severe rutting in less than one year. Pavement failure was found to be related to repeated heavy loads, exposure of a layer constructed in the 1940s that did not contain an anti-stripping agent, inexperience with the stone-matrix asphalt technique used in the rehabilitation, and variable mix gradation and AC content. Although the study confirms that heavier trucks do, indeed, contribute to accelerated pavement wear, especially with faulty pavement designs, the findings are too general to contribute to the analysis methods we rely upon in the current 2014 CTSW Study.

Barnes, Gary, and Peter Langworthy. "Per Mile Costs of Operating Automobiles and Trucks." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1864. TRB. National Research Council. Washington, D.C., 71-77. 2004. Available online in pre-published form at http://www.hhh.umn.edu/centers/slp/pdf/reports_papers/per_mile_costs.pdf

Citing other studies, the report concludes that IRIs below 80 (PSIs greater than 3.5) add nothing to vehicle operating costs, but IRIs of 170 (PSI 2.0) result in 2.5 cents per mile in additional operating cost. The additional cost derives from reduced vehicle life and in increased repair and maintenance costs. User costs are not being modeled in the current 2014 CTSW Study, so the findings are of only general interest.

Dodoo, Nii Amoo, and Neil Thorpe. "Road User Charging for Heavy Goods Vehicles." Presented at 7th International Symposium on Heavy Vehicle Weights and Dimensions, Delft, The Netherlands, June 2002. http://road-transport-technology.org/Proceedings/7%20-%20ISHVWD//Road%20User%20Charging%20For%20Heavy%20Goods%20Vehicles%20-%20%20Dodoo.pdf

Although many countries in Europe and North America have explored charging vehicles based on operating axle weights and the associated pavement damage, charging for actual damage at the point of use through use of WIM or other scales becomes problematic because of the high cost of installing weight station and the poor correlation between static and dynamic axle load. The authors instead propose an on-board system consisting of dynamic axle-load measurement combined with vehicle location measuring devices (now widely known as GPS systems). Interesting approach, but well beyond the scope of this analysis in this 2014 CTSW Study, since we have not been asked to consider alternative user-fee charging mechanisms.

Fernando, Emmanuel G. “Investigation of the Effects of Routine Overweight Truck Traffic on SH4/48.” Texas Transportation Institute, Project 0-4184, Summary Report. April 2006. http://ftp.dot.state.tx.us/pub/txdot-info/rti/psr/0-4184-s.pdf

After Texas authorized 125,000-lb trucks to routinely use a state highway in Brownsville, TTI collected data to assess the impact of overweight trucks on that route. They first used ground-penetrating radar to estimate layer thicknesses and to subdivide the route into uniform subsections, where they used falling weight deflectometer tests to monitor load response over time. They also took cores at selected locations to both verify the penetrating radar thickness estimates and to characterize asphalt concrete properties. The research found good correlations between AC moduli back calculated from static and dynamic analysis and that the additional ESALs from overweight truck traffic will likely result in accelerated pavement deterioration. The study could be used to check consistency of their findings with the revised AASHTOWare Pavement ME Design® model, but we do not have enough calendar time and did not propose enough effort to second-guess the models incorporated in AASHTOWare Pavement ME Design®.

Gibby, A. R., Ryuichi Kitamura, and Huichun Zhao. “Evaluation of Truck Impacts on Pavement Maintenance Costs,” Transportation Research Record: Journal of the Transportation Research Board, No. 1262, pp. 48 - 56. TRB. National Research Council. Washington, D.C., 1990. http://publications.its.ucdavis.edu/publication_detail.php?id=1008

The study randomly selected 1,100 one-mile sections of state highways, collected data on traffic, weather, geometric conditions, and pavement maintenance costs on those sections, and used that data to develop a model of pavement maintenance costs. Incremental maintenance costs were expressed in terms of average annual maintenance cost per vehicle. An interesting analysis that is not directly applicable to the current 2014 CTSW Study.

Hernandez, Sarah, Andre Tok, and Stephen G. Ritchie, “Integration of Weigh-in-Motion and Inductive Signature Technology for Advanced Truck Monitoring.” Institute of Transportation Studies, University of California, Irvine, Report # UCI-ITS-WP-13-3, August 2013. http://www.its.uci.edu/its/publications/papers/ITS/UCI-ITS-WP-13-3.pdf

The study points out the high rates of error when inductive loop technology alone is used to classify trucks and demonstrates how the error rates can be reduced by including axle weight data from WIM. Further, the study explores using inductive loop devices with high sampling rate detector cards to identify characteristic body type signature. This allows users to identify truck body types and dramatically reduce classification vehicle classification errors. While the technology is not currently used by states in reporting WIM data to FHWA, this study provides analysis of error rates for several vehicle classes that the team can compare to our error rates, and perhaps use to refine our WIM data classification algorithm.

Luskin, David, and C. Michael Walton. Effects of Truck Size and Weights on Highway Infrastructure and Operations: A Synthesis Report. Center for Transportation Research: The University of Texas at Austin. Report No. FHWA/TX-0-2122-1. March 2001 http://www.utexas.edu/research/ctr/pdf_reports/2122_1.pdf

In reviewing a number of truck size and weight studies, including FHWA's 2000 study, the authors found that shifting away from the dominant 3S2 and increasing gross vehicle weights would not necessarily increase pavement costs, and might make them lower, but it would likely increase bridge costs. Safety effects were inconclusive. The ESAL assumption, as well as the wide range of diversion assumptions, makes the findings of only general interest to the current 2014 CTSW Study, although it does illustrate that heavier vehicle weights do not automatically result in higher pavement costs.

Papagiannakis, Athanassios, Nasir Gharaibeh, Jose Weissmann, and Andrew Wimsatt. Pavement Score Synthesis. Texas Transportation Institute, Report No. FHWA/TX-09/0-6386-1. January 2009. http://d2dtl5nnlpfr0r.cloudfront.net/tti.tamu.edu/documents/0-6386-1.pdf

The synthesis summarizes the use of pavement scores by states, including rating methods and how the scores are used for recommending pavement maintenance and rehabilitation actions. Some states considered only the dominant distress in rehab strategies, while others considered all the distresses present. Most states considered both range and severity of distress. Differences in rating systems make comparison of overall pavement conditions among states invalid. Good overview of rating systems, but not of direct relevance to this 2014 CTSW Study.

Regehr, Jonathon David, Exposure Modelling of Productivity-Permitted General Freight Trucking on Uncongested Highways. Doctoral Dissertation for University of Manitoba Civil Engineering Department. October 2009. http://hdl.handle.net/1993/3167

The paper describes a methodology for improving estimates of LCV exposure data for the Canadian Prairie Region. The dataset for the study integrated output from a classification algorithm, field observations, and industry intelligence. The classification algorithm is of particular interest to this 2014 CTSW Study, since it broke the LCV classes into a larger number of vehicle types than FHWA commonly uses, thereby allowing a higher degree of certainty in some of the most important LCV classes. The team will use the algorithm to refine the WIM analysis and to help us evaluate how many of vehicle classes to use in the analysis.

Regehr, J. D, J. Montufar, and D. Middleton. Applying a Vehicle Classification Algorithm to Model Long Multiple Trailer Truck Exposure. Published in IET Intelligent Transport Systems, February 2009. Abstract available at: http://digital-library.theiet.org/content/journals/10.1049/iet-its.2008.0066

The paper describes an algorithm also described in the Regehr dissertation. The team will use the algorithm to refine the WIM analysis and to help us evaluate how many of vehicle classes to use in this analysis.

Rouen, Chhooeuy, and Mom Mony. "Damage Effects of Road Pavements Due to Overloading in Cambodia", Academia.edu, Undated. http://www.academia.edu/1375429/Damage_Effects_of_Road_Pavements_due_to_Overloading_in_Cambodia

Synthesis of previous studies in many other countries shows that truck overloading is a serious problem that can greatly increase pavement costs. Not directly usable for this 2014 CTSW Study, since there is insufficient information about the axle loads, the pavements, or the materials.

129,000 Pound Pilot Project: Report to the 62nd Idaho State Legislature. Idaho Transportation Department (IDT). January 2013. http://itd.idaho.gov/newsandinfo/Docs/129000%20Pound%20Pilot%20Project%20Report.pdf

Idaho raised the operating GVW limit from 105.5 kips to 129 kips as a pilot project on selected routes in the state in 2003, 2005, and 2007. The 105.5 kip trucks typically operated with 8 axles, while the 129 kip trucks typically operated with 10 or 11 axles. The state legislature asked IDT to study the impact of the pilot on safety, bridges, and pavements and report to the legislature every three years. Participating trucking companies reported making 264,169 trips by 1,359 trucks between 2004 and 2012, and ITD did not observe any significant effects on safety, bridges or pavements, while participating trucking companies reported great savings in costs and number of trips. Normal maintenance and repair activities occurred during the pilot, but ITD did not tabulate their relative frequency on the pilot and non-pilot routes, so one cannot conclude that there was no effect on pavement or bridge deterioration, only that regular maintenance and repair activities were able to compensate for any change in deterioration rates. That lack of data, plus the small sample size of routes, trips, and pilot duration make any conclusions from the project somewhat tentative at this point.

Estimating Truck-Related Fuel Consumption and Emissions in Maine: A Comparative Analysis for a 6-axle, 100,000 Pound Vehicle Configuration, American Transportation Research Institute, September 2009.

The performance of a 6-axle vehicle configuration operating at a maximum GVW of 100,000 pounds was analyzed over two roughly parallel routes between Augusta and Brewer, Maine. The existing route (Route 9) reflects current conditions where trucks greater than 80,000 pounds GVW are not allowed on I-95 north of State Route 3 due to federal weight restrictions. The alternative route (I-95) assumes trucks up to 100,000 pounds GVW would be allowed to travel on I-95 north of State Route 3. This report relates only very limited information that relates to the impact of increased truck loads on pavement response. It deals instead with energy consumption and emissions.

"How Vehicle Loads Affect Pavement Performance." Wisconsin Transportation Bulletin No. 2, Undated. http://epdfiles.engr.wisc.edu/pdf_web_files/tic/bulletins/Bltn_002_Vehicle_Load.pdf

Explains ESALs and the basics of pavement fatigue and pavement strength to a lay audience. The ESAL assumption makes the findings of only general interest to this 2014 CTSW Study, but the explanation of why pavement damage goes up faster than axle weight could be helpful in summary reports intended for a non-technical audience

Research Projects – Multiple documents. Multimodal Transportation & Infrastructure Consortium. Available at http://www.mticutc.org/research/research-projects/

Several projects underway appear to have some possible relevance, but all of the final reports for these projects are listed as "Coming Soon" so will not be available soon enough for this 2014 CTSW Study.

"Section 5 - Truck Weight Monitoring", Traffic Monitoring Guide. Federal Highway Administration, May 1, 2001 http://www.fhwa.dot.gov/ohim/tmguide/tmg5.htm

Describes the truck weight monitoring program under which states collect and report WIM data. Excellent reference material for using and understanding the WIM data that we will use in the 2014 CTSW Study.

(6) Additions Suggested During May 29, 2013 Webinar

Di Cristoforo, R., Regehr, J.D., Germanchev, A., and Rempel, G. (2012). "Survival of the Fittest: Using Evolution Theory to Examine the Impact of Regulation on Innovation in Australian and Canadian Trucking," Heavy Vehicle Transport Technology 12, Stockholm, Sweden. *This publication is not directly related to pavement issues. It examines the impact of regulation on trucking in Australia and Canada by applying evolution theory and deals more with regulatory issues.*

Jablonski, B., Regehr, J.D., Kass, S., and Montufar, J. (2010). "Data Mining to Produce Truck Traffic Inputs for Mechanistic-Empirical Pavement Design," 8th International Transportation Specialty Conference, Canadian Society for Civil Engineering, Winnipeg, Manitoba. *This presentation is related to the overall 2014 CTSW Study, but not overly useful for our task.*

Jablonski, B., J.D. Regehr, G., Rempel, T. Baumgartner, A, Nuñez, K. Patmore, M. Moshiri, H. Hernandez, and J. Montufar, J. (2010). "Traffic Data Requirements for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures in Manitoba," prepared for Manitoba Infrastructure and Transportation by UMTIG in association with Regehr Consulting. *This paper is useful from its title but is propriety (a consulting report) and therefore not publicly-available.*

Malbasa, A., Regehr, J.D., and Clayton, A. (2005). "A Performance-Based Approach to On-Road Regulatory Compliance of Commercial Vehicle Operations in Manitoba," UMTIG, prepared for the Compliance and Regulatory Services Branch, Manitoba Transportation and Government Services. *This paper seems related more to compliance from its title, plus it is propriety (a consulting report) and therefore not publicly-available.*

Montufar, J., J.D. Regehr, G. Rempel, T. Baumgartner, and B. Jablonski (2008). "The Impacts of Increased Truck Gross Vehicle Weights: Environmental Scan," Montufar & Associates and UMTIG, prepared for Alberta Infrastructure and Transportation. *This paper may be useful from its title but is propriety (a consulting report) and therefore not publicly-available.*

Montufar, J., J.D. Regehr, C. Milligan, and M. Alfaro (2011). "Roadbed Stability in Areas of Permafrost and Discontinuous Permafrost: A Synthesis of Best Practices," prepared for Transport Canada – Surface – Prairie and Northern Region by Montufar & Associates in association with Regehr Consulting and UMTIG. *This paper is actually most pertinent to railroads in the northern Canadian context and a publicly-available paper is forthcoming in ASCE Journal of Cold Regions Engineering.*

Radstrom, B., Regehr, J.D., Arango, J., Steindel, M., Rempel, G., Jablonski, B., Montufar, J., and Clayton, A. (2007). "Traffic on Manitoba Highways 2006," University of Manitoba Transport Information Group, prepared for the Traffic Engineering Branch, Manitoba Infrastructure and Transportation. *This paper may be marginally-useful for its Level 1traffic data, specifically the percentages and load range for 3-S2 and B-train truck types.*

Regehr, J.D. (2012). “Truck Exposure to Inform Size and Weight Policy Decisions,” presentation prepared for the Transportation Research Board Annual Meeting, Washington, D.C. *This presentation is related to compliance.*

Regehr, J.D. (2011). “Understanding and Anticipating Truck Fleet Mix Characteristics for Mechanistic-Empirical Pavement Design,” Transportation Research Board Annual Meeting CD-ROM, Washington, D.C. *This paper analyzes vehicle classification data to support the implementation of the Mechanistic-Empirical Pavement Design Guide (MEPDG). A cluster analysis and expert judgment are applied to vehicle classification data from Manitoba to produce six jurisdiction-specific truck traffic classification groups (TTCGs). These groups are used to estimate truck volumes by class at locations where no site-specific classification data exist. The unique vehicle classification distributions evident from these groups, particularly the relative predominance of six-axle tractor semitrailers and multiple-trailer trucks within the fleet, demonstrate the importance of developing truck traffic data inputs based on local conditions and expertise. This publication is relevant to pavements, but specifically looks at vehicle class rather than weight.*

Regehr, J.D. (2010). “Leveraging Truck Traffic Data from Mechanistic-Empirical Pavement Design to Support Other Transportation Engineering Decisions,” presentation prepared for the North American Travel Monitoring Exposition and Conference, Seattle, Washington. *This presentation is related by not overly useful for this 2014 CTSW Study as it is not detailed enough.*

Regehr, J.D. (2009). “Truck Loading on Highway Infrastructure in the Canadian Prairie Region,” presentation prepared for the Vehicle-Infrastructure Interaction Workshop, Winnipeg, MB. *This presentation is related by not detailed enough to be useful for this 2014 CTSW Study.*

Regehr, J.D. (2003). “Estimating Live Truck Loads for Roads and Bridges: A Sectoral Approach Applied to Grain Transport,” presentation prepared for the Institute of Transportation Engineers (Manitoba Section), Winnipeg, Manitoba. *This presentation is related by not detailed enough to be useful for this 2014 CTSW Study.*

Regehr, J.D. (2002). “Aspects of Agriculture-Related Trucking in Manitoba,” UMTIG. *This presentation is related by not overly useful for this 2014 CTSW Study.*

Regehr, J.D., Baumgartner, T., Nuñez, A., and Montufar, J. (2009). “Measuring and Estimating Recreational Traffic in Manitoba,” presentation prepared for the Recreational Traffic Monitoring Workshop, Lakewood, CO. *This presentation is related by not detailed enough to be useful for this 2014 CTSW Study.*

Regehr, J.D., Jablonski, B., Rempel, G., Baumgartner, T., Nuñez, A., Patmore, K., Moshiri, M., Hernandez, H., and Montufar, J. (2010). “Traffic Data Requirements for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures in Manitoba,” presentation prepared for the MEPDG User Group, Transportation Association of Canada Spring Technical Meetings, Ottawa, ON. *This paper seems useful from its title, perhaps for the traffic classification, but it is propriety (a consulting report) and therefore not publicly-available.*

Regehr, J.D. and Montufar, J. (2007). “Classification Algorithm for Characterizing Long Multiple Trailer Truck Movements,” Transportation Research Board Annual Meeting CD-ROM, Washington, D.C. *This presentation is related by not overly useful for this 2014 CTSW Study. It deals with development of an algorithm that provides the core dataset for modelling long-truck exposure in terms of the volume of trips, and their weight and cubic characteristics. It is embedded within a modelling approach in which exposure is an explanatory variable needed for predicting transportation system impacts related to long-truck operations. Table 2 may be useful in that it contains WIM data related to long trucks from highways between Winnepeg and Brandon, MB, and Figure 3 includes the load spectra.*

Regehr, J.D. and Montufar J. (2012). “Traffic Data and the State of the Practice in Canada,” presentation prepared for the North American Travel Monitoring Exposition and Conference, Dallas, Texas. *This presentation is related by not overly useful for this 2014 CTSW Study.*

Regehr, J.D., Montufar, J., and Clayton, A. (2009). “Lessons Learned about the Impacts of Size and Weight Regulations on the Articulated Truck Fleet in the Canadian Prairie Region,” Canadian Journal of Civil Engineering, vol. 36, no. 4, pp. 607-616. *This publication is not directly related to pavement issues. It deals more with the state-of-the-practice and policy issues, but includes some information on the shift in traffic percentages related to articulated trucks. This paper is not useful for this 2014 CTSW Study.*

Regehr, J.D., Montufar, J., and Clayton, A. (2009). “Options for Exposure-Based Charging for Long Multiple Trailer Truck Permits,” Transportation Research Record: Journal of the Transportation Research Board, no. 2097, pp. 35-42. *This presentation is related by not overly useful for this 2014 CTSW Study as it appears to deal more with compliance issues.*

Regehr, J.D., Radstrom, B., Arango, J., Isaacs, C., Han, K., Rempel, G., Montufar, J., and Clayton, A. (2006). “Traffic on Manitoba Highways 2005,” UMTIG, prepared for the Traffic Engineering Branch, Manitoba Transportation and Government Services. *This presentation is related by not overly useful for this 2014 CTSW Study.*

Reimer, M. and Regehr, J.D. (2012). “Clustering of Vehicle Classification Data to Support Regional Implementation of the Mechanistic-Empirical Pavement Design Guide,” presentation prepared for the North American Travel Monitoring Exposition and Conference, Dallas, Texas. *This presentation will soon be published as a TRB TRR Journal article. It is relevant to pavements, but specifically looks at vehicle class rather than weight.*

Transportation Research Board of the National Academies. All Motor Carrier Publications. http://www.trb.org/MotorCarriers/Publications1.aspx *The relevant projects shown in this publications list appear to have already been included in this desk scan.*

APPENDIX B - PROJECT PLAN/SCHEDULE

1.1 General Approach for Pavement Comparative Analysis:

This section provides an overview of the approach that will be followed in completing the pavement comparative analysis. A total of 40 representative pavement sections, 10 sections within each of the 4 primary climatic zones in the United States, will be selected for analysis in this area of the Project. The AASHTOWare *Pavement ME Design*® model will be used in this analysis and will be run for each of these 40 sections to determine a base case for the expected pavement life cycle under representative average traffic conditions (e.g., representative of the mix of vehicle types and operating weights that might be expected based on compilation and analysis of large quantities of Weigh-in-Motion (WIM) data. An initial analysis of climate variability within each climate zone will be performed to ensure that the sites selected represent typical weather effects for that zone. To the extent possible, Long Term Pavement Performance (LTPP) program sections will be used as a basis for each sample section and will adjust base case parameters as required to make sure that each sample section represents the pavement performance history that would typically be expected.

While compiling the data required for each of the 40 selected sections, the first step will be to perform a complete analysis of a single pavement section to illustrate, evaluate, and, if necessary, adjust the analysis method that will be used for all the sections. For this pilot section, traffic inputs will be varied in ways that represent traffic shifts likely to occur as a result of the various truck scenarios, and will estimate the effects of a small sample of illustrative vehicle class and operating weight groups on the life of the pavement. This will require a series of runs of the *Pavement ME Design*® model during which all factors except traffic are held constant.

The multiple runs for each sample section will enable an evaluation of changes in pavement service life as a result of changes in truck travel associated with each modal shift scenario. These changes in pavement service life will be translated into pavement cost changes associated with size and weight scenarios.

1.2 Detailed Project Plan – Comparative Analysis of Truck Weight Impacts on Pavements

As outlined above, this analysis will consist of seven steps:

1) Select representative locations in each climate zone,
2) Select sample pavement sections,
3) Apply *Pavement ME Design*® to pilot sample section,
4) Apply *Pavement ME Design*® to base case traffic conditions,
5) Apply *Pavement ME Design*® to changes in travel by selected illustrative vehicles,
6) Apply *Pavement ME Design*® to scenario traffic variations, and
7) Expand sample results nationally.

The sections below describe each of these steps.

1.2.1 Select Representative Locations in Each Climate Zone

In this subtask, an analysis of sample sections of each pavement type in each of the four broad climate types—wet freeze, dry freeze, wet no-freeze, dry no-freeze—will be completed. These represent the traditional pavement climatic zones as well as the broad categories covered in the LTPP. *Pavement ME Design®* uses very detailed climatic data that varies considerably within each broad climatic region, so this exercise will help to assure that the weather station data shows reasonable values and candidate sections can be pared down to one location per climatic zone that represents the entire region. The study team will note the five parameters predicted in the Climate Summary, along with elevation, for each of the locations per climatic zone. The team will perform base runs at five different locations within each zone and will select sites representative of the five locations that to provide results that best represent the overall climate zone. Locations that make use of at least two, and ideally three or more, weather stations will be chosen to minimize issues of missing and spurious data that are sometimes observed for individual weather stations.

In this preliminary analysis, the traffic will be held constant using the default data in the *Pavement ME Design®* model and will be limited to one representative flexible pavement section and one representative rigid pavement section. Cross-sectional thicknesses that are designed to develop noticeable levels of distress will be used for the purposes of this preliminary analysis. Noticeable levels of distress are defined as the level that would trigger some type of rehabilitation action. The level of distresses initially selected will be the threshold values included in the MEPDG Manual of Practice. Other climate constants will be based on the LTPP SPS-8 original experimental plan (SHRP 1992a) sections and LTPP GPS sites, as per the four climatic zones. The results of this first analysis will be used to select the actual sections (to be used in the full factorial analysis) that most closely follow the median of the characteristics for this broad climate zone. The median will be based on the freezing index (primary factor) and number of freeze/thaw cycles (secondary factor) for the dry-freeze and wet-freeze sections and the mean annual precipitation (primary factor) for the dry-no freeze and wet-no freeze sections.

1.2.2 Select Sample Pavement Sections

Four pavement types will be considered for selection—flexible: new asphalt concrete (AC) and AC overlay on AC; rigid: jointed plain concrete pavement (JPCP); and, composite: AC over JPCP. Together, these pavement types represent the overwhelming majority of pavements used on streets and highways in the U.S. The basic premise is that the analysis should isolate the impacts of traffic shifts and load configurations, while holding other parameters constant. In order to achieve this goal, the baseline pavement sections will be based on the following criteria: 1) use actual traffic characteristics on our highways currently, 2) use sections with modern-day designs and materials (as close to actual site sections as possible), and 3) use the subgrade properties on site (preferred). The pavement layer thicknesses and material types will represent the median values included in the LTPP database. There have been multiple studies that have prepared histograms of the different pavement structures and layer thicknesses. These results will be used to establish the initial structures.

For rehabilitation, AC over AC or AC over JPCP, the condition of the existing pavement prior to overlay placement can have a significant impact on the predicted distresses. Thus, the condition at rehabilitation will be the threshold condition established above as the design criteria. These are provided in the AASHTO *MEPDG Manual of Practice*. This assumption will ensure that the new design and rehabilitation design will represent consistent values triggering some type of rehabilitation.

Within each climatic zone, truck traffic levels can vary by several orders of magnitude, with corresponding effects on pavement design and performance. Different truck travel values will be selected that correspond to three different truck traffic levels for flexible pavements and two different truck traffic levels for rigid pavements. For example, in identifying the two different traffic levels for rigid pavements those levels will be defined as above-average rural interstate highway traffic levels and below-average rural principal arterial traffic levels, respectively, in each climatic zone, making use of truck travel levels reported on the most recent available HPMS sample data file. Existing sections of interstate or other State primary routes that are instrumented for monitoring traffic, temperature profiles through the layer, and pavement responses will be considered within the climatic zones first (e.g., through the HPMS). Examples of these include active LTPP sections as well as State-led sections like the Weigh-in-Motion Pavement Investigation (WIMPI) in Maine (flexible pavement), various Superpave Instrumented Stress-Strain Investigation (SISSI) flexible pavement sites in Pennsylvania, and the MnRoad sections near Minneapolis, as well as the LTPP GPS and SPS-8 sections. LTPP has defined about 23 sites that are considered their “gold” standard in terms of accurate normalized axle load distributions for the standard truck classes. These WIM sites were identified and established within the FHWA/LTPP pool fund study. These sites will be reviewed as potential sites for extracting and using the axle weight data as the baseline condition for different roadway classifications. The normalized axle load distribution from each site has already been established under an LTPP-sponsored project.

Because the base modulus may have a significant effect on the relative magnitudes of damage caused by heavy trucks in rigid pavements, an analysis of two different base types for the JPCP will be used: granular and asphalt-stabilized. Consideration of asphalt and aggregate base type for flexible pavement is directly considered by varying the thickness of the asphalt surface layer for varying truck volumes. A conventional asphalt concrete pavement structure, as defined by the MEPDG Manual of Practice will be used. This family of flexible pavements includes an unbound aggregate layer beneath the asphalt surface layer. The thickness of the crushed aggregate base layer will be determined from the median values included in the LTPP database for this family of pavements.

The subgrade material property inputs can be generated for any location in the US using any of four different sources: 1) the actual subgrade properties on site (preferred); 2) the Level 3 soil properties from the AASHTO *MEPDG Manual of Practice*; 3) the LTPP GPS or SPS database (this database was used to establish the level 3 resilient modulus inputs), or 4) the NCHRP Project 9-23B Arizona State University (ASU) Soil Unit Map Application®. The ASU software tool displays an online GIS-enabled national soil database and features a query tool to identify the soil characteristics for inputs that are required by the *Pavement ME Design*® software. Any differences between the soil properties available in the Soil Unit Map Application and their LTPP database counterparts will be noted, where applicable.

The full factorial is shown in Figure 1 for all pavement types, climate zones and traffic levels.

- New flexible pavement total = 4 climate * 3 traffic = 12 cells.
- Asphalt overlay of flexible pavement total = 4 climate * 3 traffic = 12 cells.
- New rigid pavement total = 4 climate * 2 traffic = 8 cells.
- Asphalt overlay of rigid pavement total = 4 climate * 2 traffic = 8 cells.

This represents a total of 40 cells for a full factorial. These cells are representative of a large proportion of the US highway network (or National Highway System). Sections will be selected sections with typical representative design and materials characteristics.

1.2.3 Apply AASHTOWare Pavement ME Design® Model to Pilot Sample Section

In this phase of the project, a preliminary sample section will be identified as described in the next section and simulate traffic variations that might be expected to result from each scenario. Model runs will be used to estimate the changes in pavement life associated with changes in travel by selected illustrative vehicles. Likely illustrative vehicles will include, for example, (1) an 80,000-pound five-axle tractor-semitrailer combination vehicle with a tandem drive axle and a tandem trailer axle, (2) a 97,000-pound six-axle tractor-semitrailer combination vehicle with a tandem drive axle and a tridem trailer axle, and (3) an 88,000-pound five-axle tractor-semitrailer-full-trailer combination vehicle with single drive and trailer axles. By the time this stage is reached in the analysis, other scenario vehicles will be specified and included in the analysis as illustrative vehicles, also.

Since *Pavement ME Design®* includes only the 10 HPMS truck classes in its traffic inputs, the study team will subdivide several of those classes to allow a range of scenario vehicles as well as the additional detail needed for traffic shift analyses. Travel shifts will be computed and added to the detailed traffic composition by the vehicle classes, then recombined to the 10 classes needed for model input.

Applying *Pavement ME Design®* to the pilot section will allow evaluation and refinement of the specific model application procedures the study team will use for the rest of the sample pavement sections, as described below.

1.2.4 Apply AASHTOWare Pavement ME Design® Model to Base Case Sections

In this phase of the project, each actual selected section will be replicated as closely as possible using the input variables available in *Pavement ME Design®*, adjusting as necessary to match the observed distresses. If it is not possible to reasonably match observed distresses for a particular section, selection of an alternative section may be required.

As a first step in applying *Pavement ME Design®*, detailed traffic levels need to be set for each sample section. The rough traffic parameters known from HPMS section data (total ADT, combination truck ADT, and single-unit truck ADT) will be used for the particular selected section, as well as an appropriate set of axle weight and vehicle class distribution factors derived from a combination of current WIM data and updated FHWA VMT estimates. WIM data provided by the States to FHWA will be used, as well as the full set of WIM data collected under LTPP.

While compiling the WIM data in each State and for each highway type, the distances between combination vehicle load axles will be compiled so that modification of the *Pavement ME Design*® default values will be performed to better match truck characteristics for each sample section.

The full pavement analysis plan is going to answer the question of "when does the traffic shift being analyzed cause the pavement damage for a particular distress to exceed its targeted threshold value?" Exceeding the distress threshold presumably triggers the need for pavement repairs.

The analysis criteria (thresholds for each distress type), reliability level, and design period will be selected by following recommendations published in the AASHTO *MEPDG Manual of Practice* for each pavement type and roadway functional classification. The baseline pavement section will be held at no traffic growth (0 percent rate). Based on the findings of the preliminary climate analysis, the same multi-weather-station location will be used within each climate zone. Figure 15 shows a schematic that represents the distribution of sample sections. Within each of the cells shown, all factors will be held constant except traffic to evaluate the effect of each size and weight scenario, as described in the next section.

Figure 15: Schematic Matrix for Sample Pavement Section Selection

	Climate*			
Pavement type	Wet-freeze	Wet- No freeze	Dry-freeze	Dry- No freeze
New Flexible	High Truck ADT Med Truck ADT Low Truck ADT	High Truck ADT Med Truck ADT Low Truck ADT	High Truck ADT Med Truck ADT Low Truck ADT	High Truck ADT Med Truck ADT Low Truck ADT
AC Overlay over Flex.	High Truck ADT Med Truck ADT Low Truck ADT	High Truck ADT Med Truck ADT Low Truck ADT	High Truck ADT Med Truck ADT Low Truck ADT	High Truck ADT Med Truck ADT Low Truck ADT
New Rigid	High Truck ADT Low Truck ADT	High Truck ADT Low Truck ADT	High Truck ADT Low Truck ADT	High Truck ADT Low Truck ADT
AC Overlay over Rigid	High Truck ADT Low Truck ADT	High Truck ADT Low Truck ADT	High Truck ADT Low Truck ADT	High Truck ADT Low Truck ADT

* From one representative location for each, based on preliminary analysis

1.2.5 Apply Pavement ME Design® to Changes in Travel by Selected Illustrative Vehicles

Appropriate illustrative vehicles, the "alternative configurations" to be assessed in the study, will be identified, including two or three "base case" vehicles in common current use, as described in Section 1.2.3, and five to eight "scenario" vehicles.

For each illustrative vehicle, base case traffic mix will be added with sufficient travel by the vehicle of interest to result in an identifiable increment of loss of pavement life. For example, a loss of life of 3 months may be selected. The model will be run with a few selected increments of added travel by the vehicle of interest, and the level of travel will be adjusted until the target loss of life for each section is identified. This will enable a comparison of the relative effects of base and target vehicles, but will enable the estimation of the life cycle costs associated with each illustrative vehicle. The FHWA spreadsheet entitled RealCost will be used to calculate the life cycle cost for each scenario and example or truck traffic modal shift.

1.2.6 Apply Pavement ME Design® to Scenario Traffic Variations

Each study size and weight scenario will estimate the degree of travel shift among modes, vehicle configurations, and operating weights. The analysis of these anticipated shifts is crucial and is perhaps the most complex component of this pavement analysis, since the direction and magnitude of traffic shifts for each highway type in a State is a function of current regulations on and off the Interstate system, as well as details of each scenario. The study team will closely coordinate this effort with the work being performed under *Volume II: Modal Shift Comparative Analysis*, the first document in this volume of the 2014 CTSW study, to make sure the estimates generated through the modal shift analytical work—which are likely to include differential traffic impacts for groups of States with similar current size and weight regulations—are readily translated into detailed traffic inputs needed for *Pavement ME Design®* model runs.

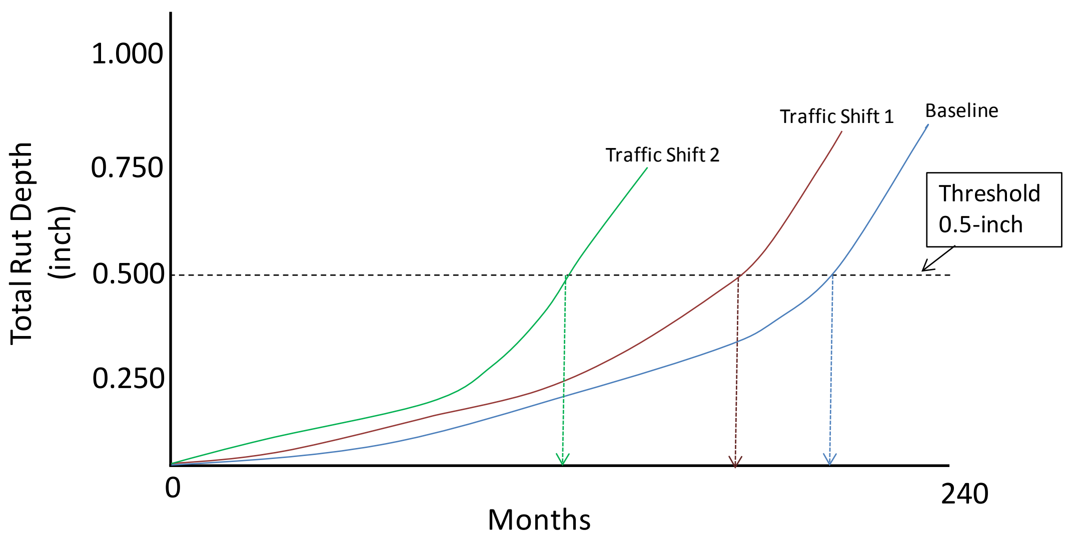
The effects of traffic shift per pavement type will be defined by the time to reach critical performance criteria. Note that these performance criteria are those typically used in design and pavement management by State highway agencies. In the context of this analysis, failure will be defined as number of months at which the key target pavement distresses are exceeded. In the case of new flexible pavements, the key distresses considered are bottom-up fatigue cracking, total rutting, and pavement ride quality (IRI). In the case of new rigid pavements, the key distresses considered are percent of slabs transverse cracked, amount of joint faulting, and pavement ride quality (IRI). The threshold values selected will be those included in the *MEPDG Manual of Practice* because they represent typical values used by agencies across the United States. In the case of flexible overlays, total cracking (bottom-up fatigue plus reflective cracking) will also be analyzed for its time to failure. Conducting the impact analysis in this manner allows for the calculation of the difference in pavement life (prior to pavement repair), as based solely on the traffic variables. The schematic in **Figure 16** illustrates a sample of the traffic shifting matrix for a new flexible pavement and **Figure 17** presents a sample of the new flexible pavement analysis of the impacts of traffic shifts on rutting.

It should be noted that the local calibration coefficients documented as examples under NCHRP Project 1-40B and included in the appendices of the *MEPDG Local Calibration Guide* will be used for predicting distress. Some agencies have also completed local calibration studies for both flexible and rigid pavements; however, these results will not be used within this study. The reason for considering the use of the results from NCHRP Project 1-40B is that the examples included some of the SPS experiments and test sections included in the LTPP program. This will then be consistent with the input level 3 and other parameters recommended in the *MEPDG Manual of Practice*.

Figure 16: Sample Traffic Shifting Analysis Matrix for a New Flexible Pavement (*fictional data for Demonstration Purposes only*).

	Months to Failure of Distress Type		
Traffic Set	Rutting	Bottom-Up Cracking	IRI
Baseline	192	172	189
Shift 1	190.8	180	180
Shift 2	191	164	177
. . .	. . .	. . .	. . .
Shift "n"	165	145	157

Figure 17: Sample Traffic Shifting Impacts on Total Rutting in a New Flexible Pavement (*fictional data for Demonstration Purposes only*).



Changes in pavement life will be translated into life cycle cost estimates using FHWA's RealCost software. The study team will focus on highway agency costs, meaning that the detailed temporal variation of traffic, capacity analysis, or value-of-time parameters needed for complete analysis of user costs will not be included. Instead, user costs, when they are present, will be noted whenever pavement rehabilitation is needed, as will changes in intervals of rehabilitation that will result in changes in user costs. Simplifying assumptions used in determining the user costs between different scenarios will be documented.

1.2.7 Expand Sample Results Nationally

Scenario traffic conditions will be selected for the pavement section of each type that most closely matches the characteristics of a given State and functional class, weighting the pavement types based on number of lane miles. In some cases, more than one environmental zone for a State will be applied, and the prevalence of each zone for that State will be assigned a weight, again based on lane miles. Similarly, interpolation may need to be performed on the differences between the two traffic levels in cases where highway classes have traffic that is not close to one of the sample traffic levels.

Estimates for every highway system, not just the Interstate System and the National Highway System, will be developed since these other road systems will also see traffic shifts, and traffic shifts in rural and urban areas will need to be considered separately.

1.3 Data Requirements for Pavement Comparative Analysis

The proposed approach to meet the requirements of the pavement analysis task requires a variety of data inputs, some of which are precisely the same data required by other tasks, and some of which are either unique to this task or requiring more detail than the other tasks.

1.3.1 Pavement Design and Materials Data

AASHTOWare Pavement ME Design® requires a large number of pavement design details, soil data, and other materials data. The software package includes the climate data needed for proper program operation, and includes a large quantity of nationally derived default data for nearly everything else. To properly analyze the sample pavement sections, however, pavement materials and design parameters need to be carefully matched to typical in-use pavement sections in each climate zone and at each traffic level. Steps will be taken to ensure proper and reasonable inputs are utilized and the LTPP database will serve as a reference data set.

1.3.2 Vehicle Classification Data

Vehicle classification data will be used, as appropriate, for initial estimates of truck travel for broad classes of trucks in each State on functional class. If appropriate, HPMS area wide travel counts reported by the States for the 13 HPMS vehicle classes on each highway system will be used. If these reports are not considered to be sufficiently reliable, the State-reported data will be ignored, adjusted, or aggregated as required. This has been done in previous cost allocation and size and weight studies. As noted previously, it is recommended that the LTPP “gold” WIM sites be used where appropriate to establish not only vehicle classification data, but more importantly the normalized axle load spectra for each truck class. These WIM sites were identified from the pool fund study. Using these sites adequately ties the normalized vehicle classification distribution to the normalized axle load distribution in terms of establishing a baseline condition or trend.

1.3.3 Weigh in Motion (WIM) Data

All available WIM data compiled by FHWA will be used for multiple purposes in this 2014 CTSW study, as well as the most recent years of WIM data collected for LTPP, as noted above. In previous compilations of national travel estimates and truck travel characteristics, a database has been constructed using the most recent consecutive 12 months of WIM data for each State. A battery of computer programs has been assembled to compile and analyze this data, which has been used in previous such compilations. The computer programs will be revised and updated as necessary and will provide compiled WIM data in whatever formats are required by other tasks in this study.

Detailed Vehicle Class Travel Estimates. Since raw WIM data reported to FHWA includes axle weights and distances between axles for each observed vehicle, the vehicle classifications provided by the standard axle-spacing algorithms used by the States can be estimated. This data can then be subdivided into the 13 HPMS vehicle classes and subdivided again into the more detailed classes required by the CTSW study. In general, the WIM data will be used to allocate control totals for broader vehicle class travel estimates provided by FHWA’s traffic monitoring system. If estimates of travel by the full 13 classes are used, the WIM data will be used to adjust State estimates for some or all of the truck classes based on previous observations of systematic misclassification of some vehicles. Class 13, for example, often includes two closely following vehicles whose axle spacings look like a double-trailer combination, but whose axle weights reveal that this is not the case.

In previous FHWA studies, individual WIM observations have been evaluated for validity based on the reported axle weights and spacings, and either reclassified or rejected according to explicit editing criteria. The editing criteria will be updated, refined, and adjusted to fit the needs of this study, as appropriate.

Operating Gross Weight (OGW) Distributions for Each Vehicle Class. Following the refinement of the WIM record editing criteria, operating weight distributions will be complied for each detailed truck class in each State and on each available highway class. Ideally, each State would report enough WIM data to FHWA to allow independent operating weight distributions for each vehicle class on each type of highway. In most cases, however, States collect WIM data on Interstate and arterial highways, especially rural arterial highways. Also, many States have found it difficult to collect and process traffic data accurately using the 13 vehicle class categories and so have adopted simplified schemes for classifying truck traffic. Also, some configurations can only be identified through inspection of special permit files, and some configurations cannot be identified at all. Therefore, highway types and sometimes States will be grouped to develop valid OGW distributions for many vehicle classes. In developing the estimates of OGW distributions, care will be taken to distinguish among States with varying weight regulation on Interstate and non-Interstate highways.

Axle Weight and Type Distributions. Axle weights and types have large effects on pavement deterioration and service life. WIM data provides an excellent source of information about the actual distribution of axle weights for the weight groups in each vehicle class, so that the use of unrealistic “idealized” axle weights to typify a weight class can be avoided. For example, an 80,000-pound 3-S2 is often characterized as having a 12,000-pound steering axle and two 34,000-pound tandem load axles. If the actual distribution of axle weights is 10,000 / 37,000 / 33,000 pounds, however, the vehicle will cause significantly more pavement damage than would be estimated by the standard weight distribution.

For consistency with *Pavement ME Design*® traffic input requirements, axle weight frequencies will be tabulated in 1,000-pound weight groups for steering axles and single load axles, 2,000-

pound increments for tandem axles, and 3,000-pound increments for tridem axles. Separate frequency distributions will be developed for each weight group and each vehicle class.

1.3.4 HPMS Section Data

The latest year of HPMS section data that is available will be used and along with all available traffic estimates, single-unit truck traffic estimates, combination truck traffic estimates, and pavement condition, design, and age data that are available on this data set. This data will be used in the selection of the pavement sections, to provide a check on large-category truck travel estimates, and to expand the results of the sample pavement sections to the national highway system.

1.4 Contingency Plan for Scenario Analysis

After performing a set of varied-traffic *Pavement ME Design®* runs for a single pavement section as described in Section 1.2.3, an analysis of the workability of the scenario traffic variation and illustrative vehicle schemes will be conducted. If it is determined that the scheme is unworkable, modifications to the work plan will be made. Rationales for finding the vehicle schemes unworkable will be documented.

1.5 Proposed Schedule for Completion

The work described in this Plan will be completed according to the following schedule:

Desk Scan

• Draft	August 28, 2013
• Final	November 8, 2013

Comparative Analysis of Truck Weight Impacts on Pavements

• Complete pilot section analysis	Dec. 4, 2013
• Complete base runs for all sections	Jan. 10, 2014
• Complete scenario runs	March 21, 2014
• Draft pavement impact report	March 28, 2014
• Final pavement impact report	April 22, 2014
• Final Technical Report	May 4, 2014

APPENDIX C – CLIMATE ANALYSIS SUMMARY DATA

	Geographic Location	State	County or City	Climate Location used in Pavement ME Design	Route ID	Elevation (ft)	Mean Air Temp.	Mean Annual Precipitation (inch)	Freezing Index (days)	Number of Wet Days	Average Annual Number of Freeze/Thaw Cycles	Pavement Structure Type	Data Source
1	#1	Montana	Deer Lodge		273	3828	45.6	10.9	2073.9	157.7	112.8	AC over granular base	SPS-8 table 2
2		South Dakota			1804	1720	48.1	15.7	2193	159	97.5	AC over granular base	SPS-8 table 2
3		Washington			91150	1168	53.8	17.4	302	162.8	38	AC over granular base	SPS-8 table 2
4		Washington	Walla Walla			1168	53.8	17.4	302	162.8	38	JPCP over unbound base	SPS-8 table 2
5		Colorado	Adams	Denver		5431	50.4	13.5	984	9.5	71.4	JPCP over unbound base	SPS-8 table 2
6		Idaho	Caldwell		I-84	2814	53	10.6	602.9	128.3	74.8		http://www.ncenet.com/ltpp/SPS_GPS_Maps-NEW.html
7		Wyoming	Cheyenne		I-25	6115	47	12.8	1725.2	159.1	117.9		http://www.ncenet.com/ltpp/SPS_GPS_Maps-NEW.html
8		New Mexico	Grant	Albuquerque	1014	5310	57.8	9.1	454	108.3	75.9	AC with granular base	SPS-8 table 2
9		Utah	Wasatch	Salt Lake City	SH-35	4220	53.2	14.6	909	148.2	74.7	AC with granular base	SPS-8 table 2
10		New Mexico	Santa Fe		I-25	6282	51.9	10.3	1244	125.7	144.1		http://www.ltppsrco.com/gps.php
11		Texas	Gray	Amarillo	I-40	3592	57.4	19.6	692.8	103.9	81.5		http://www.ltppsrco.com/gps.php
12		Utah	Richfield	Provo	I-70	4497	51.4	2.2	1210.9	15.8	96.9		http://www.ncenet.com/ltpp/SPS_GPS_Maps-NEW.html
					Mean	3845	52	13	1058	120	85		
13	#2	New Jersey			T/W 0	156	55	46.2	486.6	150.3	31.9	AC over granular base	SPS-8 table 2
14		New York	Orleans	Rochester	947 A	538	48.7	35.2	1360.3	225.1	56.3	AC over granular base	SPS-8 table 2
15		Ohio	Delaware	Columbus	23	812	53.7	40.8	889.2	193.9	50.1	JPCP over unbound base & AC over granular base	SPS-8 table 2
16		Wisconsin			29	885	46	30	2423.5	185.8	73.8	AC over granular base	SPS-8 table 2
17		Missouri	Christian	Springfield	65WOR	1262	56.4	40.8	792.7	177	61.6	JPCP over unbound base & AC over granular base	SPS-8 table 2
18		Maine	Bangor		I-95	148	44.7	34.4	2460.7	196	84		http://ltpp.stantec.com/naro/sites.htm
19		Virginia	Prince George		I-95 NB	163	58.4	44.8	407.6	158.3	53.8		http://ltpp.stantec.com/naro/sites.htm
20		Pennsylvania		Harrisburg (14751)	1-81 EB	336	53.3	42.6	874.8	185	61.3		http://ltpp.stantec.com/naro/sites.htm
21		Minnesota	Albany		1-94 WB	1420	43.3	21.9	2881.4	176.8	59.6		ltpp.stantec.com/ncro/sites.htm
22		Missouri	Harrison	Kansas City	US 61	743	57	41.7	816.5	154	54.3	JPCP over unbound base & AC over granular base	SPS-8 table 2
					Mean	646	52	38	1339	180	59		
23	#3	California	Merced	Merced		333	64	11.1	16.7	88	6	JPCP over unbound base & AC over granular base	SPS-8 table 2
24		Arizona	Phoenix	Phoenix	I-10	1485	73.2	8.7	0.7	62	0.3		http://www.ncenet.com/ltpp/SPS_GPS_Maps-NEW.html
25		California	Needles		I-40	890	75.2	4.1	25.5	44	1		http://www.ncenet.com/ltpp/SPS_GPS_Maps-NEW.html
26		Arizona	Tucson	Tucson	I-19	2549	69.5	10.2	20.7	83.4	8.9		http://www.ncenet.com/ltpp/SPS_GPS_Maps-NEW.html
27		California	Tracy		I-5	26	60.7	14.9	40.3	102.1	12.6		http://www.ncenet.com/ltpp/SPS_GPS_Maps-NEW.html
28		California	San Diego	San Diego	I-15	520	60.8	10.5	6.8	90.5	0.1		http://www.ncenet.com/ltpp/SPS_GPS_Maps-NEW.html
29		California	Barstow	Los Angeles (23174)	I-40	112	62.2	14.7	0	68.6	0		http://www.ltppsrco.com/gps.php
					Mean	845	67	11	16	77	4		
30	#4	North Carolina	Onslow	Wilmington	1245	24	63.5	53.6	146.1	156.6	26	AC over granular base	SPS-8 table 2
31		Texas	Bell		2620	655	68.9	36.3	33.5	128.3	7.1	JPCP over unbound base	SPS-8 table 2
32		Texas	Brazos	College Station	2223	306	68.3	41.6	48.9	144.4	12.4	AC over granular base	SPS-8 table 2
33		Florida	Hillsborough	St. Petersburg	I-75	5	73.5	49.7	41.9	146.8	0.6		http://www.ltppsrco.com/gps.php
34		Mississippi	Warren	Jackson	I-20	337	64.7	50.5	120.3	149.8	22.8		http://www.ltppsrco.com/gps.php
35		Oregon	Creswell	Eugene	I-5	355	52.5	37	108.3	195.9	25.6		http://www.ncenet.com/ltpp/SPS_GPS_Maps-NEW.html
36		Arkansas	Jefferson		65	253	62.4	42.8	259.8	159.5	36.8	AC over granular base & JCPC over unbound base	SPS-8 table 2
37		Mississippi	Panola	Jackson	315	305	63	51.8	240	154.2	30.2	AC over granular base	SPS-8 table 2
38		South Carolina	Richland		I-77	225	63.7	40.9	206.6	144.9	38.5		http://www.ltppsrco.com/gps.php
39		Georgia	Franklin		I-85	798	60.2	44.6	389.4	165.5	55.4		http://www.ltppsrco.com/gps.php
					Mean	326	64	45	159	155	26		

APPENDIX D – IDENTIFICATION OF COMMON SOIL TYPES FOR ANALYSIS

Geogr. Location	City	State	Latitude	Longitude	Soil Types	Map Char. Code
#1	Christian	Missouri	36.98	-93.27	A-6, A-6, A-7-6	S72
	Harrison	Missouri	40.41	-94.02	A-6, A-7-6, A-7-6	S33
	Orleans	New York	44.19	-75.93	A-4, A-4, A-2-4	CV3
	Columbus	Ohio	39.98	-83.01	A-4, A-7-6, A-6	N34
	Harrisburg	Pennsylvania	40.27	-76.87	A-4, A-4, A-4	NA1
	Albany	New York	42.20	-73.45	A-4, A-2-4, A-2-4, A-1-a	CW8
	Baltimore	Maryland	39.18	-76.38	A-4, A-4. A-2-4	MY8
	Boston	Massachusetts	42.21	-71.50	A-2-5, A-2-5, A-2-5	HA2
	Philadelphia	Pennsylvania	39.57	-75.10	A-4, A-6, A-4	MY7
	Montpelier	Vermont	44.15	-72.32	A-4, A-4, A-4	WM4
	Providence	Rhode Island	41.50	-71.24	A-4, A-2-4, A-1-b, A-1-a	PJ8
	Washington	DC	38.53	-77.02	A-4, A-4, A-4	R42
	Bridgeport	Connecticut	41.11	-73.11	A-4	508
	Dover	Delaware	39.80	-75.28	A-4, A-4, A-2-4	526
	Pittsburgh	Pennsylvania	40.30	-80.13	A-6, A-6, A-7-6, A-7-6	MW8
	Bangor	Maine	44.48	-68.50	A-4, A-4, A-4	S24
	Concord	New Hampshire	43.12	-71.30	A-4, A-4, A-1-b	AS9
	Chicago	Illinois	41.50	-87.37	A-7-6, A-7-6, A-6	N28
	Detroit	Michigan	42.20	-83.30	A-2-5, A-2-5, A-2-5	HC6
	Des Moines	Iowa	41.35	-93.37	A-7-6, A-7-6, A-4, A-6	ER5
	Duluth	Minnesota	46.49	-92.50	A-7-6, A-7-5, A-7-5	XZ2
	Fargo	North Dakota	46.52	-96.48	A-7-6, A-7-6, A-7-6	AC8
	Madison	Wisconsin	43.80	-89.20	A-4, A-6, A-2-5	YK1
	Hot Springs	Arkansas	34.31	-93.30	A-6, A-7-6, A-7-6, A-7-6	242
	Little Rock	Arkansas	34.44	-92.14	A-6, A-7-6, A-7-6, A-7-6	242
	Montgomery	Alabama	32.23	-86.22	A-2-4, A-2-4, A-4, A-6	083
	Wichita	Kansas	37.43	-97.17	A-6, A-7-6, A-7-6	P59
	Topeka	Kansas	39.40	-95.38	A-4, A-4	P89
	Minneapolis	Minnesota	44.97	-93.26	A-4, A-6, A-6	JL0
	Madison	Wisconsin	43.80	-89.20	A-4, A-6, A-2-5	YK1
	Indianapolis	Indiana	39.46	-86.10	A-4, A-6, A-6, A-6	N57
	St. Louis	Missouri	38.35	-90.12	A-4, A-6, A-6	S63
	Knoxville	Tennessee	35.57	-83.56	A-4, A-7-6, A-2-6	WB3
	Louisville	Kentucky	38.15	-85.46	A-4, A-7-6, A-7-6	GR0
	Milwaukee	Wisconsin	43.20	-87.55	A-4, A-7-6, A-7-6, A-6	N02
	Oklahoma City	Oklahoma	35.26	-97.28	A-4, A-7-6, A-7-6	NJ9
	Tulsa	Oklahoma	36.12	-95.54	A-4, A-7-6, A-7-6	NR9
	Valparaiso	Indiana	41.31	-87.20	A-4, A-7-6, A-6	N34
	Waterloo	Iowa	42.33	-92.24	A-6, A-4	ET1
	Lexington	Kentucky	38.20	-84.36	A-4, A-7-6, A-7-5	GR4
	Richmond	Virginia	37.30	-77.20	A-4, A-6, A-2-4	VN9
	Lincoln	Nebraska	40.51	-96.45	A-6, A-4	PB5
Geogr. Location	**City**	**State**	**Latitude**	**Longitude**	**Soil Types**	**Map Char. Code**
#2	St. Petersburg	Florida	27.78	-82.66	A-3, A-2-5, A-2-5	634
	Jackson	Mississippi	32.29	-90.18	A-4, A-6, A-6, A-6	KF7
	Onslow	North Carolina	34.80	-77.54	A-2-5, A-4, A-2-5	Z78
	Creswell	Oregon	43.91	-123.01	A-6, A-7-6, A-7-6	DW6
	Brazos	Texas	30.67	-96.28	A-4, A-6	RC0
	Atlanta	Georgia	33.45	-84.23	A-4, A-4, A-7-5	756
	Charleston	South Carolina	32.47	-79.56	A-4, A-6, A-6	PW6
	Miami	Florida	25.77	-80.24	A-3, A-4	649
	Panama City	Florida	30.40	-85.35	A-3, A-3	557
	Raleigh	North Carolina	35.52	-78.47	A-2-4, A-4, A-6	Z88
	Jackson	Mississippi	32.20	-90.12	A-4, A-6, A-6, A-6	KF6
	Seattle	Washington	47.37	-122.20	A-1-b, A-1-b	XJ9
	Houston	Texas	29.45	-95.21	A-4, A-6, A-6	RF7
	Baton Rouge	Louisiana	30.32	-91.90	A-7-6, A-6, A-6	FA0
	Panola County	Mississippi	34.37	-89.96	A-4, A-4, A-4	KB7
#3	Adams	Colorado	39.90	-104.49	A-2-4	A49
	Grant	New Mexico	35.15	-107.84	A-2-4, A-7-6, A-6	BK7
	Gray	Texas	35.40	-100.82	A-6, A-6	RA1
	Richfield	Utah	38.76	-112.09	A-4, A-4, A-4	VK2
	Wasatch	Utah	40.37	-111.15	A-4, A-6, A-2-6, A-2-4, A-6	TX1
	Albuquerque	New Mexico	35.05	-106.39	A-2-4, A-2-4, A-1-a	BR8
	Denver	Colorado	39.45	-105.00	A-4, A-6, A-4	897
	Cheyenne	Wyoming	41.90	-104.52	A-2-4, A-4	1C8
	Helena	Montana	46.35	-112.20	A-2-4, A-2-4, A-1-b, A-1-b	X25
	Pierre	South Dakota	44.22	-100.21	A-7-5, A-7-5, A-7-5	QE9
	Santa Fe	New Mexico	35.37	-106.50	A-6, A-6	BH9
	Billings	Montana	45.48	-108.32	A-4, A-4, A-4	W78
	Rock Springs	Wyoming	41.36	-109.00	A-4, A-4, A-4	ZF6
	Boise	Idaho	43.34	-116.13	A-4, A-4, A-1-a	K60
	Amarillo	Texas	35.11	-101.50	A-6, A-7-6, A-7-6, A-6	SP2
#4	Phoenix	Arizona	33.63	-112.09	A-4, A-4	279
	Tucson	Arizona	32.18	-110.88	A-1-a, A-1-a	310
	Barstow	California	34.89	-117.02	A-2-4, A-6, A-2-4, A-1-b	A46
	Merced	California	37.32	-120.48	A-6, A-7-6, A-4	E48
	San Diego	California	32.85	-117.12	A-4, A-4, A-7-6, A-7-6	G92
	San Jose	California	37.48	-122.16	A-4	G65
	Las Vegas	Nevada	36.10	-115.12	A-2-4	LM1
	Needles	California	34.83	-114.59	A-1-a, A-2-4, A-1-b	B79
	Flagstaff	Arizona	35.18	-111.64	A-6, A-4, A-4, A-4	486
	Fresno	California	36.74	-119.74	A-4, A-4, A-2-4	E40

#1	Tally Counts per soil	Soil	Most often observed Soil Type
	2	A-1-a	
	2	A-1-b	
	9	A-2-4	
	8	A-2-5	
	1	A-2-6	
	1st 45	A-4	A-4
	0	A-5	
	3rd 27	A-6	
	3	A-7-5	
	2nd 33	A-7-6	

#2	Tally Counts per soil	Soil	Most often observed Soil Type
	0	A-1-a	
	2	A-1-b	
	1	A-2-4	
	4	A-2-5	
	3rd 4	A-3	
	2nd 13	A-4	
	0	A-5	
	1st 15	A-6	A-6
	1	A-7-5	
	3	A-7-6	

#3	Tally Counts per soil	Soil	Most often observed Soil Type
	2	A-1-a	
	2	A-1-b	
	3rd 8	A-2-4	
	0	A-2-5	
	1	A-2-6	
	1st 15	A-4	A-4
	0	A-5	
	2nd 10	A-6	
	3	A-7-5	
	3	A-7-6	

#4	Tally Counts per soil	Soil	Most often observed Soil Type
	3	A-1-a	
	2	A-1-b	
	2nd 5	A-2-4	
	0	A-2-5	
	1st 11	A-4	A-4
	0	A-5	
	3rd 3	A-6	
	0	A-7-5	
	3	A-7-6	

APPENDIX E – CLIMATE ANALYSIS INPUT DATA SUMMARY

	Location Site		ASU Soil Map Tool (from NCHRP Project 9-23 B)			
	Geographic Location	**SHRP ID LTPP GPS/SPS #**	**MapChar Code**	**Soil Type (inch): Top layer, … , semi-infinite layer**	**Resilient modulus (psi) used for Subgrade Layer**	Default Depth to GW Table (ft) from Pavement ME Design
1	Christian County, Missouri	29-0801, 29-0802, 29-0807, 29-0808	S72	A-6 (2), A-6 (3.1), A-7-6 (30.7)	11,655	5.34
2	Harrison County, Missouri	29-A801, 29-A802, A807, A808	S33	A-6 (9.1), A-7-6 (20.1), A-7-6 (42.9)	7,210	5.34
3	Orleans County, New York	36-0801, 36-0802, 36-0859	CV3	A-4 (7.9), A-4 (18.1), A-2-4 (46.1)	17,696	5.34
4	Columbus, Ohio	39-0809, 39-0810, 39-A803, 39-A804	N34	A-4 (9.1), A-7-6 (13.8), A-6(37)	10,316	5.34
5	Harrisburg, Pennsylvania	42-1598	NA1	A-4 (9.8), A-4 (35), A-4 (23.2)	21,402	5.34
6	St. Petersburg, Florida	12-4057	634	A-3 (7.1), A-2-5 (55.9), A-2-5 (16.9)	16,135	5.34
7	Panola County, Mississippi	28-0805, 28-0806	KB7	A-4 (7.9), A-4 (18.1), A-4 (39)	16,729	5.34
8	Onslow County, North Carolina	37-0801, 37-0802, 37-0859	Z78	A-2-5 (35.8), A-4 (13), A-2-5 (29.1)	19,092	5.34
9	Creswell County, Oregon	41-5022	DW6	A-6 (11), A-7-6 (7.1), A-7-6 (11.8)	7,020	5.34
10	Brazos County, Texas	48-0801, 48-0802	RC0	A-4 (7.9), A-6 (22)	14,188	5.34
11	Adams County, Colorado	8-0811, 8-0812	A49	A-2-4 (9.8)	29,542	5.34
12	Grant County, New Mexico	35-0801, 35-0802	BK7	A-2-4 (3.1), A-7-6 (13), A-6 (2.8)	14,800	5.34
13	Gray County, Texas	48-5335	RA1	A-6 (9.8), A-6 (50)	12,916	5.34
14	Richfield County, Utah	49-7083	VK2	A-4 (5.9), A-4 (34.3), A-4 (19.7)	13,548	5.34
15	Wasatch County, Utah	49-0803, 49-0804	TX1	A-4 (9.8), A-6 (4.3), A-2-6 (13.8), A-2-4 (16.9), A-6 (15.7)	14,813	5.34
16	Phoenix, Arizona	04-1007	279	A-4 (5.9), A-4 (53.9)	12,872	5.34
17	Tucson, Arizona	04-6054	310	A-1-a (22.8), A-1-a (35.8)	30,733	5.34
18	Barstow, California	6-0811, 6-0812, 6-A805, 6-A806	A46	A-2-4 (5.1), A-6 (15.7), A-2-4 (18.1), A-1-b (20.9)	22,928	5.34
19	Merced County, California	06-0500	E48	A-6 (11.8), A-7-6 (8.3), A-4 (11.8)	9,167	5.34
20	San Diego, California	06-3010	G92	A-4 (15), A-4 (3.9), A-7-6 (3.1), A-7-6 (2)	12,609	5.34

APPENDIX F – PRELIMINARY FLEXIBLE AND RIGID PAVEMENT PERFORMANCE ANALYSIS

Flexible Pavement Performance Analyses **Heavy vehicle level: 1,500 trucks per day and 6.5" thick HMA**

Geogr. Location (City or County)	SHRP ID LTPP GPS/SPS #	Geogr Loc	Total Rutting (inch)	Year rutting threshold was exceeded	Thermal crackg (ft/mi)	Year TC threshold was exceeded	% lane area bottom-up	Year FC threshold was exceeded	IRI (in/mile)	Year IRI threshold was exceeded
Christian, Missouri	0801, 0802, 0807, 0808	#1	1.15	8/30 yr	27.17	not exceeded	23.75	not exceeded	219.30	15/30 yr
Harrison, Missouri	A801, A802, A807, A808	#1	1.15	8/30 yr	27.17	not exceeded	24.88	not exceeded	220.20	15/30 yr
Orleans, New York	0801, 0802, 0859	#1	0.78	27/30 yr	26.19	not exceeded	19.53	not exceeded	202.70	19/30 yr
Columbus, Ohio	0809, 0810, A803, A804	#1	1.01	12/30 yr	27.17	not exceeded	23.31	not exceeded	211.50	17/30 yr
Harrisburg, Pennsylvania	421598	#1	0.93	17/30 yr	27.17	not exceeded	22.29	not exceeded	209.20	18/30 yr
St. Petersburg, Florida	124057	#2	0.82	24/30yr	27.17	not exceeded	6.76	not exceeded	171.60	25/30yr
Panola, Mississippi	0805, 0806	#2	0.83	23/30yr	27.17	not exceeded	10.61	not exceeded	195.20	20/30yr
Onslow, North Carolina	0801, 0802, 0859	#2	0.89	20/30 yr	27.17	not exceeded	20.47	not exceeded	198.30	20/30 yr
Creswell, Oregon	415022	#2	0.96	13/30 yr	27.17	not exceeded	20.86	not exceeded	203.20	18/30 yr
Brazos, Texas	0801, 0802	#2	0.89	18/30yr	27.17	not exceeded	10.62	not exceeded	193.20	20/30yr
Adams, Colorado	0811, 0812	#3	0.68	not exceeded	27.17	not exceeded	6.75	not exceeded	185.90	27/30 yr
Grant, New Mexico	0801, 0802	#3	0.98	14/30 yr	27.17	not exceeded	19.90	not exceeded	190.90	20/30 yr
Gray, Texas	485335	#3	0.97	15/30 yr	27.17	not exceeded	21.58	not exceeded	206.30	18/30 yr
Richfield, Utah	0803, 0804	#3	1.04	13/30 yr	27.17	not exceeded	19.97	not exceeded	196.20	20/30 yr
Wasatch, Utah	497083	#3	0.95	15/30 yr	27.17	not exceeded	19.06	not exceeded	201.80	19/30 yr
Phoenix, Arizona	041007	#4	1.37	7/30 yr	27.17	not exceeded	22.45	not exceeded	199.00	17/30 yr
Tucson, Arizona	46054	#4	1.16	20/30yr	27.17	not exceeded	1.45	not exceeded	182.10	17/30yr
Barstow, California	0811, 0812, A805, A806	#4	0.65	not exceeded	27.17	not exceeded	6.75	not exceeded	164.50	28/30 yr
Merced, California	060500	#4	1.00	13/30 yr	27.17	not exceeded	21.09	not exceeded	194.00	20/30 yr
San Diego, California	063010	#4	0.85	21/30 yr	27.17	not exceeded	19.27	not exceeded	180.10	23/30 yr

Rigid Pavement Performance Analyses **Heavy vehicle level: 1500 trucks per day and 8" thick JPCP**

Geographic Location	SHRP ID LTPP GPS/SPS #	Geogr Loc	Faulting (inch)	Year faulting threshold was exceeded	% slab fatigue cracked	Year cracking threshold was exceeded	IRI (in/mile)	Year IRI threshold was exceeded
Christian, Missouri	0801, 0802, 0807, 0808	#1	0.10	not exceeded	26.57	11/30yr	189.07	23/30yr
Harrison, Missouri	A801, A802, A807, A808	#1	0.10	not exceeded	20.5	15/30yr	180.32	24/30yr
Orleans, New York	0801, 0802, 0859	#1	0.07	not exceeded	6.38	not exceeded	170.93	27/30yr
Columbus, Ohio	0809, 0810, A803, A804	#1	0.10	not exceeded	19.33	15/30yr	187.68	22/30yr
Harrisburg, Pennsylvania	421598	#1	0.09	not exceeded	19.33	15/30yr	177.61	25/30yr
St. Petersburg, Florida	124057	#2	0.05	not exceeded	17.69	16/30yr	118.65	not exceeded
Panola, Mississippi	0805, 0806	#2	0.07	not exceeded	26.85	12/30yr	144.02	not exceeded
Onslow, North Carolina	0801, 0802, 0859	#2	0.06	not exceeded	14.85	19/30yr	123.34	not exceeded
Creswell, Oregon	415022	#2	0.07	not exceeded	19.65	15/30yr	135.86	not exceeded
Brazos, Texas	0801, 0802	#2	0.06	not exceeded	21.53	14/30yr	130.59	not exceeded
Adams, Colorado	0811, 0812	#3	0.03	not exceeded	4.25	not exceeded	106.16	not exceeded
Grant, New Mexico	0801, 0802	#3	0.06	not exceeded	11.77	25/30yr	132.18	not exceeded
Gray, Texas	485335	#3	0.09	not exceeded	29.33	10/30yr	181.22	24/30yr
Richfield, Utah	0803, 0804	#3	0.06	not exceeded	27.63	11/30yr	187.29	23/30yr
Wasatch, Utah	497083	#3	0.08	not exceeded	7.67	not exceeded	163.50	28/30yr
Phoenix, Arizona	041007	#4	0.06	not exceeded	28.22	11/30yr	133.75	not exceeded
Tucson, Arizona	46054	#4	0.06	not exceeded	61.59	5/30yr	167.17	27/30yr
Barstow, California	0811, 0812, A805, A806	#4	0.03	not exceeded	5.55	not exceeded	99.01	not exceeded
Merced, California	060500	#4	0.06	not exceeded	36.87	10/30yr	145.62	not exceeded
San Diego, California	063010	#4	0.05	not exceeded	15.61	20/30yr	116.53	not exceeded

APPENDIX G – SUMMARY OF ADTT FOR VARIOUS INTERSTATE VOLUMES AND OTHER NHS ARTERIAL FROM FHWA VMT DATA

Average Daily Truck Travel by State and Major Highway Type, 2011

Selected: HV MV LV

		Annual Truck VMT (Millions)				Highway System Miles				Average Trucks per Day					
		Interstate		Other NHS		Interstate		Other NHS		Interstate			Other NHS		
State	**Geo Loc**	**Rural**	**Urban**	**Rural**	**Urban**	**Rural**	**Urban**	**Rural**	**Urban**	**Rural**	**Urban**	**Combined**	**Rural**	**Urban**	**Combined**
CT	#1	114.47	1144.28	112.21	289.29	43.3	302.9	422.5	920.4	7246	10350	9962	728	861	819
DC	#1	0.00	34.22	0.00	104.27	0.0	12.8	0.0	121.0	0	7344	7344	0	2362	2362
DE	#1	0.00	60.69	133.39	101.18	0.0	40.6	275.4	207.1	0	4094	4094	1327	1339	1332
IA	#1	1411.51	715.09	1358.62	193.08	628.4	153.7	7368.2	824.9	6154	12748	7450	505	641	519
IL	#1	2418.65	2323.05	1362.36	1342.51	1356.2	826.0	7024.5	3239.0	4886	7705	5953	531	1136	722
IN	#1	2591.60	2256.64	1287.22	1092.93	712.3	459.2	3659.2	2030.5	9968	13463	11338	964	1475	1146
KY	#1	2205.19	513.43	884.98	271.14	596.8	203.9	4300.1	914.0	10124	6898	9302	564	813	607
MA	#1	159.72	767.63	112.83	548.75	91.4	482.5	555.7	2171.1	4790	4359	4427	556	692	665
MD	#1	473.55	851.65	322.42	599.04	183.6	297.4	1285.3	1392.2	7068	7847	7549	687	1179	943
ME	#1	352.73	74.04	376.40	33.24	299.2	69.5	1804.5	161.4	3230	2920	3172	571	564	571
MI	#1	813.37	1698.74	1166.15	1532.16	608.9	635.2	7583.6	2633.4	3660	7327	5532	421	1594	724
MN	#1	984.59	675.54	1252.62	581.43	629.5	284.4	10200.3	827.9	4285	6508	4977	336	1924	456
MO	#1	2214.82	1692.67	1712.15	610.57	723.6	482.6	7119.9	1547.7	8386	9609	8875	659	1081	734
NH	#1	65.82	66.61	152.28	43.31	149.2	75.9	820.1	272.1	1208	2403	1611	509	436	491
NJ	#1	232.31	1338.34	152.28	1329.58	65.0	366.3	566.9	2107.7	9792	10010	9977	736	1728	1518
NY	#1	924.70	2320.77	866.70	2499.26	846.8	857.2	5285.0	3646.8	2992	7418	5218	449	1878	1032
OH	#1	2294.19	2475.37	1749.79	1438.42	723.3	850.3	4630.1	2909.6	8690	7976	8304	1035	1354	1159
PA	#1	4476.10	3197.52	1515.80	1555.91	1119.7	735.6	6449.4	3357.1	10952	11910	11332	644	1270	858
RI	#1	59.30	101.03	18.16	117.70	21.5	49.6	113.8	448.8	7566	5583	6182	437	719	662
VA	#1	1143.19	1241.79	788.80	488.98	656.6	462.6	4820.4	1619.2	4770	7354	5838	448	827	544
VT	#1	186.60	21.13	202.75	16.74	279.9	40.3	1047.8	120.6	1826	1435	1777	530	380	515
WI	#1	1122.46	628.21	1485.48	1340.76	477.8	264.7	7994.5	2227.6	6437	6501	6460	509	1649	757
WV	#1	368.97	427.54	852.92	201.42	368.0	186.6	2423.2	345.3	2747	6279	3935	964	1598	1043
All	#1	24613.86	24625.99	17866.32	16331.66	10580.9	8139.8	85750.1	34045.4	6373	8289	7206	571	1314	782
AL	#2	1446.30	1399.42	2001.77	1206.27	536.8	369.0	6217.8	1118.7	7382	10390	8607	882	2954	1198
AR	#2	1644.04	445.86	984.51	289.41	441.5	214.1	5194.4	775.5	10203	5706	8734	519	1023	585
FL	#2	1884.88	2667.31	1778.67	2792.17	748.4	746.8	5262.8	4181.2	6900	9785	8341	926	1830	1326
GA	#2	3192.46	3025.91	1170.17	2225.13	716.3	531.6	7880.7	2071.5	12210	15594	13652	407	2943	935
LA	#2	849.63	797.34	878.46	405.81	532.8	364.8	2778.7	1136.7	4369	5988	5027	866	978	899
MS	#2	528.60	381.37	931.52	330.76	501.7	200.5	5593.5	1065.5	2887	5210	3550	456	850	519
NC	#2	625.56	874.60	822.24	688.74	541.3	632.3	4522.2	2321.4	3166	3789	3502	498	813	605
SC	#2	835.59	562.36	771.43	957.69	580.5	270.1	4576.9	1137.2	3944	5704	4503	462	2307	829
TN	#2	2184.79	1165.32	1387.36	982.11	687.5	417.0	5083.1	1699.5	8707	7656	8310	748	1583	957
HI	#2	26.50	71.10	26.90	51.66	6.4	48.6	400.9	266.6	11432	4012	4871	184	531	322
All	#2	13218.35	11390.59	10753.06	9929.77	5293.1	3794.9	47511.0	15773.8	6842	8223	7419	620	1725	895
CO	#3	647.27	641.57	804.44	500.75	684.6	268.1	6072.5	1495.3	2590	6555	3706	363	917	473
NE	#3	1539.85	187.56	906.71	172.61	417.5	64.2	6865.7	498.0	10105	7999	9824	362	950	402
ND	#3	420.95	50.20	609.21	35.45	519.2	51.7	5450.2	166.3	2221	2658	2261	306	584	314
SD	#3	559.53	72.71	549.45	28.65	602.4	76.5	5873.9	152.9	2545	2603	2551	256	513	263
WY	#3	255.73	10.13	133.73	6.45	812.4	101.1	3244.8	223.0	862	274	797	113	79	111
AK	#3	142.90	59.06	56.39	79.46	1005.4	78.5	1243.4	123.0	389	2060	510	124	1771	272
WA	#3	751.48	974.98	811.19	1182.94	467.3	296.9	3891.6	1732.1	4406	8996	6189	571	1871	971
ID	#3	751.95	277.14	470.84	156.41	521.6	90.0	3134.8	455.9	3950	8432	4610	412	940	479
MT	#3	691.03	82.56	672.12	153.67	1129.4	62.9	5601.9	190.7	1676	3599	1778	329	2207	391
NV	#3	671.51	426.06	348.16	569.29	449.2	121.4	2282.8	409.4	4096	9618	5270	418	3809	934
OR	#3	1180.38	568.71	821.32	353.34	553.1	176.5	5158.5	818.4	5847	8828	6568	436	1183	538
UT	#3	1026.30	1347.08	770.34	472.85	723.9	212.9	2461.6	415.5	3884	17332	6940	857	3118	1184
KS	#3	884.63	267.10	913.56	122.91	656.0	217.9	7355.8	988.0	3694	3359	3611	340	341	340
OK	#3	890.12	441.16	761.72	268.86	683.5	249.1	5031.1	1247.9	3568	4851	3911	415	590	450
All	#3	10413.63	5406.02	8629.18	4103.64	9225.6	2067.9	63668.7	8916.5	3093	7162	3838	371	1261	481
AZ	#4	1951.60	340.91	808.19	618.37	980.5	187.6	2620.5	1613.2	5453	4978	5377	845	1050	923
NM	#4	1054.26	69.57	921.93	302.55	847.6	152.3	3807.8	693.8	3408	1251	3079	663	1195	745
TX	#4	4546.21	5317.05	6705.19	5042.29	2049.5	1187.7	17593.7	7344.5	6077	12265	8347	1044	1881	1291
CA	#4	3556.51	4642.84	3382.81	8595.47	1278.7	1174.0	10202.9	8089.0	7620	10835	9159	908	2911	1794
All	#4	11108.58	10370.37	11818.12	14558.69	5156.3	2701.7	34224.8	17740.5	5902	10516	7489	946	2248	1391
All States		59354.43	51792.97	49066.68	44923.76	30255.83	16704.29	231154.65	76476.12	5375	8495	6485	582	1609	837

APPENDIX H – PAVEMENT THICKNESS AND TYPE GROUPINGS FROM HPMS 2012 DATA

Flexible Pavement Thickness

WF

Geographic Location #1

	Interstate	Pct	Cum %	Arterial	Pct	Cum %
0.1 to 1.0	0.8	0.1%	0.1%	849.6	3.5%	3.5%
1.1 to 2.0	71.4	5.4%	5.4%	1148.5	4.8%	8.3%
2.1 to 3.0	101.4	7.6%	13.0%	1397.3	5.8%	14.1%
3.1 to 4.0	180	13.5%	26.6%	4718.3	19.6%	33.7%
4.1 to 5.0	95.8	7.2%	33.8%	2179.8	9.0%	42.7%
5.1 to 6.0	76.2	5.7%	39.5%	3322.4	13.8%	56.5%
6.1 to 7.0	31.6	2.4%	41.8%	2494.9	10.4%	66.9%
7.1 to 8.0	28.9	2.2%	44.0%	1712.9	7.1%	74.0%
8.1 to 9.0	197.2	14.8%	58.8%	912.2	3.8%	77.8%
9.1 to 10.0	31.3	2.4%	61.2%	957.1	4.0%	81.8%
10.1 to 11.0	62.4	4.7%	65.9%	600.1	2.5%	84.2%
11.1 to 12.0	47.8	3.6%	69.5%	3234.9	13.4%	97.7%
12.1 to 13.0	191.4	14.4%	83.8%	156.9	0.7%	98.3%
13.1 to 14.0	83.8	6.3%	90.1%	84.1	0.3%	98.7%
14.1 to 15.0	25.8	1.9%	92.1%	60.1	0.2%	98.9%
15.1 to 16.0	59.2	4.4%	96.5%	92.9	0.4%	99.3%
> 16	46.5	3.5%	100.0%	164.8	0.7%	100.0%
Missing	1727.9			25487.1		
	3,059.4			**49,573.9**		

JPCP Pavement Thickness

WF

Geographic Location #1

	Interstate	Pct	Cum %	Arterial	Pct	Cum %
0.1 to 1.0	0.7	0.0%	0.0%	0	0.0%	0.0%
1.1 to 2.0	0.0	0.0%	0.0%	3.1	0.0%	0.0%
2.1 to 3.0	0.0	0.0%	0.0%	0	0.0%	0.0%
3.1 to 4.0	0.0	0.0%	0.0%	93.6	1.4%	1.5%
4.1 to 5.0	5.9	0.4%	0.5%	1.9	0.0%	1.5%
5.1 to 6.0	6.4	0.5%	0.9%	169.9	2.6%	4.0%
6.1 to 7.0	2.3	0.2%	1.1%	262.4	3.9%	8.0%
7.1 to 8.0	39.5	2.8%	3.9%	884.1	13.3%	21.3%
8.1 to 9.0	212.9	15.0%	18.9%	1441.1	21.7%	43.0%
9.1 to 10.0	278.4	19.6%	38.5%	3010.1	45.3%	88.3%
10.1 to 11.0	349.4	24.6%	63.1%	537.3	8.1%	96.4%
11.1 to 12.0	312.7	22.0%	85.1%	99.7	1.5%	97.9%
12.1 to 13.0	127.9	9.0%	94.2%	51.9	0.8%	98.6%
13.1 to 14.0	46.9	3.3%	97.5%	5.5	0.1%	98.7%
14.1 to 15.0	25.7	1.8%	99.3%	0	0.0%	98.7%
15.1 to 16.0	0.0	0.0%	99.3%	65.9	1.0%	99.7%
> 16	10.3	0.7%	100.0%	19.3	0.3%	100.0%
Missing	459.8			1054.7		
	1,878.8			**7,700.5**		

Flexible Pavement Thickness WNF

Geographic Location #2

	Interstate	Pct	Cum %	Arterial	Pct	Cum %
0.1 to 1.0	7.8	0.6%	0.6%	1253	4.5%	4.5%
1.1 to 2.0	38.4	3.0%	3.7%	1724	6.3%	10.8%
2.1 to 3.0	132.8	10.5%	14.2%	3377.8	12.3%	23.1%
3.1 to 4.0	87.8	6.9%	21.1%	9107.5	33.1%	56.1%
4.1 to 5.0	190.1	15.0%	36.1%	2615	9.5%	65.6%
5.1 to 6.0	140.8	11.1%	47.3%	4776.5	17.3%	83.0%
6.1 to 7.0	102.6	8.1%	55.4%	940.6	3.4%	86.4%
7.1 to 8.0	172.9	13.7%	69.1%	1655.7	6.0%	92.4%
8.1 to 9.0	44.5	3.5%	72.6%	562.3	2.0%	94.4%
9.1 to 10.0	31.4	2.5%	75.1%	558.2	2.0%	96.4%
10.1 to 11.0	30.8	2.4%	77.5%	309.9	1.1%	97.6%
11.1 to 12.0	60.2	4.8%	82.3%	93.4	0.3%	97.9%
12.1 to 13.0	102.1	8.1%	90.3%	265.9	1.0%	98.9%
13.1 to 14.0	16.7	1.3%	91.6%	167.2	0.6%	99.5%
14.1 to 15.0	25.9	2.0%	93.7%	121.2	0.4%	99.9%
15.1 to 16.0	5.7	0.5%	94.1%	6.3	0.0%	99.9%
> 16	74.0	5.9%	100.0%	14.1	0.1%	100.0%
Missing	2494.0			27497.8		
	3,758.5			**55,046.4**		

JPCP Pavement Thickness WNF

Geographic Location #2

	Interstate	Pct	Cum %	Arterial	Pct	Cum %
0.1 to 1.0	0	0.0%	0.0%	5.1	0.3%	0.3%
1.1 to 2.0	0	0.0%	0.0%	11.5	0.8%	1.1%
2.1 to 3.0	0.4	0.1%	0.1%	290.7	19.1%	20.1%
3.1 to 4.0	0	0.0%	0.1%	757	49.6%	69.8%
4.1 to 5.0	0	0.0%	0.1%	42.1	2.8%	72.5%
5.1 to 6.0	0	0.0%	0.1%	71.3	4.7%	77.2%
6.1 to 7.0	0.6	0.1%	0.2%	66.4	4.4%	81.6%
7.1 to 8.0	11.2	2.0%	2.2%	109	7.1%	88.7%
8.1 to 9.0	235.5	41.8%	43.9%	66.1	4.3%	93.1%
9.1 to 10.0	185.7	32.9%	76.9%	99.1	6.5%	99.6%
10.1 to 11.0	26.5	4.7%	81.6%	3.2	0.2%	99.8%
11.1 to 12.0	65.4	11.6%	93.2%	0.1	0.0%	99.8%
12.1 to 13.0	1.9	0.3%	93.5%	2	0.1%	99.9%
13.1 to 14.0	7	1.2%	94.7%	0	0.0%	99.9%
14.1 to 15.0	1	0.2%	94.9%	0	0.0%	99.9%
15.1 to 16.0	0	0.0%	94.9%	0	0.0%	99.9%
> 16	28.6	5.1%	100.0%	1.5	0.1%	100.0%
Missing	1287.2			1345.5		
	1,851.0			**2,870.6**		

Flexible Pavement Thickness

DF

Geographic Location #3

	Interstate	Pct	Cum %	Arterial	Pct	Cum %
0.1 to 1.0	83.7	2.6%	2.6%	641.7	2.1%	2.1%
1.1 to 2.0	89.6	2.8%	5.5%	3298.8	10.9%	13.0%
2.1 to 3.0	255.9	8.1%	13.6%	2853.9	9.4%	22.4%
3.1 to 4.0	180.1	5.7%	19.2%	4700.1	15.5%	37.9%
4.1 to 5.0	348.7	11.0%	30.3%	4223.6	13.9%	51.8%
5.1 to 6.0	228	7.2%	37.5%	3766	12.4%	64.2%
6.1 to 7.0	1006.9	31.8%	69.3%	5427.1	17.9%	82.1%
7.1 to 8.0	381.6	12.1%	81.3%	1586.8	5.2%	87.4%
8.1 to 9.0	200.3	6.3%	87.7%	1265.2	4.2%	91.5%
9.1 to 10.0	108	3.4%	91.1%	652.9	2.2%	93.7%
10.1 to 11.0	94.2	3.0%	94.0%	420.3	1.4%	95.1%
11.1 to 12.0	77.4	2.4%	96.5%	464	1.5%	96.6%
12.1 to 13.0	57.8	1.8%	98.3%	234.8	0.8%	97.4%
13.1 to 14.0	3.4	0.1%	98.4%	282.5	0.9%	98.3%
14.1 to 15.0	8	0.3%	98.7%	175.7	0.6%	98.9%
15.1 to 16.0	0.9	0.0%	98.7%	15.3	0.1%	98.9%
> 16	40.9	1.3%	100.0%	320.1	1.1%	100.0%
Missing	1890.2			15990.9		
	5,055.6			**46,319.7**		

JPCP Pavement Thickness

DF

Geographic Location #3

	Interstate	Pct	Cum %	Arterial	Pct	Cum %
0.1 to 1.0	0.9	0.1%	0.1%	2.8	0.1%	0.1%
1.1 to 2.0	0	0.0%	0.1%	29	0.8%	0.8%
2.1 to 3.0	0	0.0%	0.1%	4.5	0.1%	0.9%
3.1 to 4.0	0	0.0%	0.1%	5.5	0.1%	1.1%
4.1 to 5.0	0	0.0%	0.1%	10.4	0.3%	1.4%
5.1 to 6.0	0.5	0.0%	0.1%	40.5	1.1%	2.4%
6.1 to 7.0	1	0.1%	0.2%	115.4	3.0%	5.4%
7.1 to 8.0	50.2	3.2%	3.3%	1555	40.5%	45.9%
8.1 to 9.0	275.5	17.5%	20.9%	943.9	24.6%	70.5%
9.1 to 10.0	476	30.3%	51.2%	486.5	12.7%	83.2%
10.1 to 11.0	267.5	17.0%	68.2%	29.4	0.8%	84.0%
11.1 to 12.0	273.4	17.4%	85.6%	21.1	0.5%	84.5%
12.1 to 13.0	64.6	4.1%	89.7%	168.7	4.4%	88.9%
13.1 to 14.0	21.6	1.4%	91.1%	327.4	8.5%	97.4%
14.1 to 15.0	0	0.0%	91.1%	34.3	0.9%	98.3%
15.1 to 16.0	47.5	3.0%	94.1%	6.4	0.2%	98.5%
> 16	92.5	5.9%	100.0%	58.1	1.5%	100.0%
Missing	382.1			1063		
	1,953.3			**4,901.9**		

Flexible Pavement Thickness DNF

Geographic Location #4

	Interstate	Pct	Cum %	Arterial	Pct	Cum %
0.1 to 1.0	0	0.0%	0.0%	2195.4	8.5%	8.5%
1.1 to 2.0	135.8	8.8%	8.8%	3907.7	15.1%	23.5%
2.1 to 3.0	209.3	13.6%	22.4%	7833.5	30.2%	53.8%
3.1 to 4.0	42.9	2.8%	25.2%	3295.2	12.7%	66.5%
4.1 to 5.0	14.2	0.9%	26.1%	1761.9	6.8%	73.3%
5.1 to 6.0	534.9	34.7%	60.7%	3881.4	15.0%	88.2%
6.1 to 7.0	52.7	3.4%	64.2%	460.7	1.8%	90.0%
7.1 to 8.0	213.2	13.8%	78.0%	1210.7	4.7%	94.7%
8.1 to 9.0	109.2	7.1%	85.1%	376.7	1.5%	96.1%
9.1 to 10.0	65.3	4.2%	89.3%	509.2	2.0%	98.1%
10.1 to 11.0	26.9	1.7%	91.0%	67.4	0.3%	98.4%
11.1 to 12.0	90.9	5.9%	96.9%	165.5	0.6%	99.0%
12.1 to 13.0	0	0.0%	96.9%	34.3	0.1%	99.1%
13.1 to 14.0	0	0.0%	96.9%	81.3	0.3%	99.5%
14.1 to 15.0	0	0.0%	96.9%	47.7	0.2%	99.6%
15.1 to 16.0	0.7	0.0%	97.0%	24.3	0.1%	99.7%
> 16	46.7	3.0%	100.0%	70.2	0.3%	100.0%
Missing	929.1			13466.5		
	2,471.8			**39,389.6**		

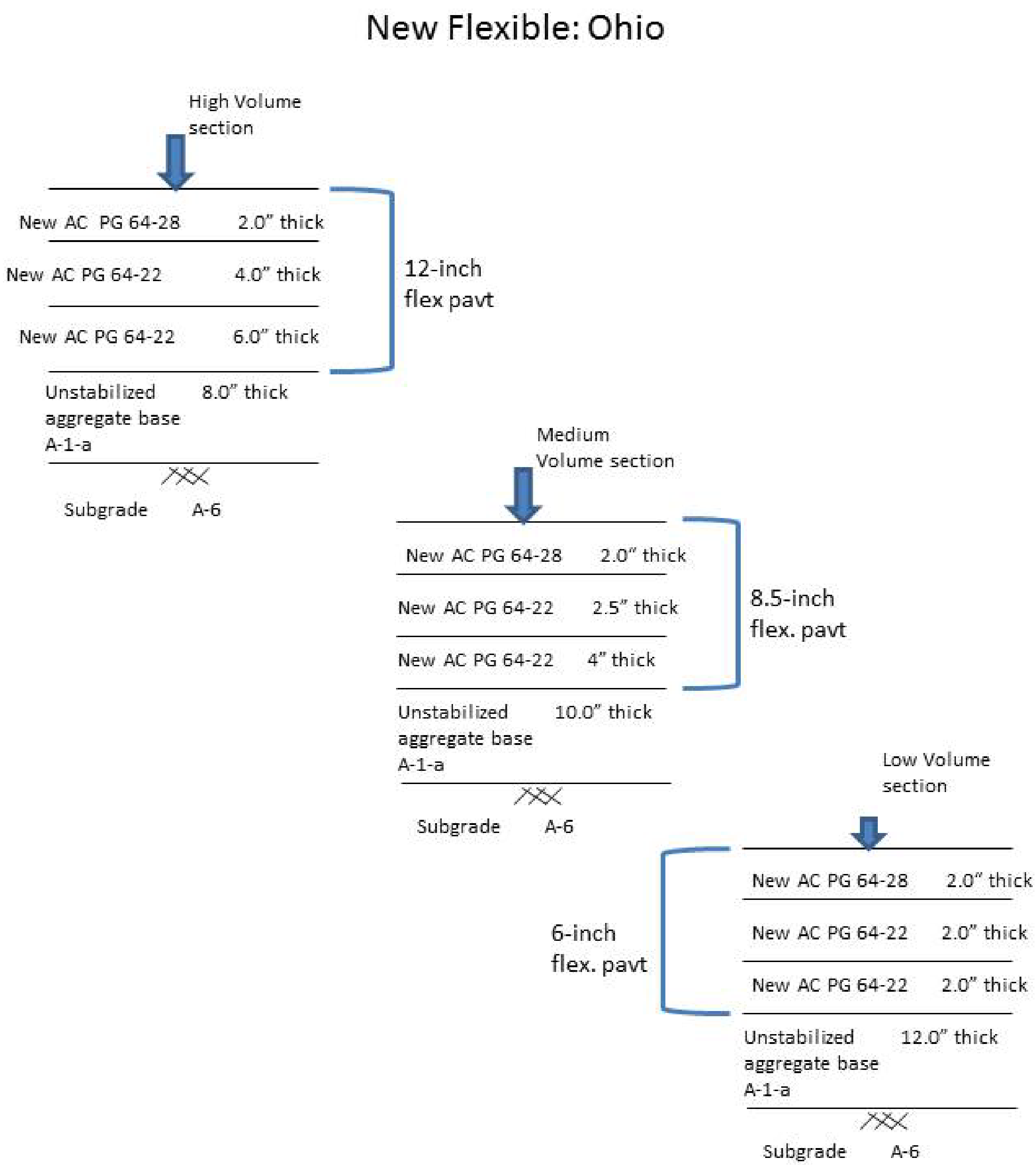
JPCP Pavement Thickness DNF

Geographic Location #4

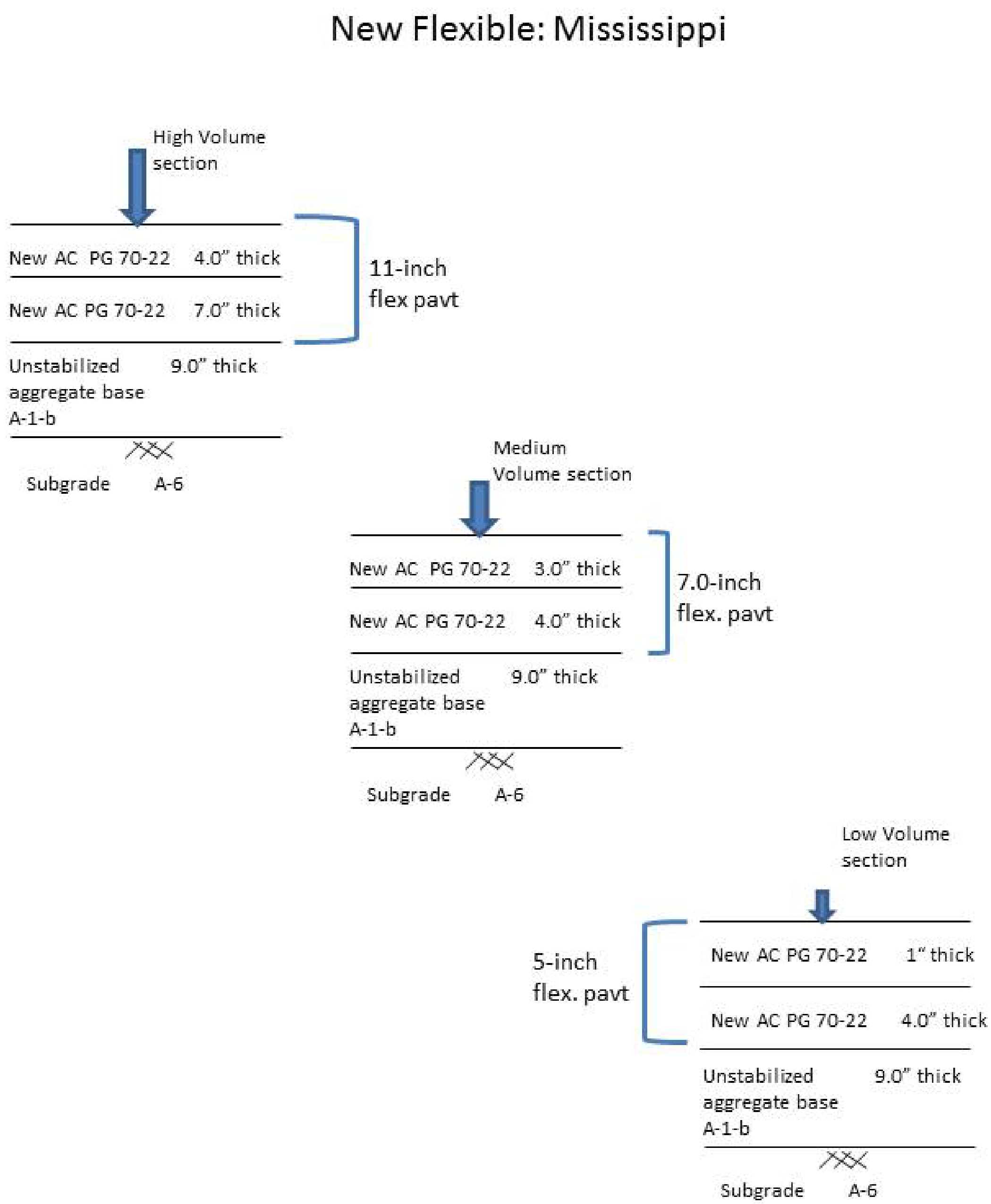
	Interstate	Pct	Cum %	Arterial	Pct	Cum %
0.1 to 1.0	0	0.0%	0.0%	3.4	0.2%	0.2%
1.1 to 2.0	0	0.0%	0.0%	6.1	0.4%	0.6%
2.1 to 3.0	0	0.0%	0.0%	0	0.0%	0.6%
3.1 to 4.0	0	0.0%	0.0%	0	0.0%	0.6%
4.1 to 5.0	0	0.0%	0.0%	0	0.0%	0.6%
5.1 to 6.0	0	0.0%	0.0%	65.9	4.4%	5.1%
6.1 to 7.0	0	0.0%	0.0%	11.6	0.8%	5.9%
7.1 to 8.0	5.8	58.0%	58.0%	793.6	53.5%	59.4%
8.1 to 9.0	1.4	14.0%	72.0%	67.1	4.5%	63.9%
9.1 to 10.0	0.9	9.0%	81.0%	477.6	32.2%	96.1%
10.1 to 11.0	0	0.0%	81.0%	20.5	1.4%	97.5%
11.1 to 12.0	0	0.0%	81.0%	11.6	0.8%	98.3%
12.1 to 13.0	0	0.0%	81.0%	1.5	0.1%	98.4%
13.1 to 14.0	1.1	11.0%	92.0%	0	0.0%	98.4%
14.1 to 15.0	0.8	8.0%	100.0%	24.4	1.6%	100.0%
15.1 to 16.0	0	0.0%	100.0%	0	0.0%	100.0%
> 16	0	0.0%	100.0%	0	0.0%	100.0%
Missing	739.1			1043.7		
	749.1			**2,527.0**		

APPENDIX I – SAMPLE FLEXIBLE PAVEMENT SECTIONS

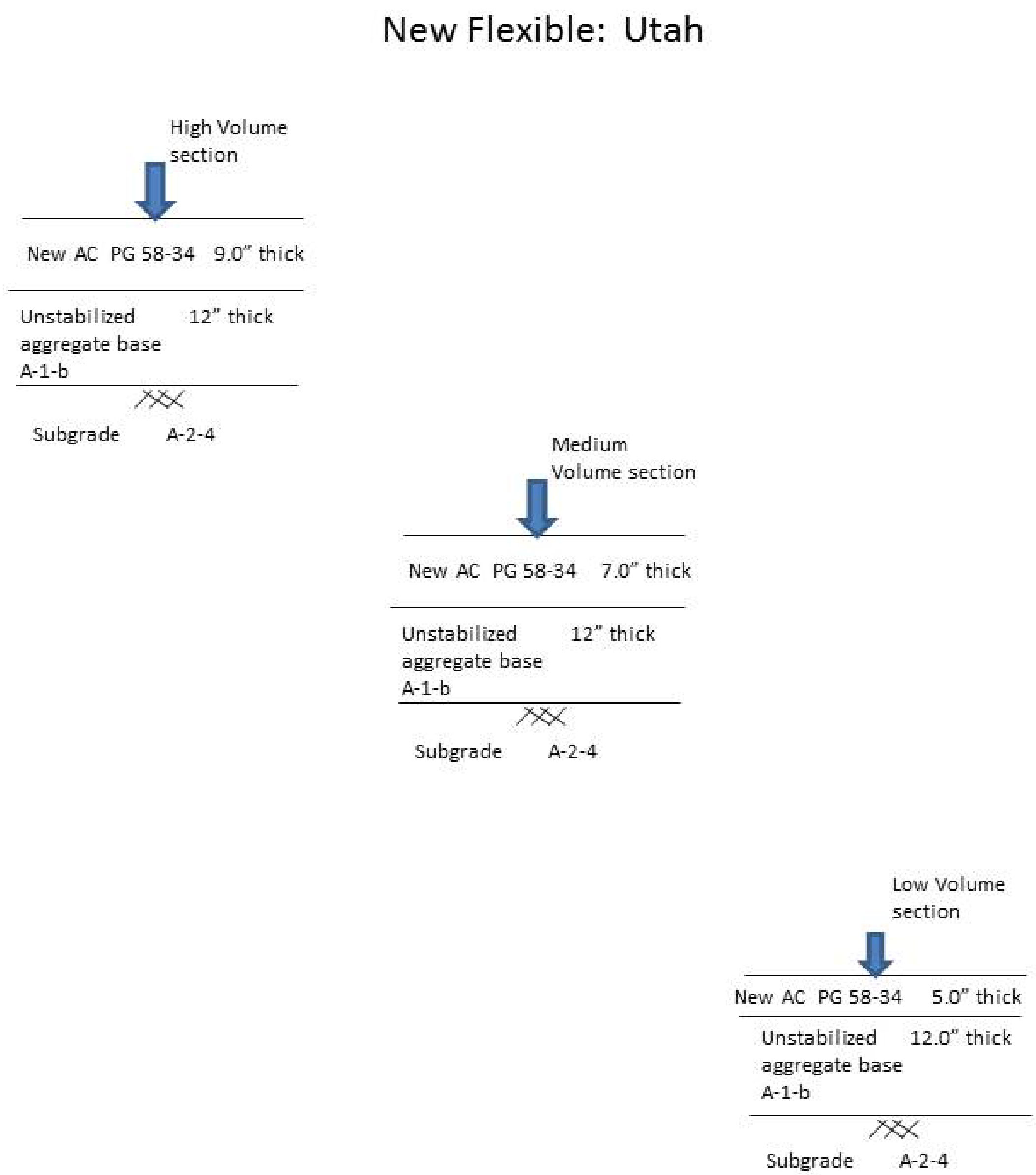
Geographic Location #1 – Flexible Pavement: OHIO



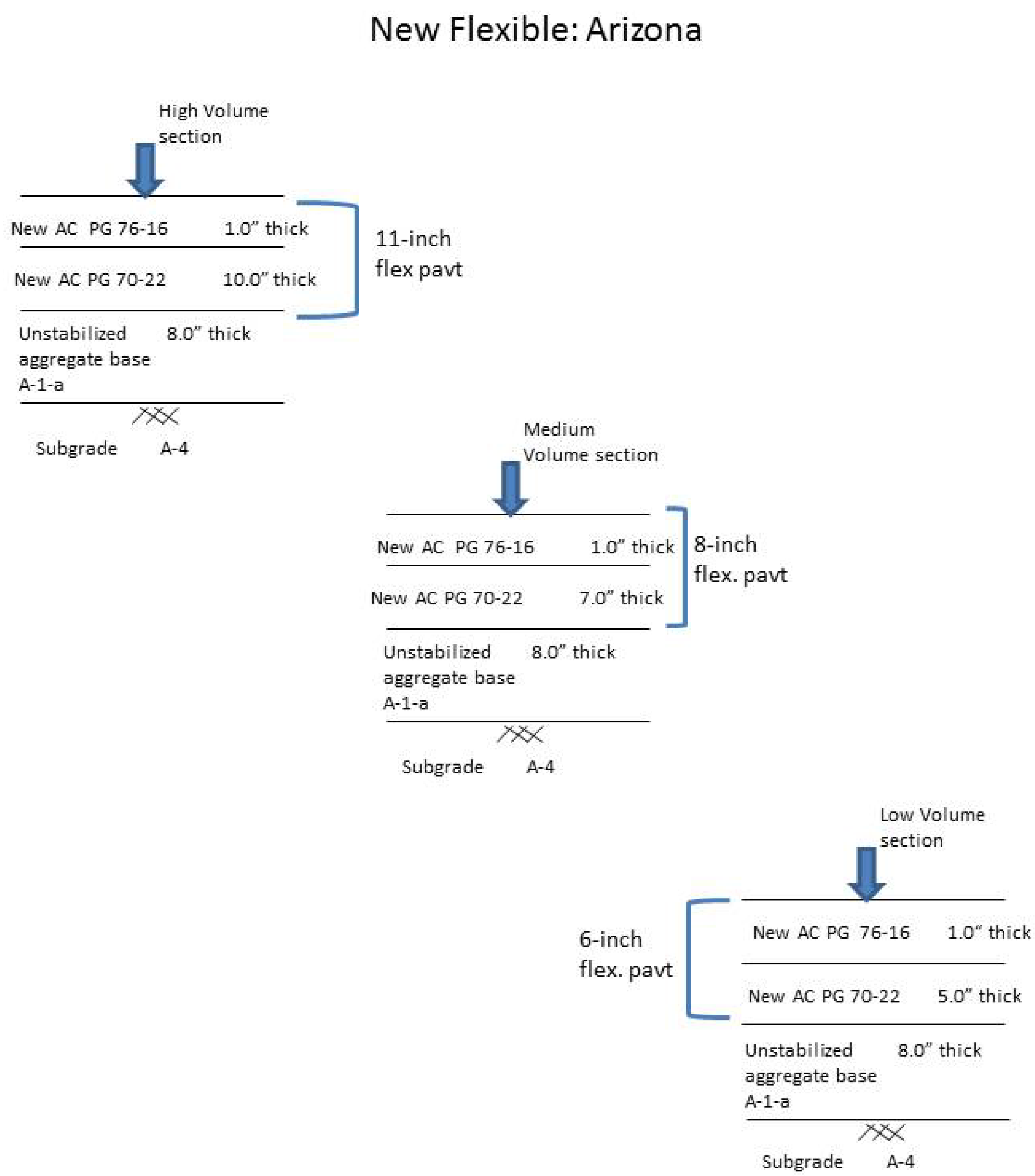
Geographic Location #2 – Flexible Pavement: MISSISSIPPI



Geographic Location #3 – Flexible Pavement: UTAH

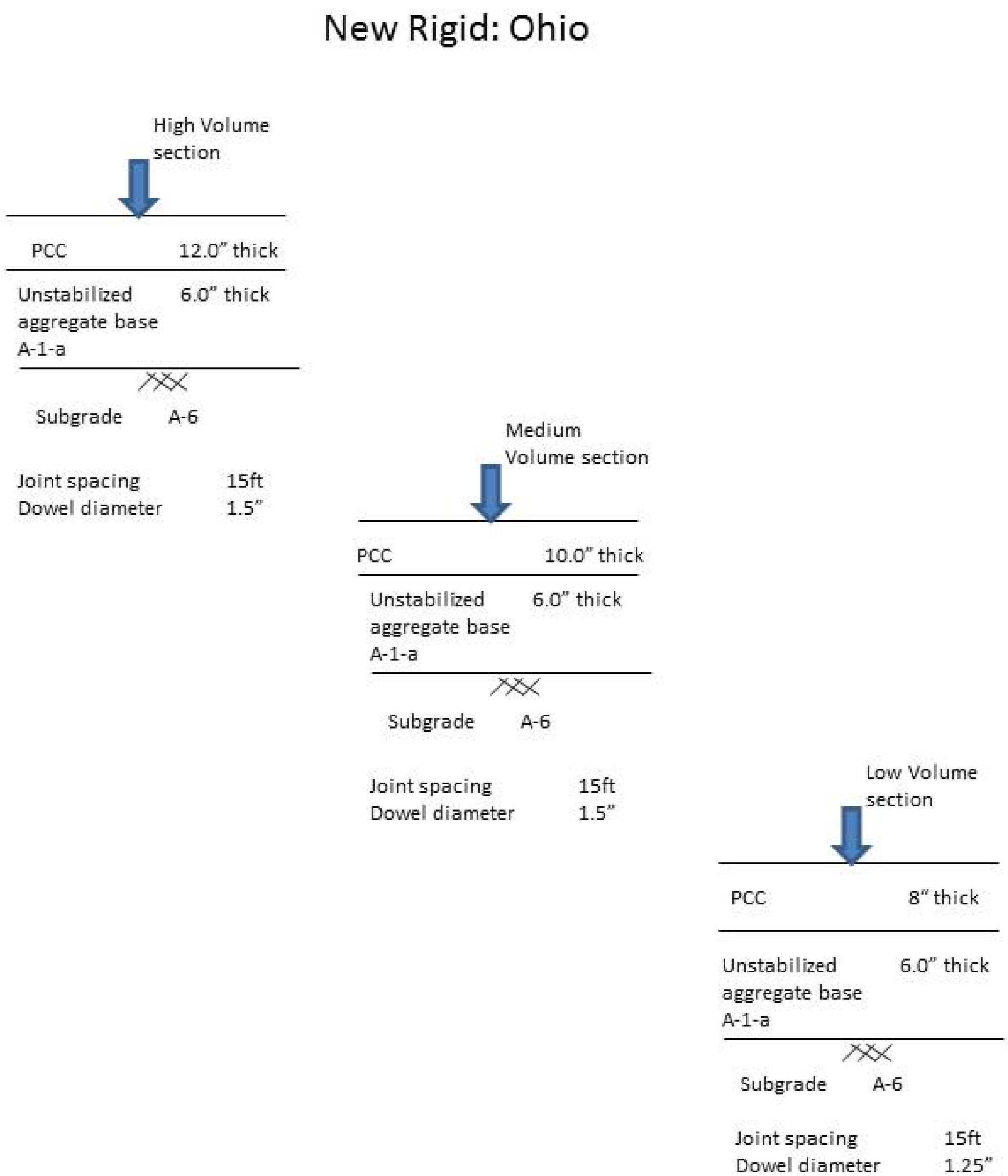


Geographic Location #4 – Flexible Pavement: ARIZONA

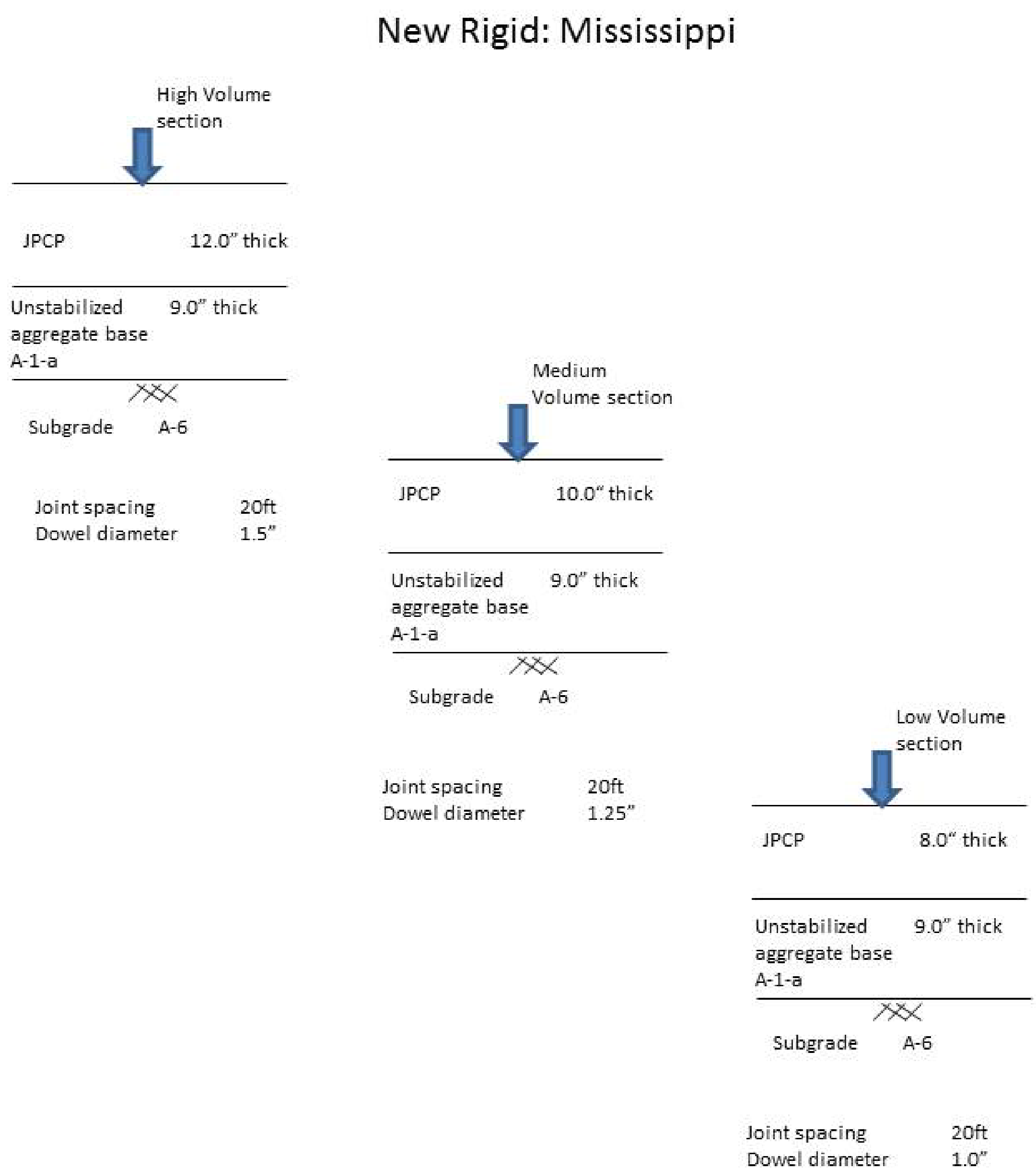


APPENDIX J – SAMPLE RIGID PAVEMENT SECTIONS

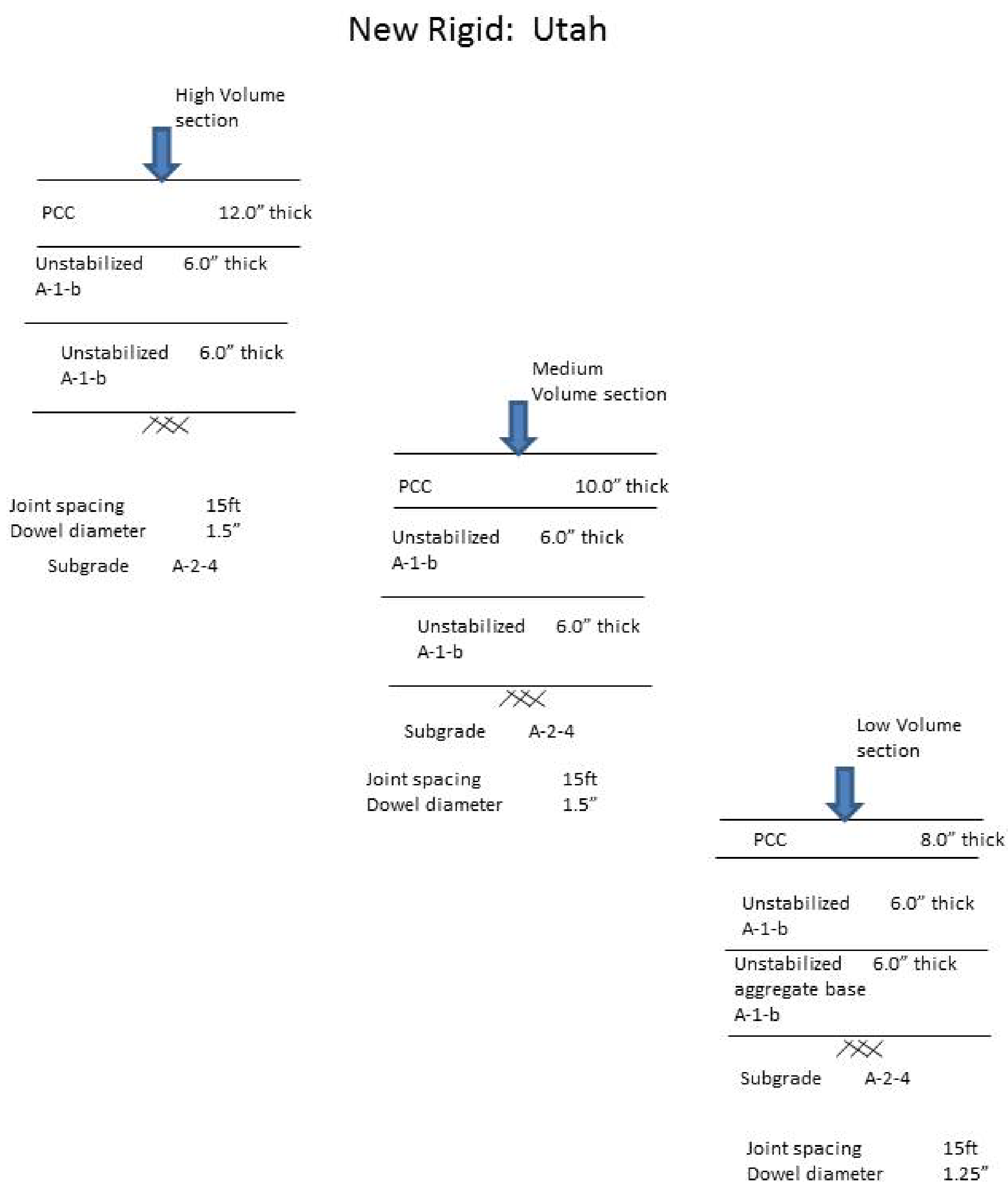
Geographic Location #1 – Rigid Pavement: OHIO



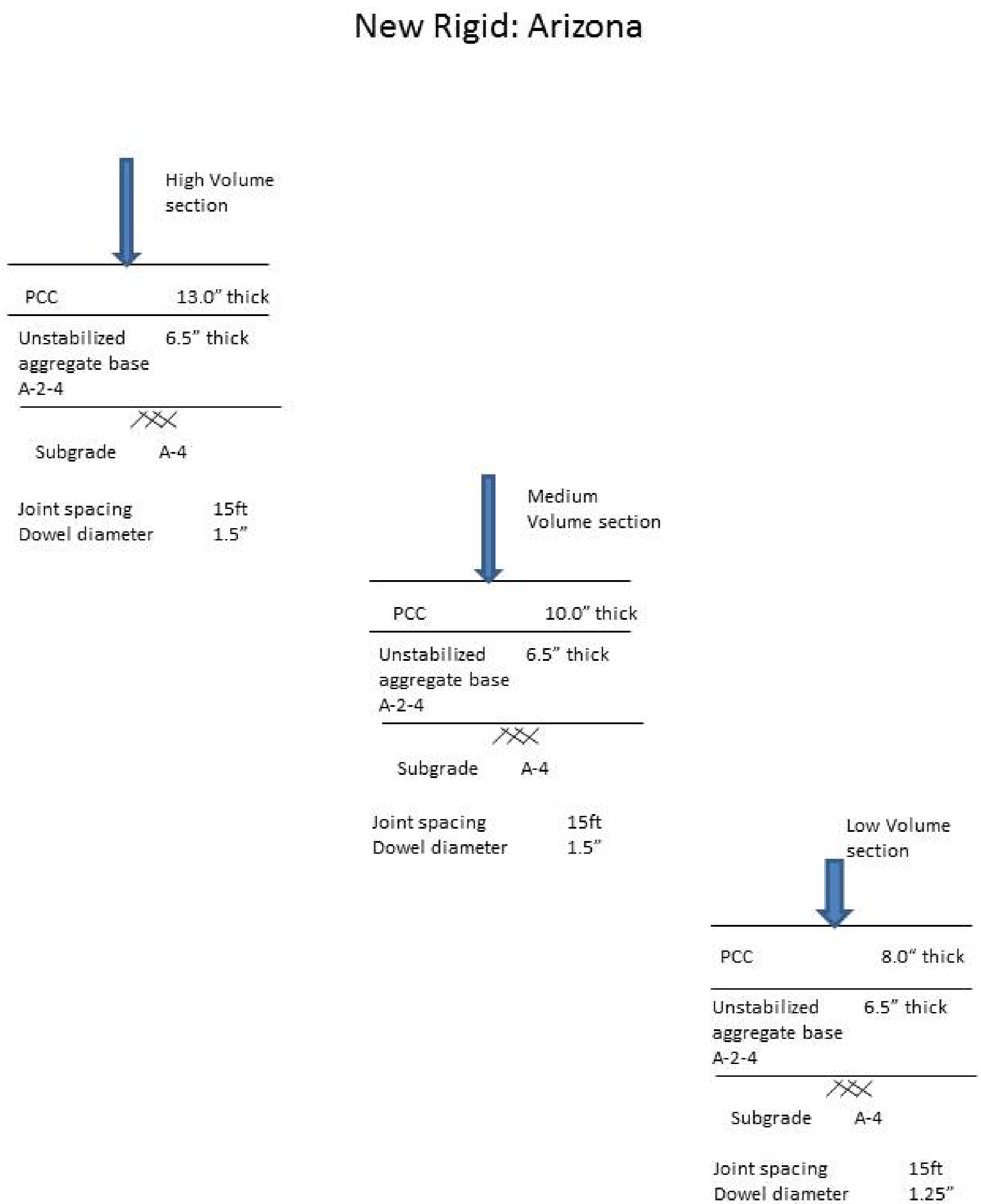
Geographic Location #2 – Rigid Pavement: MISSISSIPPI



Geographic Location #3 – Rigid Pavement: UTAH



Geographic Location #4 – Rigid Pavement: ARIZONA



APPENDIX K – COMPILATION OF BASE CASE PAVEMENT ANALYSIS TABLES

The following tables present a tabulation of the number of years (service life) until predicted distress levels reach a specified trigger value at the mean predicted distress values. Note that the individual pavement sections considered in the 2014 CTSW study analysis, presented here and in Appendices L, M, and N, cover a range of pavement types and design details, climates, traffic levels, axle loads, and other key factors. Since a small number of sections were used in each climate zone and traffic range, the results cannot be used to assess the relative effects of climate or traffic level. Together, the range of sections produces a plausible assessment of the relative effects of each scenario within the parameters and assumptions of the analysis.

Geographic Location #1 Base Case Flexible Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #1 Base Case Flexible Sections		
		HV Int, 11338 ADTT	MV Int, 7206 ADTT	LV Artl, 782 ADTT
IRI @	160	35.527	34.250	37.197
Rutting	0.4	31.983	26.068	> 50
Fatigue Crk @	7.5%	> 50	30.853	> 50

Geographic Location #1 Base Case Rigid Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #1 Base Case Rigid Sections		
		HV Int, 11338 ADTT	MV Int, 7206 ADTT	LV Artl, 782 ADTT
IRI @	160	21.987	22.657	40.529
Faulting @	0.15	26.440	28.553	> 50
Transv Crk @	7.5%	> 50	> 50	> 50

Geographic Location #2 Base Case Flexible Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #2 Base Case Flexible Sections		
		HV Int, 13562 ADTT	MV Int, 7419 ADTT	LV Artl, 895 ADTT
IRI @	160	15.454	15.170	15.755
Rutting	0.4	6.069	4.164	16.158
Fatigue Crk @	7.5%	41.705	7.107	14.226

Geographic Location #2 Base Case Rigid Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #2 Base Case Rigid Sections		
		HV Int, 13562 ADTT	MV Int, 7419 ADTT	LV Artl, 895 ADTT
IRI @	160	46.357	29.317	31.897
Faulting @	0.15	44.103	27.487	31.452
Transv Crk @	7.5%	> 50	> 50	35.532

Geographic Location #3 Base Case Flexible Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #3 Base Case Flexible Sections		
		HV Int, 9824 ADTT	MV Int, 3838 ADTT	LV Artl, 481 ADTT
IRI @	160	37.223	38.277	41.734
Rutting	0.4	15.947	25.212	> 50
Fatigue Crk @	7.5%	> 50	> 50	> 50

Geographic Location #3 Base Case Rigid Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #3 Base Case Rigid Sections		
		HV Int, 9824 ADTT	MV Int, 3838 ADTT	LV Artl, 481 ADTT
IRI @	160	25.226	33.499	> 50
Faulting @	0.15	27.560	43.875	> 50
Transv Crk @	7.5%	> 50	> 50	> 50

Geographic Location #4 Base Case Flexible Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #4 Base Case Flexible Sections		
		HV Int, 9159 ADTT	MV Int, 7489 ADTT	LV Artl, 1391 ADTT
IRI @	160	38.835	35.915	36.973
Rutting	0.4	8.173	5.059	24.243
Fatigue Crk @	7.5%	39.164	13.544	21.768

Geographic Location #4 Base Case Rigid Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #4 Base Case Rigid Sections		
		HV Int, 9159 ADTT	MV Int, 7489 ADTT	LV Artl, 1391 ADTT
IRI @	160	> 50	> 50	> 50
Faulting @	0.15	> 50	> 50	> 50
Transv Crk @	7.5%	> 50	> 50	19.686

APPENDIX L – COMPILATION OF COMPARISON (BASE CASE TO SCENARIOS) PAVEMENT ANALYSIS TABLES

The following tables present a tabulation of the number of years until predicted distress levels reach a specified trigger value (initial service interval) for the base case and for each of six size and weight scenarios (described in the report body), for each pavement section displayed in **Appendix K**.

Geographic Location #1 Flexible Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #1 High Volume Interstate Flexible Section						
		HV Base 11388.0 ADTT	HV Scen 1 11293.6 ADTT	HV Scen 2 11276.0 ADTT	HV Scen 3 11188.7 ADTT	HV Scen 4 11154.2 ADTT	HV Scen 5 11191.8 ADTT	HV Scen 6 11189.5 ADTT
IRI @	160	35.527	35.500	35.553	35.568	35.460	35.610	35.527
Rutting	0.4	31.983	31.960	32.065	32.075	31.223	32.210	32.010
Fatigue Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.023	0.082	0.092	-0.760	0.227	0.028
Change (pct)		-	-0.1%	0.3%	0.3%	-2.4%	0.7%	0.1%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #1 Medium Volume Interstate Flexible Section						
		MV Base, 7206.0 ADTT	MV Scen 1, 7177.8 ADTT	MV Scen 2, 7166.6 ADTT	MV Scen 3, 7111.1 ADTT	MV Scen 4, 7089.2 ADTT	MV Scen 5, 7113.1ADTT	MV Scen 6, 7111.6 ADTT
IRI @	160	34.250	34.110	34.352	34.303	34.140	34.360	34.250
Rutting	0.4	26.068	26.108	26.218	26.086	25.942	27.031	26.091
Fatigue Crk @	7.5%	30.853	27.818	32.298	32.335	28.140	31.335	30.197
Change (years)		-	0.040	0.150	0.018	-0.126	0.963	0.023
Change (pct)		-	0.2%	0.6%	0.1%	-0.5%	3.7%	0.1%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #1 Low Volume Arterial Flexible Section						
		LV Base, 782.0 ADTT	LV Scen 1, 780.1 ADTT	LV Scen 2, 779.3 ADTT	LV Scen 3, 775.6 ADTT	LV Scen 4, 774.5 ADTT	LV Scen 5, 777.7 ADTT	LV Scen 6, 777.5 ADTT
IRI @	160	37.197	37.165	37.277	37.197	37.165	37.197	37.190
Rutting	0.4	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Fatigue Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.032	0.080	0.000	-0.032	0.000	-0.007
Change (pct)		-	-0.1%	0.2%	0.0%	-0.1%	0.0%	0.0%

Geographic Location #1 Rigid Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #1 High Volume Interstate Rigid Section						
		HV Base, 11388.0 ADTT	HV Scen 1, 11293.6 ADTT	HV Scen 2, 11276.0 ADTT	HV Scen 3, 11188.7 ADTT	HV Scen 4, 11154.2 ADTT	HV Scen 5, 11191.8 ADTT	HV Scen 6, 11189.5 ADTT
IRI @	160	21.987	21.963	22.377	22.454	21.882	22.377	22.028
Faulting @	0.15	26.440	26.415	27.290	27.460	26.284	27.290	26.520
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.024	0.390	0.467	-0.105	0.390	0.041
Change (pct)		-	-0.1%	1.8%	2.1%	-0.5%	1.8%	0.2%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #1 Medium Volume Interstate Rigid Section						
		MV Base, 7206.0 ADTT	MV Scen 1, 7177.8 ADTT	MV Scen 2, 7166.6 ADTT	MV Scen 3, 7111.1 ADTT	MV Scen 4, 7089.2 ADTT	MV Scen 5, 7113.1ADTT	MV Scen 6, 7111.6 ADTT
IRI @	160	22.657	22.639	23.081	23.180	22.588	23.081	22.704
Faulting @	0.15	28.553	28.553	29.484	29.710	28.447	29.473	28.682
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.017	0.424	0.523	-0.069	0.424	0.047
Change (pct)		-	-0.1%	1.9%	2.3%	-0.3%	1.9%	0.2%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #1 Low Volume Arterial Rigid Section						
		LV Base, 782.0 ADTT	LV Scen 1, 780.1 ADTT	LV Scen 2, 779.3 ADTT	LV Scen 3, 775.6 ADTT	LV Scen 4, 774.5 ADTT	LV Scen 5, 777.7 ADTT	LV Scen 6, 777.5 ADTT
IRI @	160	40.529	40.503	40.981	41.086	40.260	40.670	40.441
Faulting @	0.15	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.026	0.452	0.557	-0.269	0.141	-0.088
Change (pct)		-	-0.1%	1.1%	1.4%	-0.7%	0.3%	-0.2%

Geographic Location #2 Flexible Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #2 High Volume Interstate Flexible Section						
		HV Base, 13562.0 ADTT	HV Scen 1, 13510.3 ADTT	HV Scen 2, 13490.1 ADTT	HV Scen 3, 13388.3 ADTT	HV Scen 4, 13377.0 ADTT	HV Scen 5, 13413.3 ADTT	HV Scen 6, 13410.7 ADTT
IRI @	160	15.454	15.454	15.470	15.466	15.450	15.477	15.460
Rutting	0.4	6.069	6.050	6.130	6.126	6.009	6.151	6.078
Fatigue Crk @	7.5%	41.705	41.152	42.973	42.234	39.020	42.303	41.210
Change (years)		-	-0.019	0.061	0.057	-0.060	0.082	0.009
Change (pct)		-	-0.3%	1.0%	0.9%	-1.0%	1.3%	0.1%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #2 Medium Volume Interstate Flexible Section						
		MV Base, 7419.0 ADTT	MV Scen 1, 7390.9 ADTT	MV Scen 2, 7379.7 ADTT	MV Scen 3, 7324.0 ADTT	MV Scen 4, 7317.8 ADTT	MV Scen 5, 7337.7 ADTT	MV Scen 6, 7336.2 ADTT
IRI @	160	15.170	15.160	15.190	15.190	15.159	15.190	15.170
Rutting	0.4	4.164	4.155	4.201	4.196	4.143	4.219	4.167
Fatigue Crk @	7.5%	7.107	7.042	7.693	7.710	6.360	7.197	7.076
Change (years)		-	-0.009	0.037	0.032	-0.021	0.055	0.003
Change (pct)		-	-0.2%	0.9%	0.8%	-0.5%	1.3%	0.1%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #2 Low Volume Arterial Flexible Section						
		LV Base, 895.0 ADTT	LV Scen 1, 892.8 ADTT	LV Scen 2, 891.9 ADTT	LV Scen 3, 887.6 ADTT	LV Scen 4, 887.9 ADTT	LV Scen 5, 891.3 ADTT	LV Scen 6, 891.1 ADTT
IRI @	160	15.755	15.750	15.761	15.761	15.750	15.760	15.750
Rutting	0.4	16.158	16.133	16.298	16.277	16.098	16.194	16.142
Fatigue Crk @	7.5%	14.226	14.133	15.121	15.116	13.892	14.186	14.125
Change (years)		-	-0.093	0.895	0.890	-0.334	-0.040	-0.101
Change (pct)		-	-0.7%	6.3%	6.3%	-2.3%	-0.3%	-0.7%

Geographic Location #2 Rigid Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #2 High Volume Interstate Rigid Section						
		Base Traffic, 13562.0 ADTT	Scenario 1, 13510.3 ADTT	Scenario 2, 13490.1 ADTT	Scenario 3, 13388.3 ADTT	Scenario 4, 13377.0 ADTT	Scenario 5, 13413.3 ADTT	Scenario 6, 13410.7 ADTT
IRI @	160	46.357	46.296	47.694	47.972	46.091	47.480	46.484
Faulting @	0.15	44.103	44.040	45.384	45.625	43.803	45.190	44.218
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.063	1.282	1.523	-0.299	1.088	0.115
Change (pct)		-	-0.1%	2.9%	3.5%	-0.7%	2.5%	0.3%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #2 Medium Volume Interstate Rigid Section						
		MV Base, 7419.0 ADTT	MV Scen 1, 7390.9 ADTT	MV Scen 2, 7379.7 ADTT	MV Scen 3, 7324.0 ADTT	MV Scen 4, 7317.8 ADTT	MV Scen 5, 7337.7 ADTT	MV Scen 6, 7336.2 ADTT
IRI @	160	29.317	29.289	30.288	30.503	28.972	30.110	29.370
Faulting @	0.15	27.487	27.468	28.415	28.616	27.152	28.250	27.532
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.019	0.928	1.129	-0.335	0.763	0.045
Change (pct)		-	-0.1%	3.4%	4.1%	-1.2%	2.8%	0.2%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #2 Low Volume Arterial Rigid Section						
		LV Base, 895.0 ADTT	LV Scen 1, 892.8 ADTT	LV Scen 2, 891.9 ADTT	LV Scen 3, 887.6 ADTT	LV Scen 4, 887.9 ADTT	LV Scen 5, 891.3 ADTT	LV Scen 6, 891.1 ADTT
IRI @	160	31.897	31.846	32.681	32.838	31.351	31.918	31.655
Faulting @	0.15	31.452	31.468	32.134	32.344	31.016	31.532	31.218
Transv Crk @	7.5%	35.532	34.678	36.806	36.814	33.564	34.782	34.560
Change (years)		-	0.016	0.682	0.892	-0.436	0.080	-0.234
Change (pct)		-	0.1%	2.2%	2.8%	-1.4%	0.3%	-0.7%

Geographic Location #3 Flexible Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #3 High Volume Interstate Flexible Section						
		HV Base, 9824.0 ADTT	HV Scen 1, 9773.1 ADTT	HV Scen 2, 9753.0 ADTT	HV Scen 3, 9653.9 ADTT	HV Scen 4, 9692.2 ADTT	HV Scen 5, 9719.8 ADTT	HV Scen 6, 9717.6 ADTT
IRI @	160	37.223	37.197	37.318	37.303	37.190	37.223	37.223
Rutting	0.4	15.947	15.245	16.108	16.090	15.221	15.933	15.989
Fatigue Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.702	0.162	0.144	-0.726	-0.013	0.042
Change (pct)		-	-4.4%	1.0%	0.9%	-4.6%	-0.1%	0.3%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #3 Medium Volume Interstate Flexible Section						
		MV Base, 3838.0 ADTT	MV Scen 1, 3818.1 ADTT	MV Scen 2, 3810.3 ADTT	MV Scen 3, 3771.5 ADTT	MV Scen 4, 3786.5 ADTT	MV Scen 5, 3797.3 ADTT	MV Scen 6, 3796.5 ADTT
IRI @	160	38.277	38.432	38.360	38.318	38.223	38.277	38.473
Rutting	0.4	25.212	26.303	26.088	26.040	25.143	25.206	27.049
Fatigue Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	1.091	0.876	0.828	-0.069	-0.006	1.837
Change (pct)		-	4.3%	3.5%	3.3%	-0.3%	0.0%	7.3%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #3 Low Volume Arterial Flexible Section						
		LV Base, 481.0 ADTT	LV Scen 1, 479.5 ADTT	LV Scen 2, 478.8 ADTT	LV Scen 3, 475.7 ADTT	LV Scen 4, 476.9 ADTT	LV Scen 5, 478.7 ADTT	LV Scen 6, 478.1 ADTT
IRI @	160	41.734	41.723	41.777	41.750	41.723	41.723	41.723
Rutting	0.4	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Fatigue Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.011	0.043	0.016	-0.011	-0.011	-0.011
Change (pct)		-	0.0%	0.1%	0.0%	0.0%	0.0%	0.0%

Geographic Location #3 Rigid Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #3 High Volume Interstate Rigid Section						
		HV Base, 9824.0 ADTT	HV Scen 1, 9773.1 ADTT	HV Scen 2, 9753.0 ADTT	HV Scen 3, 9653.9 ADTT	HV Scen 4, 9692.2 ADTT	HV Scen 5, 9719.8 ADTT	HV Scen 6, 9717.6 ADTT
IRI @	160	25.226	25.197	25.974	26.136	25.048	25.201	25.281
Faulting @	0.15	27.560	27.540	28.640	28.947	27.307	27.553	27.610
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.030	0.748	0.910	-0.178	-0.026	0.055
Change (pct)		-	-0.1%	3.0%	3.6%	-0.7%	-0.1%	0.2%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #3 Medium Volume Interstate Rigid Section						
		MV Base, 3838.0 ADTT	MV Scen 1, 3818.1 ADTT	MV Scen 2, 3810.3 ADTT	MV Scen 3, 3771.5 ADTT	MV Scen 4, 3786.5 ADTT	MV Scen 5, 3797.3 ADTT	MV Scen 6, 3796.5 ADTT
IRI @	160	33.499	33.483	34.425	34.677	33.423	33.524	33.570
Faulting @	0.15	43.875	43.875	45.540	46.000	43.710	43.915	43.960
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-0.016	0.926	1.178	-0.076	0.025	0.071
Change (pct)		-	0.0%	2.8%	3.5%	-0.2%	0.1%	0.2%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #3 Low Volume Arterial Rigid Section						
		LV Base, 481.0 ADTT	LV Scen 1, 479.5 ADTT	LV Scen 2, 478.8 ADTT	LV Scen 3, 475.7 ADTT	LV Scen 4, 476.9 ADTT	LV Scen 5, 478.7 ADTT	LV Scen 6, 478.1 ADTT
IRI @	160	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Faulting @	0.15	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	0.000	0.000	0.000	0.000	0.000	0.000
Change (pct)		-	-	-	-	-	-	-

Geographic Location #4 Flexible Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #4 High Volume Interstate Flexible Section						
		HV Base, 9159.0 ADTT	HV Scen 1, 9117.5 ADTT	HV Scen 2, 9101.4 ADTT	HV Scen 3, 9023.1 ADTT	HV Scen 4, 9032.0 ADTT	HV Scen 5, 9056.5 ADTT	HV Scen 6, 9054.4 ADTT
IRI @	160	39.020	39.000	39.027	39.027	39.000	39.020	39.011
Rutting	0.4	8.183	8.152	8.266	8.191	8.130	8.226	8.193
Fatigue Crk @	7.5%	> 50	> 50	> 50	> 50	49.125	> 50	> 50
Chg (yrs)		-	-0.031	0.083	0.008	-0.053	0.043	0.010
Change (pct)		-	-0.4%	1.0%	0.1%	-0.6%	0.5%	0.1%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #4 Medium Volume Interstate Flexible Section						
		MV Base, 7489.0 ADTT	MV Scen 1, 7455.1 ADTT	MV Scen 2, 7441.9 ADTT	MV Scen 3, 7377.9 ADTT	MV Scen 4, 7385.2 ADTT	MV Scen 5, 7405.2 ADTT	MV Scen 6, 7403.5 ADTT
IRI @	160	35.915	35.890	35.989	35.980	35.860	35.915	35.915
Rutting	0.4	5.059	5.042	5.088	5.061	5.034	5.077	5.063
Fatigue Crk @	7.5%	13.544	13.195	14.509	14.557	12.447	13.350	13.300
Chg (yrs)		-	-0.017	0.029	0.002	-0.024	0.018	0.004
Change (pct)		-	-0.3%	0.6%	0.0%	-0.5%	0.4%	0.1%

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #4 Low Volume Arterial Flexible Section						
		LV Base, 1391.0 ADTT	LV Scen 1, 1386.4 ADTT	LV Scen 2, 1384.6 ADTT	LV Scen 3, 1376.1 ADTT	LV Scen 4, 1376.9 ADTT	LV Scen 5, 1380.6 ADTT	LV Scen 6, 1380.1 ADTT
IRI @	160	> 50	> 50	> 50	> 50	36.777	36.915	36.890
Rutting	0.4	24.243	24.144	24.625	24.228	24.122	24.320	24.223
Fatigue Crk @	7.5%	21.768	21.221	23.265	23.191	20.523	21.292	21.206
Chg (yrs)		-	-0.547	1.497	1.423	-1.245	-0.476	-0.562
Change (pct)		-	-2.5%	6.9%	6.5%	-5.7%	-2.2%	-2.6%

Geographic Location #4 Rigid Sections

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #4 High Volume Interstate Rigid Section						
		HV Base, 9159.0 ADTT	HV Scen 1, 9117.5 ADTT	HV Scen 2, 9101.4 ADTT	HV Scen 3, 9023.1 ADTT	HV Scen 4, 9032.0 ADTT	HV Scen 5, 9056.5 ADTT	HV Scen 6, 9054.4 ADTT
IRI @	160	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Faulting @	0.15	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-	-	-	-	-	-
Change (pct)		-	-	-	-	-	-	-

Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #4 Medium Volume Interstate Rigid Section						
		MV Base, 7489.0 ADTT	MV Scen 1, 7455.1 ADTT	MV Scen 2, 7441.9 ADTT	MV Scen 3, 7377.9 ADTT	MV Scen 4, 7385.2 ADTT	MV Scen 5, 7405.2 ADTT	MV Scen 6, 7403.5 ADTT
IRI @	160	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Faulting @	0.15	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Transv Crk @	7.5%	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Change (years)		-	-	-	-	-	-	-
Change (pct)		-	-	-	-	-	-	-

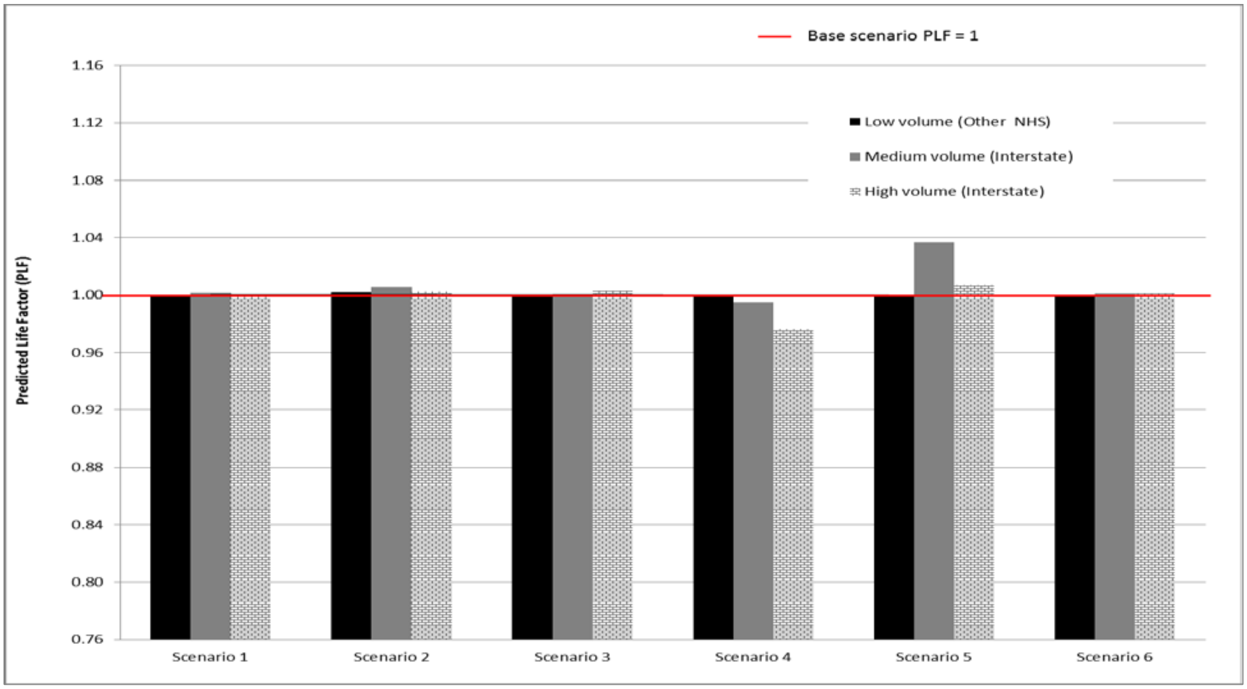
Rehab Trigger Time (Years) at Predicted Mean Values		Geographic Location #4 Low Volume Arterial Rigid Section						
		LV Base, 1391.0 ADTT	LV Scen 1, 1386.4 ADTT	LV Scen 2, 1384.6 ADTT	LV Scen 3, 1376.1 ADTT	LV Scen 4, 1376.9 ADTT	LV Scen 5, 1380.6 ADTT	LV Scen 6, 1380.1 ADTT
IRI @	160	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Faulting @	0.15	> 50	> 50	> 50	> 50	> 50	> 50	> 50
Transv Crk @	7.5%	19.686	18.790	21.307	21.628	17.730	18.835	18.690
Change (years)		-	-0.896	1.621	1.942	-1.956	-0.851	-0.996
Change (pct)		-	-4.6%	8.2%	9.9%	-9.9%	-4.3%	-5.1%

Service Interval Percent Changes by Scenario

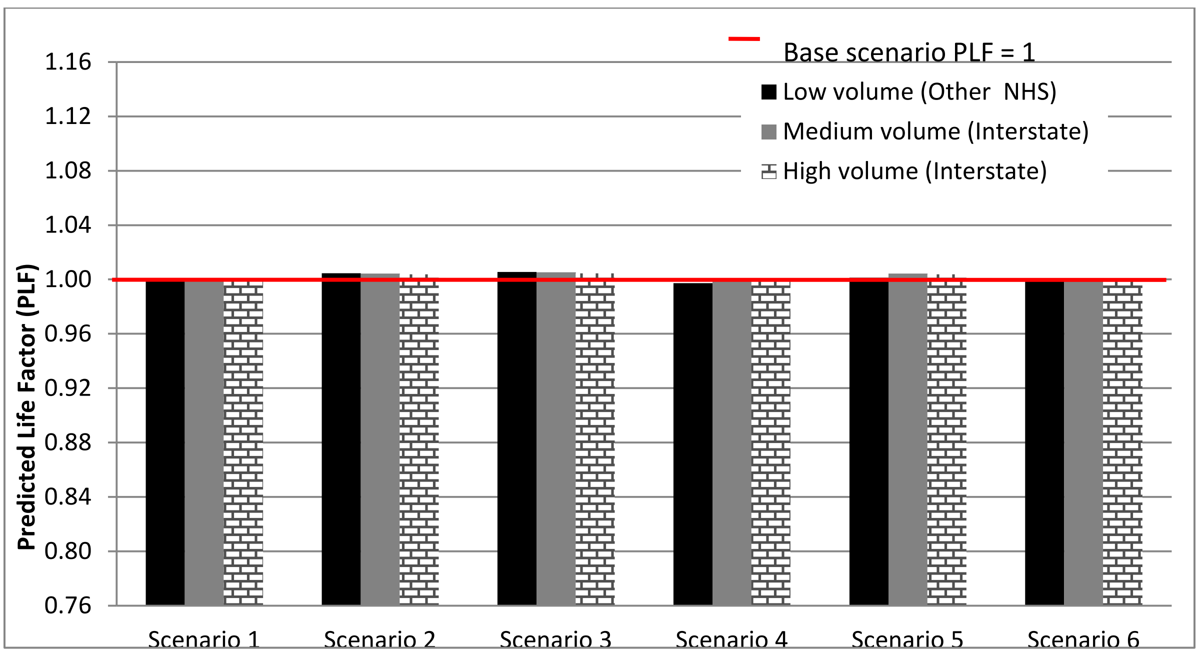
Scenario	1	2	3	4	5	6
Weighted Average % Change in Service Interval	-0.3	+2.7	+2.7	-1.6	-0.0	-0.1

APPENDIX M – IMPACTS OF SCENARIOS ON PREDICTED LIFE FACTORS

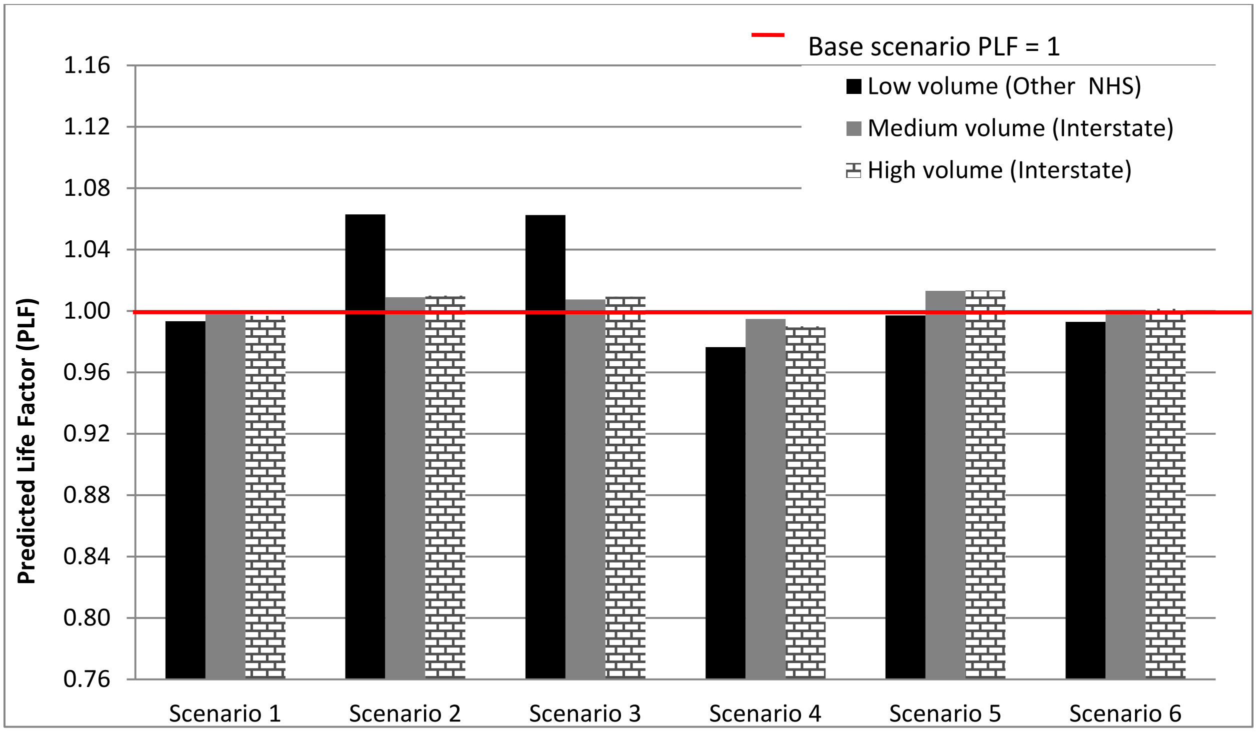
This appendix presents the predicted life factors for the flexible and rigid pavements in each geographic location. The observed differences in the impacts of the different scenarios among the four geographic locations reflect the differing combinations of pavement cross-section, climatic conditions, and traffic-loading estimates.



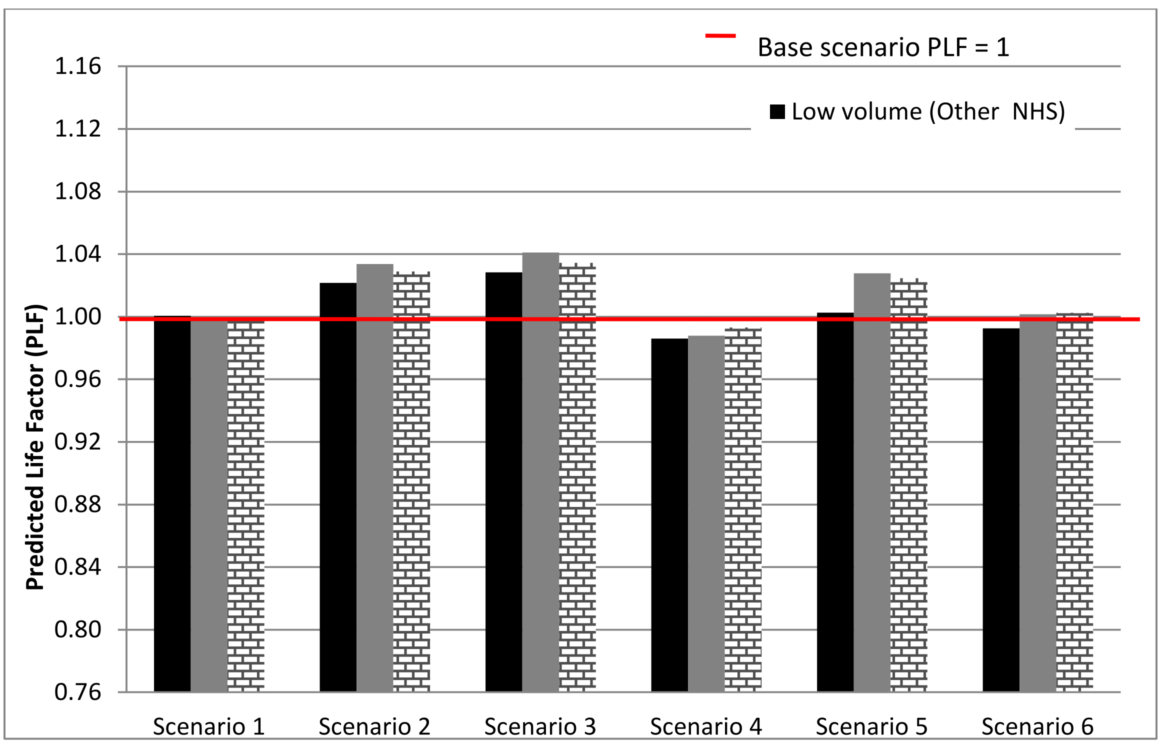
Impacts of Scenarios on Predicted Life Factor for Flexible Pavements in Geographic Location #1



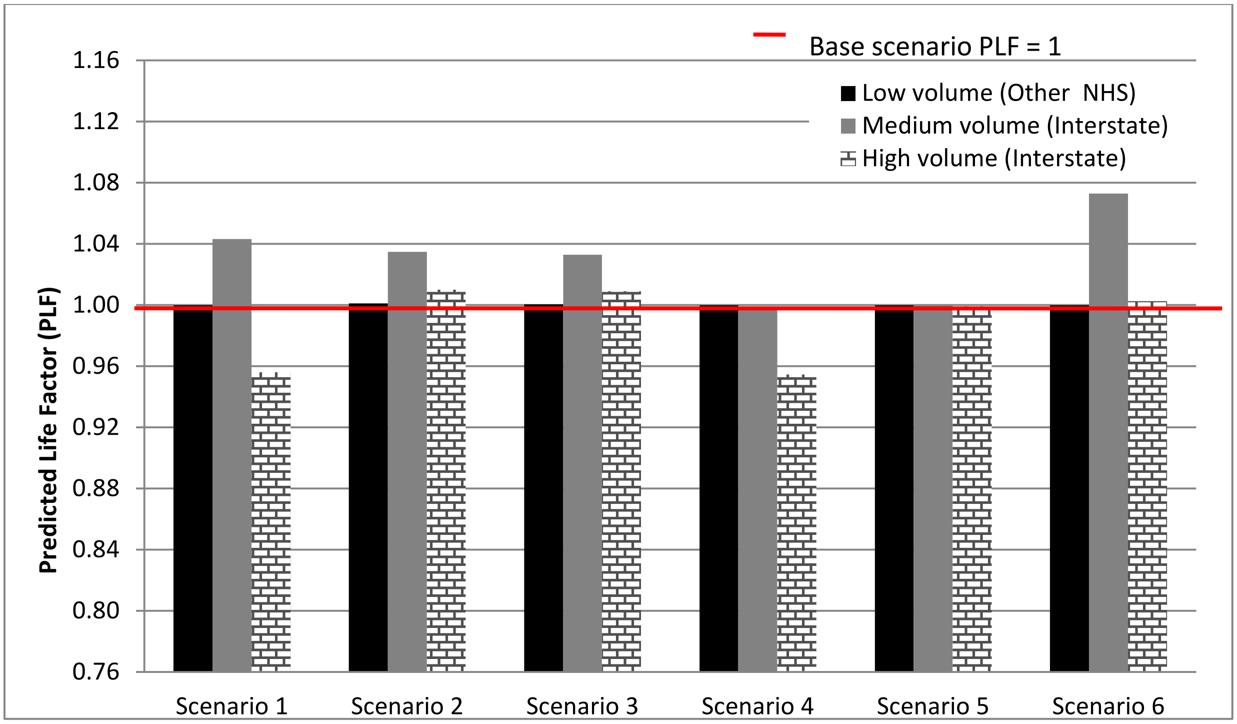
Impacts of Scenarios on Predicted Life Factor for Rigid Pavements in Geographic Location #1



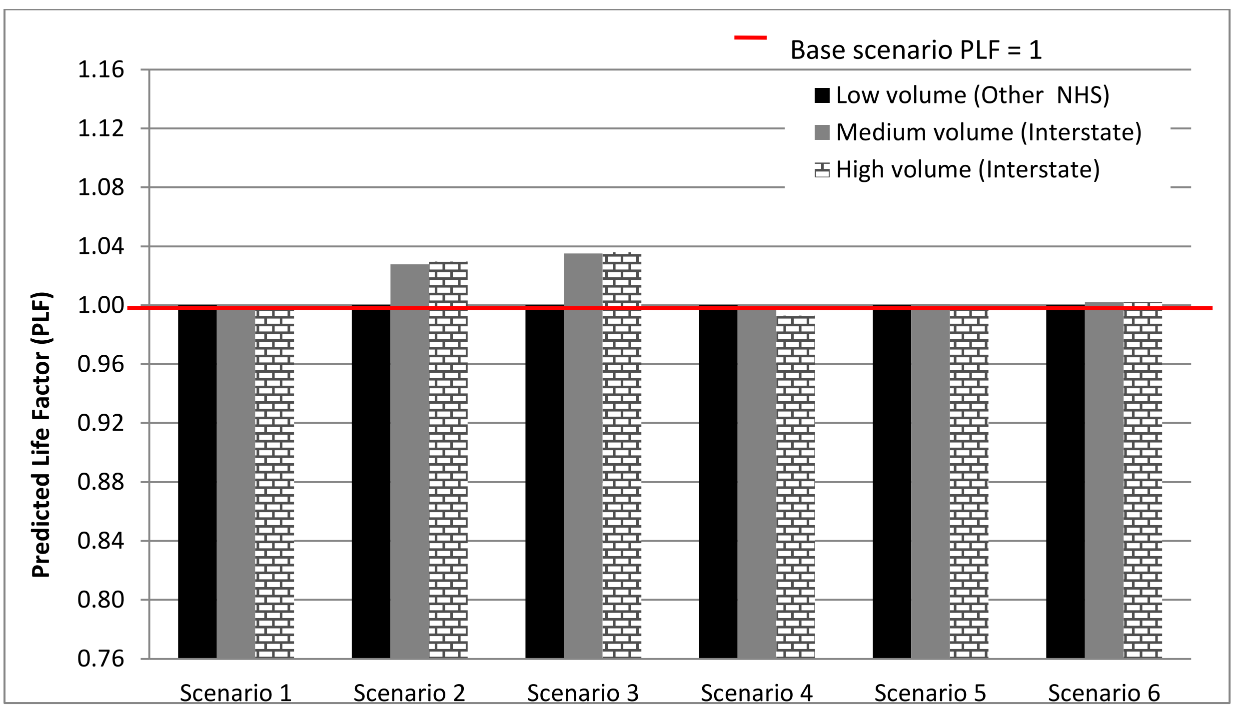
Impacts of Scenarios on Predicted Life Factor for Flexible Pavements in Geographic Location #2



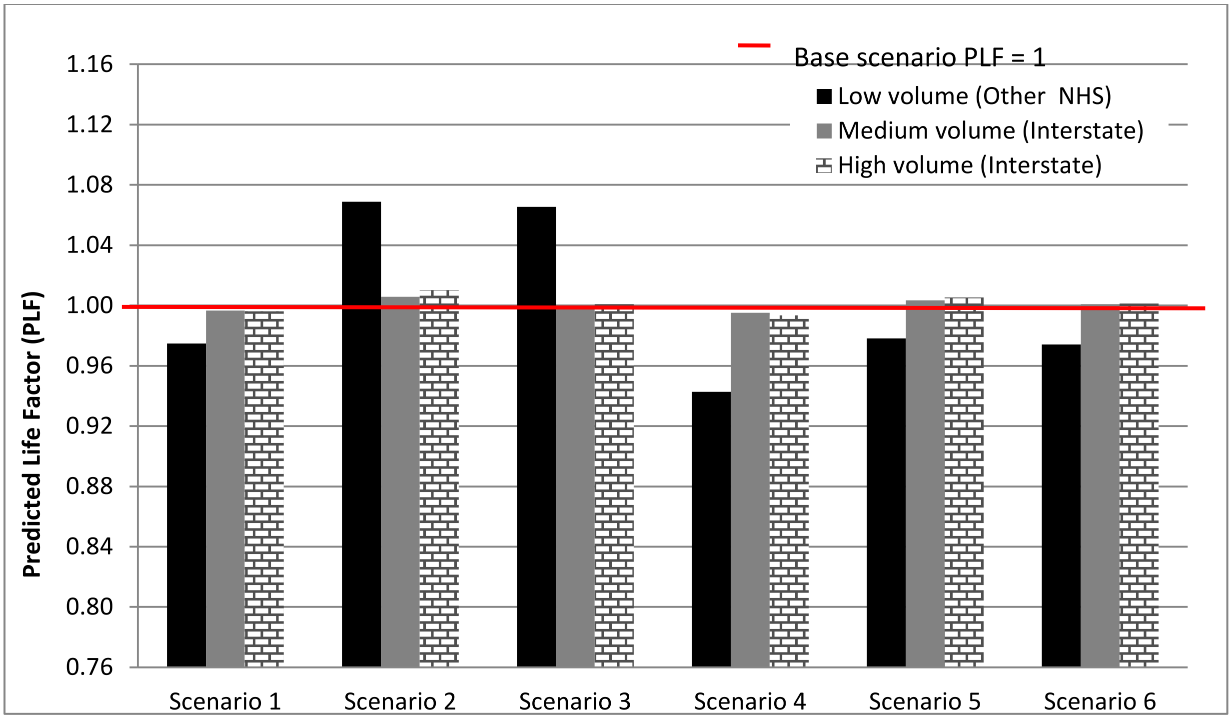
Impacts of Scenarios on Predicted Life Factor for Rigid Pavements in Geographic Location #2



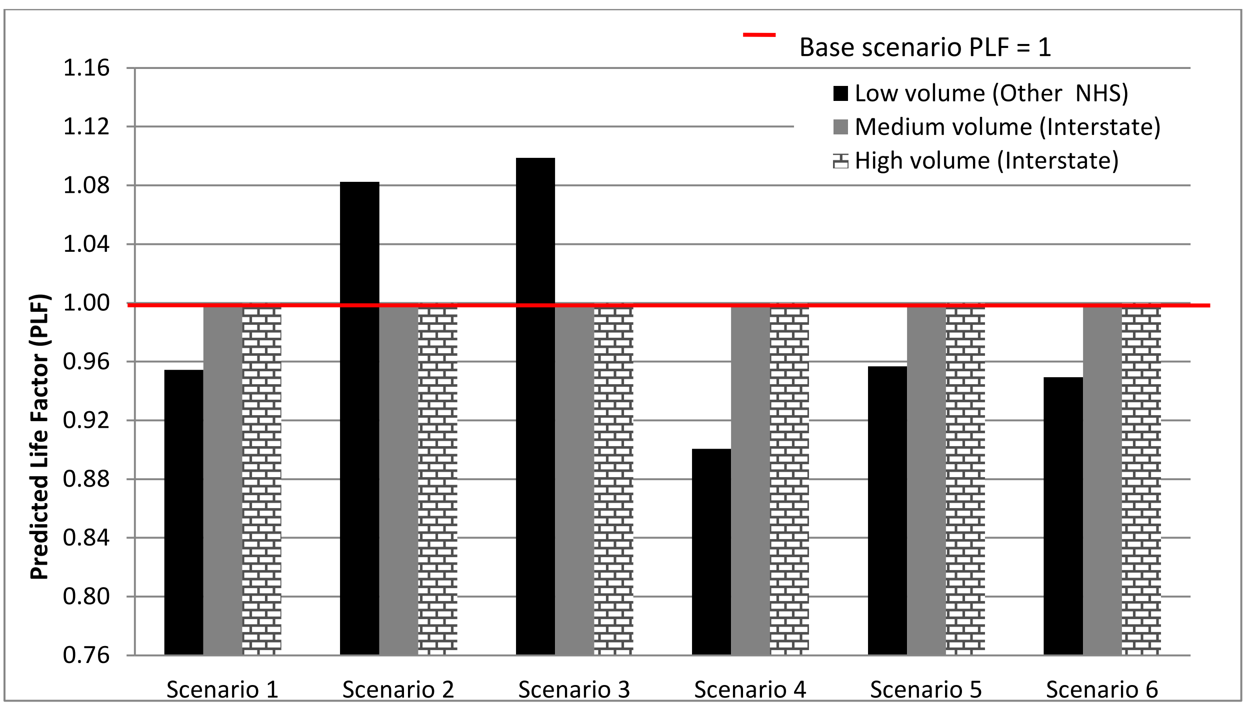
Impacts of Scenarios on Predicted Life Factor for Flexible Pavements in Geographic Location #3



Impacts of Scenarios on Predicted Life Factor for Rigid Pavements in in Geographic Location #3



Impacts of Scenarios on Predicted Life Factor for Flexible Pavements in Geographic Location #4



Impacts of Scenarios on Predicted Life Factor for Rigid Pavements in Geographic Location #4

APPENDIX N –PREDICTED PAVEMENT LIFE CYCLE COSTS FOR FLEXIBLE AND RIGID PAVEMENTS

Life Cycle Cost Analyses for Each Sample Pavement Section and Each Scenario
Part 1: Using 1.9% Discount Rate

Life Cycle Costs for:		Flexible	#1	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 65,415**	**$ 65,514**	**$ 65,059**	**$ 65,015**	**$ 68,762**	**$ 64,430**	**$ 65,295**
Time of Activity 1	31.98	31.96	32.06	32.07	31.22	32.21	32.01
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 46,781	$ 46,801	$ 46,707	$ 46,698	$ 47,467	$ 46,577	$ 46,756
Time of Activity 2	43.98	43.96	44.06	44.07	43.22	44.21	44.01
Cost 2	$ 43,324	$ 43,488	$ 42,735	$ 42,662	$ 48,793	$ 41,688	$ 43,126
Net Present Cost 2	$ 18,634	$ 18,713	$ 18,352	$ 18,317	$ 21,295	$ 17,852	$ 18,539

Life Cycle Costs for:		Flexible	#1	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 93,792**	**$ 93,581**	**$ 93,003**	**$ 93,699**	**$ 94,416**	**$ 88,795**	**$ 93,673**
Time of Activity 1	26.07	26.11	26.22	26.09	25.94	27.03	26.09
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 52,401	$ 52,361	$ 52,251	$ 52,384	$ 52,528	$ 51,442	$ 52,379
Time of Activity 2	38.07	38.11	38.22	38.09	37.94	39.03	38.09
Cost 2	$ 85,910	$ 85,622	$ 84,830	$ 85,784	$ 86,400	$ 78,976	$ 85,748
Net Present Cost 2	$ 41,391	$ 41,220	$ 40,753	$ 41,315	$ 41,727	$ 37,353	$ 41,294
Time of Activity 3	50.07	50.11	50.22	50.09	49.94	51.03	50.09
Cost 3	$ -	$ -	$ -	$ -	$ 419	$ -	$ -
Net Present Cost 3	$ -	$ -	$ -	$ -	$ 161	$ -	$ -

Life Cycle Costs for:		Flexible	#1	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 41,277**	**$ 41,384**	**$ 41,006**	**$ 41,277**	**$ 41,384**	**$ 41,277**	**$ 41,299**
Time of Activity 1	37.20	37.17	37.28	37.20	37.17	37.20	37.19
Cost 1	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000
Net Present Cost 1	$ 39,193	$ 39,216	$ 39,133	$ 39,193	$ 39,216	$ 39,193	$ 39,198
Time of Activity 2	49.20	49.17	49.28	49.20	49.17	49.20	49.19
Cost 2	$ 5,356	$ 5,567	$ 4,822	$ 5,356	$ 5,567	$ 5,356	$ 5,400
Net Present Cost 2	$ 2,084	$ 2,168	$ 1,874	$ 2,084	$ 2,168	$ 2,084	$ 2,102

Life Cycle Costs for:		Rigid	#1	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	IRI	IRI	IRI	IRI	IRI	IRI	IRI
IRI	**$ 26,091.2**	**$ 26,122.2**	**$ 25,590.7**	**$ 25,492.7**	**$ 26,226.7**	**$ 25,590.7**	**$ 26,037.7**
Time of Activity 1	21.99	21.96	22.38	22.45	21.88	22.38	22.03
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 15,807	$ 15,814	$ 15,689	$ 15,666	$ 15,839	$ 15,689	$ 15,794
Time of Activity 2	36.99	36.96	37.38	37.45	36.88	37.38	37.03
Cost 2	$ 20,908	$ 20,947	$ 20,281	$ 20,158	$ 21,076	$ 20,281	$ 20,841
Net Present Cost 2	$ 10,284	$ 10,308	$ 9,902	$ 9,827	$ 10,388	$ 9,902	$ 10,243

Life Cycle Costs for:		Rigid	#1	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	IRI	IRI	IRI	IRI	IRI	IRI	IRI
IRI	**$ 25,235**	**$ 25,257**	**$ 24,703**	**$ 24,579**	**$ 25,322**	**$ 24,703**	**$ 25,176**
Time of Activity 1	22.66	22.64	23.08	23.18	22.59	23.08	22.70
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 15,605	$ 15,610	$ 15,479	$ 15,449	$ 15,626	$ 15,479	$ 15,591
Time of Activity 2	37.66	37.64	38.08	38.18	37.59	38.08	37.70
Cost 2	$ 19,832	$ 19,859	$ 19,151	$ 18,991	$ 19,942	$ 19,151	$ 19,756
Net Present Cost 2	$ 9,630	$ 9,647	$ 9,224	$ 9,130	$ 9,697	$ 9,224	$ 9,585

Life Cycle Costs for:		Rigid	#1	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	IRI	IRI	IRI	IRI	IRI	IRI	IRI
IRI	**$ 11,076**	**$ 11,081**	**$ 10,980**	**$ 10,958**	**$ 11,133**	**$ 11,046**	**$ 11,094**
Time of Activity 1	40.53	40.50	40.98	41.09	40.26	40.67	40.44
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 11,076	$ 11,081	$ 10,980	$ 10,958	$ 11,133	$ 11,046	$ 11,094

Life Cycle Costs for:		Flexible	#2	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Rut	Rut	Rut	Rut	Rut	Rut	Rut
Rut	**$ 212,006**	**$ 212,146**	**$ 211,560**	**$ 211,593**	**$ 212,445**	**$ 211,413**	**$ 211,941**
Time of Activity 1	6.07	6.05	6.13	6.13	6.01	6.15	6.08
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 76,905	$ 76,933	$ 76,815	$ 76,821	$ 76,994	$ 76,785	$ 76,892
Time of Activity 2	18.07	18.05	18.13	18.13	18.01	18.15	18.08
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 61,092	$ 61,114	$ 61,020	$ 61,025	$ 61,162	$ 60,996	$ 61,081
Time of Activity 3	30.07	30.05	30.13	30.13	30.01	30.15	30.08
Cost 3	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 3	$ 48,530	$ 48,548	$ 48,473	$ 48,477	$ 48,586	$ 48,454	$ 48,522
Time of Activity 4	42.07	42.05	42.13	42.13	42.01	42.15	42.08
Cost 4	$ 57,104	$ 57,242	$ 56,663	$ 56,695	$ 57,538	$ 56,516	$ 57,040
Net Present Cost 4	$ 25,480	$ 25,550	$ 25,253	$ 25,269	$ 25,703	$ 25,178	$ 25,447

Life Cycle Costs for:		Flexible	#2	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Rut	Rut	Rut	Rut	Rut	Rut	Rut
Rut	**$ 226,244**	**$ 226,309**	**$ 225,958**	**$ 226,002**	**$ 226,406**	**$ 225,824**	**$ 226,217**
Time of Activity 1	4.16	4.16	4.20	4.20	4.14	4.22	4.17
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 79,767	$ 79,780	$ 79,710	$ 79,719	$ 79,799	$ 79,683	$ 79,762
Time of Activity 2	16.16	16.16	16.20	16.20	16.14	16.22	16.17
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 63,365	$ 63,376	$ 63,320	$ 63,327	$ 63,391	$ 63,299	$ 63,361
Time of Activity 3	28.16	28.16	28.20	28.20	28.14	28.22	28.17
Cost 3	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 3	$ 50,336	$ 50,344	$ 50,300	$ 50,306	$ 50,357	$ 50,283	$ 50,333
Time of Activity 4	40.16	40.16	40.20	40.20	40.14	40.22	40.17
Cost 4	$ 70,819	$ 70,881	$ 70,551	$ 70,592	$ 70,972	$ 70,425	$ 70,795
Net Present Cost 4	$ 32,775	$ 32,809	$ 32,628	$ 32,650	$ 32,859	$ 32,559	$ 32,762

Life Cycle Costs for:		Flexible	#2	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Crack	Crack	Crack	Crack	Crack	Crack	Crack
Crack	**$ 151,841**	**$ 152,413**	**$ 146,440**	**$ 146,469**	**$ 153,823**	**$ 152,086**	**$ 152,459**
Time of Activity 1	14.23	14.13	15.12	15.12	13.89	14.19	14.13
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 65,765	$ 65,883	$ 64,646	$ 64,652	$ 66,188	$ 65,816	$ 65,893
Time of Activity 2	26.23	26.13	27.12	27.12	25.89	26.19	26.13
Cost 2	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000
Net Present Cost 2	$ 48,373	$ 48,460	$ 47,549	$ 47,554	$ 48,684	$ 48,410	$ 48,467
Time of Activity 3	38.23	38.13	39.12	39.12	37.89	38.19	38.13
Cost 3	$ 78,493	$ 79,117	$ 72,527	$ 72,560	$ 80,000	$ 78,760	$ 79,167
Net Present Cost 3	$ 37,703	$ 38,070	$ 34,244	$ 34,263	$ 38,674	$ 37,860	$ 38,100
Time of Activity 4	50.23	50.13	51.12	51.12	49.89	50.19	50.13
Cost 4	$ -	$ -	$ -	$ -	$ 721	$ -	$ -
Net Present Cost 4	$ -	$ -	$ -	$ -	$ 277	$ -	$ -

Life Cycle Costs for:		Rigid	#2	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Fault	Fault	Fault	Fault	Fault	Fault	Fault
Fault	**$ 10,342**	**$ 10,354**	**$ 10,091**	**$ 10,044**	**$ 10,401**	**$ 10,128**	**$ 10,319**
Time of Activity 1	44.10	44.04	45.38	45.63	43.80	45.19	44.22
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 10,342	$ 10,354	$ 10,091	$ 10,044	$ 10,401	$ 10,128	$ 10,319

Life Cycle Costs for:		Rigid	#2	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Fault	Fault	Fault	Fault	Fault	Fault	Fault
Fault	**$ 19,567**	**$ 19,588**	**$ 18,573**	**$ 18,362**	**$ 19,933**	**$ 18,748**	**$ 19,518**
Time of Activity 1	27.49	27.47	28.42	28.62	27.15	28.25	27.53
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 14,224	$ 14,229	$ 13,973	$ 13,919	$ 14,316	$ 14,017	$ 14,212
Time of Activity 2	42.49	42.47	43.42	43.62	42.15	43.25	42.53
Cost 2	$ 12,071	$ 12,101	$ 10,580	$ 10,257	$ 12,609	$ 10,845	$ 11,999
Net Present Cost 2	$ 5,343	$ 5,358	$ 4,600	$ 4,443	$ 5,617	$ 4,731	$ 5,306

Life Cycle Costs for:		Rigid	#2	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Fault	Fault	Fault	Fault	Fault	Fault	Fault
Fault	**$ 15,521**	**$ 15,505**	**$ 14,875**	**$ 14,679**	**$ 15,941**	**$ 15,444**	**$ 15,745**
Time of Activity 1	31.45	31.47	32.13	32.34	31.02	31.53	31.22
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 13,182	$ 13,178	$ 13,011	$ 12,959	$ 13,293	$ 13,162	$ 13,242
Time of Activity 2	46.45	46.47	47.13	47.34	46.02	46.53	46.22
Cost 2	$ 5,700	$ 5,675	$ 4,605	$ 4,267	$ 6,401	$ 5,572	$ 6,076
Net Present Cost 2	$ 2,338	$ 2,327	$ 1,864	$ 1,721	$ 2,648	$ 2,282	$ 2,504

Life Cycle Costs for:		Flexible	#3	HV			
	Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 147,816**	**$ 152,197**	**$ 146,819**	**$ 146,930**	**$ 152,351**	**$ 147,898**	**$ 147,557**
Time of Activity 1	15.95	15.25	16.11	16.09	15.22	15.93	15.99
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 63,630	$ 64,492	$ 63,433	$ 63,455	$ 64,522	$ 63,646	$ 63,579
Time of Activity 2	27.95	27.25	28.11	28.09	27.22	27.93	27.99
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 50,546	$ 51,231	$ 50,390	$ 50,407	$ 51,255	$ 50,559	$ 50,506
Time of Activity 3	39.95	39.25	40.11	40.09	39.22	39.93	39.99
Cost 3	$ 72,384	$ 77,435	$ 71,220	$ 71,350	$ 77,609	$ 72,480	$ 72,082
Net Present Cost 3	$ 33,639	$ 36,474	$ 32,996	$ 33,068	$ 36,573	$ 33,693	$ 33,472

Life Cycle Costs for:		Flexible	#3	MV			
	Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 97,792**	**$ 92,557**	**$ 93,686**	**$ 93,939**	**$ 98,115**	**$ 97,820**	**$ 88,703**
Time of Activity 1	25.21	26.30	26.09	26.04	25.14	25.21	27.05
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 53,269	$ 52,165	$ 52,381	$ 52,429	$ 53,339	$ 53,275	$ 51,424
Time of Activity 2	37.21	38.30	38.09	38.04	37.14	37.21	39.05
Cost 2	$ 86,400	$ 84,216	$ 85,765	$ 86,112	$ 86,400	$ 86,400	$ 78,846
Net Present Cost 2	$ 42,316	$ 40,391	$ 41,305	$ 41,510	$ 42,372	$ 42,321	$ 37,278
Time of Activity 3	49.21	50.30	50.09	50.04	49.14	49.21	51.05
Cost 3	$ 5,674	$ -	$ -	$ -	$ 6,170	$ 5,717	$ -
Net Present Cost 3	$ 2,208	$ -	$ -	$ -	$ 2,404	$ 2,224	$ -

Life Cycle Costs for:		Flexible	#3	LV Artl			
	Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 35,926**	**$ 35,933**	**$ 35,896**	**$ 35,915**	**$ 35,933**	**$ 35,933**	**$ 35,933**
Time of Activity 1	41.73	41.72	41.78	41.75	41.72	41.72	41.72
Cost 1	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000
Net Present Cost 1	$ 35,926	$ 35,933	$ 35,896	$ 35,915	$ 35,933	$ 35,933	$ 35,933

Life Cycle Costs for:		Rigid	#3	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 22,113**	**$ 22,148**	**$ 21,251**	**$ 21,066**	**$ 22,322**	**$ 22,143**	**$ 22,049**
Time of Activity 1	25.23	25.20	25.97	26.14	25.05	25.20	25.28
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 14,855	$ 14,863	$ 14,643	$ 14,597	$ 14,905	$ 14,862	$ 14,839
Time of Activity 2	40.23	40.20	40.97	41.14	40.05	40.20	40.28
Cost 2	$ 15,703	$ 15,751	$ 14,502	$ 14,241	$ 15,990	$ 15,744	$ 15,615
Net Present Cost 2	$ 7,259	$ 7,285	$ 6,608	$ 6,469	$ 7,417	$ 7,281	$ 7,210

Life Cycle Costs for:		Rigid	#3	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 13,626**	**$ 13,640**	**$ 12,809**	**$ 12,592**	**$ 13,694**	**$ 13,603**	**$ 13,563**
Time of Activity 1	33.50	33.48	34.43	34.68	33.42	33.52	33.57
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 12,675	$ 12,679	$ 12,451	$ 12,392	$ 12,693	$ 12,669	$ 12,657
Time of Activity 2	48.50	48.48	49.43	49.68	48.42	48.52	48.57
Cost 2	$ 2,412	$ 2,438	$ 923	$ 519	$ 2,534	$ 2,371	$ 2,298
Net Present Cost 2	$ 951	$ 962	$ 358	$ 200	$ 1,001	$ 935	$ 905

Life Cycle Costs for:		Rigid	#3	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**None**	**None**	**None**	**None**	**None**	**None**	**None**
	$ -	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**
Time of Activity 1	> 50	> 50	> 50	> 50	> 50	> 50	> 50

Life Cycle Costs for:		Flexible	#4	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Rut	Rut	Rut	Rut	Rut	Rut	Rut
Rut	**$ 197,059**	**$ 197,271**	**$ 196,489**	**$ 197,006**	**$ 197,422**	**$ 196,763**	**$ 196,988**
Time of Activity 1	8.18	8.15	8.27	8.19	8.13	8.23	8.19
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 73,849	$ 73,892	$ 73,731	$ 73,838	$ 73,924	$ 73,788	$ 73,834
Time of Activity 2	20.18	20.15	20.27	20.19	20.13	20.23	20.19
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 58,664	$ 58,699	$ 58,570	$ 58,655	$ 58,723	$ 58,615	$ 58,652
Time of Activity 3	32.18	32.15	32.27	32.19	32.13	32.23	32.19
Cost 3	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 3	$ 46,601	$ 46,629	$ 46,527	$ 46,594	$ 46,649	$ 46,563	$ 46,592
Time of Activity 4	44.18	44.15	44.27	44.19	44.13	44.23	44.19
Cost 4	$ 41,883	$ 42,106	$ 41,285	$ 41,827	$ 42,264	$ 41,573	$ 41,809
Net Present Cost 4	$ 17,945	$ 18,051	$ 17,661	$ 17,919	$ 18,127	$ 17,798	$ 17,910

Life Cycle Costs for:		Flexible	#4	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Rut	Rut	Rut	Rut	Rut	Rut	Rut
Rut	**$ 219,462**	**$ 219,587**	**$ 219,245**	**$ 219,447**	**$ 219,645**	**$ 219,329**	**$ 219,429**
Time of Activity 1	5.06	5.04	5.09	5.06	5.03	5.08	5.06
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 78,409	$ 78,434	$ 78,366	$ 78,406	$ 78,446	$ 78,383	$ 78,403
Time of Activity 2	17.06	17.04	17.09	17.06	17.03	17.08	17.06
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 62,287	$ 62,307	$ 62,252	$ 62,284	$ 62,316	$ 62,266	$ 62,282
Time of Activity 3	29.06	29.04	29.09	29.06	29.03	29.08	29.06
Cost 3	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 3	$ 49,479	$ 49,495	$ 49,452	$ 49,478	$ 49,503	$ 49,463	$ 49,475
Time of Activity 4	41.06	41.04	41.09	41.06	41.03	41.08	41.06
Cost 4	$ 64,376	$ 64,496	$ 64,167	$ 64,362	$ 64,552	$ 64,248	$ 64,344
Net Present Cost 4	$ 29,286	$ 29,350	$ 29,175	$ 29,279	$ 29,380	$ 29,218	$ 29,269

Life Cycle Costs for:		Flexible	#4	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Crack	Crack	Crack	Crack	Crack	Crack	Crack
Crack	**$ 110,491**	**$ 113,186**	**$ 103,331**	**$ 103,679**	**$ 116,695**	**$ 112,836**	**$ 113,264**
Time of Activity 1	21.77	21.22	23.27	23.19	20.52	21.29	21.21
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 56,907	$ 57,507	$ 55,296	$ 55,375	$ 58,282	$ 57,429	$ 57,524
Time of Activity 2	33.77	33.22	35.27	35.19	32.52	33.29	33.21
Cost 2	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000
Net Present Cost 2	$ 41,857	$ 42,298	$ 40,672	$ 40,730	$ 42,869	$ 42,241	$ 42,311
Time of Activity 3	45.77	45.22	47.27	47.19	44.52	45.29	45.21
Cost 3	$ 28,215	$ 31,859	$ 18,232	$ 18,728	$ 36,514	$ 31,388	$ 31,963
Net Present Cost 3	$ 11,727	$ 13,381	$ 7,363	$ 7,574	$ 15,543	$ 13,166	$ 13,429

Life Cycle Costs for:	Rigid	#4	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	None	None	None	None	None	None	None
	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Time of Activity 1	> 50	> 50	> 50	> 50	> 50	> 50	> 50

Life Cycle Costs for:	Rigid	#4	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	None	None	None	None	None	None	None
	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Time of Activity 1	> 50	> 50	> 50	> 50	> 50	> 50	> 50

Life Cycle Costs for:		Rigid	#4	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Crack	Crack	Crack	Crack	Crack	Crack	Crack
Crack	$ 99,701	$ 103,677	$ 92,775	$ 91,440	$ 108,520	$ 103,475	$ 104,127
Time of Activity 1	19.69	18.79	21.31	21.63	17.73	18.84	18.69
Cost 1	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400
Net Present Cost 1	$ 46,887	$ 47,700	$ 45,452	$ 45,172	$ 48,680	$ 47,659	$ 47,791
Time of Activity 2	31.69	30.79	33.31	33.63	29.73	30.84	30.69
Cost 2	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400
Net Present Cost 2	$ 37,246	$ 37,892	$ 36,106	$ 35,884	$ 38,670	$ 37,859	$ 37,965
Time of Activity 3	43.69	42.79	45.31	45.63	41.73	42.84	42.69
Cost 3	$ 35,990	$ 41,097	$ 26,752	$ 24,918	$ 47,139	$ 40,841	$ 41,667
Net Present Cost 3	$ 15,568	$ 18,085	$ 11,218	$ 10,384	$ 21,170	$ 17,957	$ 18,371

Life Cycle Cost Percent Changes by Scenario, Using 1.9% Discount Rate

Scenario	1	2	3	4	5	6
Weighted Average % Change in LCC	+0.4	-2.4	-2.6	+1.8	+0.1	+0.1

Life Cycle Cost Analyses for Each Sample Pavement Section and Each Scenario
Part 2: Using 7.0% Discount Rate

Life Cycle Costs for:		Flexible	#1	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 10,263**	**$ 10,286**	**$ 10,178**	**$ 10,167**	**$ 11,082**	**$ 10,029**	**$ 10,234**
Time of Activity 1	31.98	31.96	32.06	32.07	31.22	32.21	32.01
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 8,482	$ 8,496	$ 8,432	$ 8,426	$ 8,963	$ 8,344	$ 8,465
Time of Activity 2	43.98	43.96	44.06	44.07	43.22	44.21	44.01
Cost 2	$ 43,324	$ 43,488	$ 42,735	$ 42,662	$ 48,793	$ 41,688	$ 43,126
Net Present Cost 2	$ 1,780	$ 1,790	$ 1,746	$ 1,742	$ 2,119	$ 1,685	$ 1,769

Life Cycle Costs for:		Flexible	#1	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 18,453**	**$ 18,381**	**$ 18,185**	**$ 18,421**	**$ 18,665**	**$ 16,799**	**$ 18,412**
Time of Activity 1	26.07	26.11	26.22	26.09	25.94	27.03	26.09
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 13,029	$ 12,992	$ 12,888	$ 13,013	$ 13,149	$ 12,150	$ 13,008
Time of Activity 2	38.07	38.11	38.22	38.09	37.94	39.03	38.09
Cost 2	$ 85,910	$ 85,622	$ 84,830	$ 85,784	$ 86,400	$ 78,976	$ 85,748
Net Present Cost 2	$ 5,423	$ 5,389	$ 5,297	$ 5,408	$ 5,504	$ 4,649	$ 5,404
Time of Activity 3	50.07	50.11	50.22	50.09	49.94	51.03	50.09
Cost 3	$ -	$ -	$ -	$ -	$ 419	$ -	$ -
Net Present Cost 3	$ -	$ -	$ -	$ -	$ 11	$ -	$ -

Life Cycle Costs for:		Flexible	#1	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 5,530**	**$ 5,549**	**$ 5,484**	**$ 5,530**	**$ 5,549**	**$ 5,530**	**$ 5,534**
Time of Activity 1	37.20	37.17	37.28	37.20	37.17	37.20	37.19
Cost 1	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000
Net Present Cost 1	$ 5,380	$ 5,392	$ 5,349	$ 5,380	$ 5,392	$ 5,380	$ 5,382
Time of Activity 2	49.20	49.17	49.28	49.20	49.17	49.20	49.19
Cost 2	$ 5,356	$ 5,567	$ 4,822	$ 5,356	$ 5,567	$ 5,356	$ 5,400
Net Present Cost 2	$ 151	$ 157	$ 135	$ 151	$ 157	$ 151	$ 152

Life Cycle Costs for:		**Rigid**	**#1**	**HV**			
	Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 6,314.8**	**$ 6,328.4**	**$ 6,096.9**	**$ 6,054.8**	**$ 6,374.6**	**$ 6,096.9**	**$ 6,291.3**
Time of Activity 1	21.99	21.96	22.38	22.45	21.88	22.38	22.03
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 4,887	$ 4,896	$ 4,751	$ 4,724	$ 4,925	$ 4,751	$ 4,873
Time of Activity 2	36.99	36.96	37.38	37.45	36.88	37.38	37.03
Cost 2	$ 20,908	$ 20,947	$ 20,281	$ 20,158	$ 21,076	$ 20,281	$ 20,841
Net Present Cost 2	$ 1,428	$ 1,433	$ 1,346	$ 1,330	$ 1,450	$ 1,346	$ 1,419

Life Cycle Costs for:		**Rigid**	**#1**	**MV**			
	Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 5,945**	**$ 5,954**	**$ 5,722**	**$ 5,671**	**$ 5,982**	**$ 5,722**	**$ 5,920**
Time of Activity 1	22.66	22.64	23.08	23.18	22.59	23.08	22.70
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 4,655	$ 4,661	$ 4,514	$ 4,482	$ 4,679	$ 4,514	$ 4,639
Time of Activity 2	37.66	37.64	38.08	38.18	37.59	38.08	37.70
Cost 2	$ 19,832	$ 19,859	$ 19,151	$ 18,991	$ 19,942	$ 19,151	$ 19,756
Net Present Cost 2	$ 1,290	$ 1,293	$ 1,208	$ 1,189	$ 1,304	$ 1,208	$ 1,280

Life Cycle Costs for:		**Rigid**	**#1**	**LV Artl**			
	Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 1,273**	**$ 1,275**	**$ 1,231**	**$ 1,222**	**$ 1,298**	**$ 1,260**	**$ 1,281**
Time of Activity 1	40.53	40.50	40.98	41.09	40.26	40.67	40.44
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 1,273	$ 1,275	$ 1,231	$ 1,222	$ 1,298	$ 1,260	$ 1,281

Life Cycle Costs for:		Flexible	#2	HV			
	Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 91,346**	**$ 91,480**	**$ 90,920**	**$ 90,951**	**$ 91,767**	**$ 90,779**	**$ 91,285**
Time of Activity 1	6.07	6.05	6.13	6.13	6.01	6.15	6.08
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 55,621	$ 55,699	$ 55,374	$ 55,392	$ 55,865	$ 55,293	$ 55,585
Time of Activity 2	18.07	18.05	18.13	18.13	18.01	18.15	18.08
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 23,283	$ 23,315	$ 23,180	$ 23,187	$ 23,385	$ 23,145	$ 23,268
Time of Activity 3	30.07	30.05	30.13	30.13	30.01	30.15	30.08
Cost 3	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 3	$ 9,746	$ 9,760	$ 9,703	$ 9,706	$ 9,789	$ 9,689	$ 9,740
Time of Activity 4	42.07	42.05	42.13	42.13	42.01	42.15	42.08
Cost 4	$ 57,104	$ 57,242	$ 56,663	$ 56,695	$ 57,538	$ 56,516	$ 57,040
Net Present Cost 4	$ 2,696	$ 2,707	$ 2,664	$ 2,666	$ 2,729	$ 2,653	$ 2,692

Life Cycle Costs for:		Flexible	#2	MV			
	Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 105,632**	**$ 105,701**	**$ 105,332**	**$ 105,379**	**$ 105,803**	**$ 105,193**	**$ 105,605**
Time of Activity 1	4.16	4.16	4.20	4.20	4.14	4.22	4.17
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 63,867	$ 63,907	$ 63,694	$ 63,721	$ 63,965	$ 63,614	$ 63,851
Time of Activity 2	16.16	16.16	16.20	16.20	16.14	16.22	16.17
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 26,735	$ 26,751	$ 26,662	$ 26,673	$ 26,776	$ 26,629	$ 26,728
Time of Activity 3	28.16	28.16	28.20	28.20	28.14	28.22	28.17
Cost 3	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 3	$ 11,191	$ 11,198	$ 11,161	$ 11,165	$ 11,208	$ 11,147	$ 11,188
Time of Activity 4	40.16	40.16	40.20	40.20	40.14	40.22	40.17
Cost 4	$ 70,819	$ 70,881	$ 70,551	$ 70,592	$ 70,972	$ 70,425	$ 70,795
Net Present Cost 4	$ 3,840	$ 3,845	$ 3,815	$ 3,819	$ 3,854	$ 3,803	$ 3,837

Life Cycle Costs for:		Flexible	#2	LV Artl			
	Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**
Failure Mode:	**Crack**	**Crack**	**Crack**	**Crack**	**Crack**	**Crack**	**Crack**
Crack	**$ 47,597**	**$ 47,960**	**$ 44,255**	**$ 44,273**	**$ 48,881**	**$ 47,752**	**$ 47,990**
Time of Activity 1	14.23	14.13	15.12	15.12	13.89	14.19	14.13
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 30,772	$ 30,981	$ 28,837	$ 28,847	$ 31,527	$ 30,861	$ 30,998
Time of Activity 2	26.23	26.13	27.12	27.12	25.89	26.19	26.13
Cost 2	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000
Net Present Cost 2	$ 11,927	$ 12,008	$ 11,177	$ 11,181	$ 12,220	$ 11,961	$ 12,015
Time of Activity 3	38.23	38.13	39.12	39.12	37.89	38.19	38.13
Cost 3	$ 78,493	$ 79,117	$ 72,527	$ 72,560	$ 80,000	$ 78,760	$ 79,167
Net Present Cost 3	$ 4,898	$ 4,971	$ 4,242	$ 4,245	$ 5,115	$ 4,929	$ 4,977
Time of Activity 4	50.23	50.13	51.12	51.12	49.89	50.19	50.13
Cost 4	$ -	$ -	$ -	$ -	$ 721	$ -	$ -
Net Present Cost 4	$ -	$ -	$ -	$ -	$ 277	$ -	$ -

Life Cycle Costs for:	Rigid	#2	HV				
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Fault**	**Fault**	**Fault**	**Fault**	**Fault**	**Fault**	**Fault**
Fault	**$ 982**	**$ 986**	**$ 895**	**$ 879**	**$ 1,003**	**$ 907**	**$ 974**
Time of Activity 1	44.10	44.04	45.38	45.63	43.80	45.19	44.22
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 982	$ 986	$ 895	$ 879	$ 1,003	$ 907	$ 974

Life Cycle Costs for:	Rigid	#2	MV				
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Fault**	**Fault**	**Fault**	**Fault**	**Fault**	**Fault**	**Fault**
Fault	**$ 3,832**	**$ 3,838**	**$ 3,518**	**$ 3,454**	**$ 3,951**	**$ 3,572**	**$ 3,816**
Time of Activity 1	27.49	27.47	28.42	28.62	27.15	28.25	27.53
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 3,279	$ 3,283	$ 3,065	$ 3,021	$ 3,359	$ 3,102	$ 3,268
Time of Activity 2	42.49	42.47	43.42	43.62	42.15	43.25	42.53
Cost 2	$ 12,071	$ 12,101	$ 10,580	$ 10,257	$ 12,609	$ 10,845	$ 11,999
Net Present Cost 2	$ 553	$ 555	$ 453	$ 433	$ 592	$ 470	$ 548

Life Cycle Costs for:	Rigid	#2	LV Artl				
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Fault**	**Fault**	**Fault**	**Fault**	**Fault**	**Fault**	**Fault**
Fault	**$ 2,655**	**$ 2,651**	**$ 2,491**	**$ 2,442**	**$ 2,765**	**$ 2,635**	**$ 2,713**
Time of Activity 1	31.45	31.47	32.13	32.34	31.02	31.53	31.22
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 2,459	$ 2,456	$ 2,340	$ 2,305	$ 2,538	$ 2,445	$ 2,501
Time of Activity 2	46.45	46.47	47.13	47.34	46.02	46.53	46.22
Cost 2	$ 5,700	$ 5,675	$ 4,605	$ 4,267	$ 6,401	$ 5,572	$ 6,076
Net Present Cost 2	$ 196	$ 195	$ 151	$ 137	$ 227	$ 190	$ 212

Life Cycle Costs for:	Flexible	#3	HV				
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 42,515**	**$ 45,029**	**$ 41,956**	**$ 42,018**	**$ 45,118**	**$ 42,562**	**$ 42,370**
Time of Activity 1	15.95	15.25	16.11	16.09	15.22	15.93	15.99
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 27,160	$ 28,578	$ 26,843	$ 26,878	$ 28,628	$ 27,186	$ 27,077
Time of Activity 2	27.95	27.25	28.11	28.09	27.22	27.93	27.99
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 11,369	$ 11,963	$ 11,236	$ 11,251	$ 11,984	$ 11,380	$ 11,334
Time of Activity 3	39.95	39.25	40.11	40.09	39.22	39.93	39.99
Cost 3	$ 72,384	$ 77,435	$ 71,220	$ 71,350	$ 77,609	$ 72,480	$ 72,082
Net Present Cost 3	$ 3,987	$ 4,488	$ 3,877	$ 3,889	$ 4,506	$ 3,996	$ 3,958

Life Cycle Costs for:	Flexible	#3	MV				
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 19,828**	**$ 18,035**	**$ 18,417**	**$ 18,503**	**$ 19,941**	**$ 19,838**	**$ 16,769**
Time of Activity 1	25.21	26.30	26.09	26.04	25.14	25.21	27.05
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 13,865	$ 12,809	$ 13,010	$ 13,056	$ 13,934	$ 13,871	$ 12,134
Time of Activity 2	37.21	38.30	38.09	38.04	37.14	37.21	39.05
Cost 2	$ 86,400	$ 84,216	$ 85,765	$ 86,112	$ 86,400	$ 86,400	$ 78,846
Net Present Cost 2	$ 5,804	$ 5,226	$ 5,406	$ 5,447	$ 5,833	$ 5,806	$ 4,635
Time of Activity 3	49.21	50.30	50.09	50.04	49.14	49.21	51.05
Cost 3	$ 5,674	$ -	$ -	$ -	$ 6,170	$ 5,717	$ -
Net Present Cost 3	$ 160	$ -	$ -	$ -	$ 174	$ 161	$ -

Life Cycle Costs for:	Flexible	#3	LV Artl				
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 3,870**	**$ 3,873**	**$ 3,858**	**$ 3,866**	**$ 3,873**	**$ 3,873**	**$ 3,873**
Time of Activity 1	41.73	41.72	41.78	41.75	41.72	41.72	41.72
Cost 1	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000
Net Present Cost 1	$ 3,870	$ 3,873	$ 3,858	$ 3,866	$ 3,873	$ 3,873	$ 3,873

Life Cycle Costs for:		Rigid	#3	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 4,711**	**$ 4,724**	**$ 4,401**	**$ 4,336**	**$ 4,788**	**$ 4,722**	**$ 4,687**
Time of Activity 1	25.23	25.20	25.97	26.14	25.05	25.20	25.28
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 3,863	$ 3,872	$ 3,659	$ 3,616	$ 3,914	$ 3,871	$ 3,848
Time of Activity 2	40.23	40.20	40.97	41.14	40.05	40.20	40.28
Cost 2	$ 15,703	$ 15,751	$ 14,502	$ 14,241	$ 15,990	$ 15,744	$ 15,615
Net Present Cost 2	$ 848	$ 852	$ 741	$ 720	$ 874	$ 851	$ 839

Life Cycle Costs for:		Rigid	#3	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**	**IRI**
IRI	**$ 2,191**	**$ 2,194**	**$ 2,007**	**$ 1,960**	**$ 2,207**	**$ 2,186**	**$ 2,176**
Time of Activity 1	33.50	33.48	34.43	34.68	33.42	33.52	33.57
Cost 1	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100	$ 24,100
Net Present Cost 1	$ 2,119	$ 2,122	$ 1,982	$ 1,946	$ 2,131	$ 2,116	$ 2,109
Time of Activity 2	48.50	48.48	49.43	49.68	48.42	48.52	48.57
Cost 2	$ 2,412	$ 2,438	$ 923	$ 519	$ 2,534	$ 2,371	$ 2,298
Net Present Cost 2	$ 71	$ 72	$ 26	$ 14	$ 75	$ 70	$ 68

Life Cycle Costs for:		Rigid	#3	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**None**	**None**	**None**	**None**	**None**	**None**	**None**
	$ -	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**
Time of Activity 1	> 50	> 50	> 50	> 50	> 50	> 50	> 50

Life Cycle Costs for:	Flexible	#4	HV				
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 77,738**	**$ 77,922**	**$ 77,247**	**$ 77,692**	**$ 78,053**	**$ 77,483**	**$ 77,677**
Time of Activity 1	8.18	8.15	8.27	8.19	8.13	8.23	8.19
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 47,710	$ 47,817	$ 47,424	$ 47,684	$ 47,894	$ 47,561	$ 47,675
Time of Activity 2	20.18	20.15	20.27	20.19	20.13	20.23	20.19
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 19,971	$ 20,016	$ 19,851	$ 19,960	$ 20,048	$ 19,909	$ 19,956
Time of Activity 3	32.18	32.15	32.27	32.19	32.13	32.23	32.19
Cost 3	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 3	$ 8,360	$ 8,379	$ 8,310	$ 8,355	$ 8,392	$ 8,334	$ 8,354
Time of Activity 4	44.18	44.15	44.27	44.19	44.13	44.23	44.19
Cost 4	$ 41,883	$ 42,106	$ 41,285	$ 41,827	$ 42,264	$ 41,573	$ 41,809
Net Present Cost 4	$ 1,696	$ 1,709	$ 1,662	$ 1,693	$ 1,718	$ 1,679	$ 1,692

Life Cycle Costs for:	Flexible	#4	MV				
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**	**Rut**
Rut	**$ 98,663**	**$ 98,788**	**$ 98,445**	**$ 98,648**	**$ 98,847**	**$ 98,529**	**$ 98,629**
Time of Activity 1	5.06	5.04	5.09	5.06	5.03	5.08	5.06
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 59,851	$ 59,924	$ 59,725	$ 59,843	$ 59,957	$ 59,774	$ 59,832
Time of Activity 2	17.06	17.04	17.09	17.06	17.03	17.08	17.06
Cost 2	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 2	$ 25,053	$ 25,084	$ 25,001	$ 25,050	$ 25,098	$ 25,021	$ 25,045
Time of Activity 3	29.06	29.04	29.09	29.06	29.03	29.08	29.06
Cost 3	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 3	$ 10,487	$ 10,500	$ 10,465	$ 10,486	$ 10,506	$ 10,474	$ 10,484
Time of Activity 4	41.06	41.04	41.09	41.06	41.03	41.08	41.06
Cost 4	$ 64,376	$ 64,496	$ 64,167	$ 64,362	$ 64,552	$ 64,248	$ 64,344
Net Present Cost 4	$ 3,271	$ 3,281	$ 3,253	$ 3,270	$ 3,286	$ 3,260	$ 3,268

Life Cycle Costs for:	Flexible	#4	LV Artl				
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	**Crack**	**Crack**	**Crack**	**Crack**	**Crack**	**Crack**	**Crack**
Crack	**$ 25,720**	**$ 26,897**	**$ 22,748**	**$ 22,887**	**$ 28,480**	**$ 26,742**	**$ 26,932**
Time of Activity 1	21.77	21.22	23.27	23.19	20.52	21.29	21.21
Cost 1	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400	$ 86,400
Net Present Cost 1	$ 17,802	$ 18,522	$ 15,968	$ 16,055	$ 19,485	$ 18,427	$ 18,543
Time of Activity 2	33.77	33.22	35.27	35.19	32.52	33.29	33.21
Cost 2	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000	$ 80,000
Net Present Cost 2	$ 6,900	$ 7,179	$ 6,189	$ 6,223	$ 7,552	$ 7,142	$ 7,187
Time of Activity 3	45.77	45.22	47.27	47.19	44.52	45.29	45.21
Cost 3	$ 28,215	$ 31,859	$ 18,232	$ 18,728	$ 36,514	$ 31,388	$ 31,963
Net Present Cost 3	$ 1,019	$ 1,197	$ 590	$ 610	$ 1,443	$ 1,173	$ 1,202

Life Cycle Costs for:		Rigid	#4	HV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	None	None	None	None	None	None	None
	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Time of Activity 1	> 50	> 50	> 50	> 50	> 50	> 50	> 50

Life Cycle Costs for:		Rigid	#4	MV			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	None	None	None	None	None	None	None
	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Time of Activity 1	> 50	> 50	> 50	> 50	> 50	> 50	> 50

Life Cycle Costs for:		Rigid	#4	LV Artl			
	Base	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
Failure Mode:	Crack	Crack	Crack	Crack	Crack	Crack	Crack
Crack	$ 24,764	$ 26,656	$ 21,671	$ 21,104	$ 29,080	$ 26,558	$ 26,876
Time of Activity 1	19.69	18.79	21.31	21.63	17.73	18.84	18.69
Cost 1	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400
Net Present Cost 1	$ 16,391	$ 17,492	$ 14,572	$ 14,236	$ 18,891	$ 17,435	$ 17,620
Time of Activity 2	31.69	30.79	33.31	33.63	29.73	30.84	30.69
Cost 2	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400	$ 68,400
Net Present Cost 2	$ 6,861	$ 7,322	$ 6,100	$ 5,959	$ 7,908	$ 7,298	$ 7,376
Time of Activity 3	43.69	42.79	45.31	45.63	41.73	42.84	42.69
Cost 3	$ 35,990	$ 41,097	$ 26,752	$ 24,918	$ 47,139	$ 40,841	$ 41,667
Net Present Cost 3	$ 1,511	$ 1,842	$ 999	$ 909	$ 2,281	$ 1,824	$ 1,881

Life Cycle Cost Percent Changes by Scenario, Using 7.0% Discount Rate

Scenario	1	2	3	4	5	6
Weighted Average % Change in LCC	+0.7	-4.2	-4.1	+2.7	+0.2	+0.2

APPENDIX O – IMPACTS OF OVERWEIGHT AXLES ON INITIAL PAVEMENT SERVICE INTERVALS

The tables that follow tabulate the number of years until predicted distress levels reach a specified trigger value (initial service interval) for the base case and under a hypothetical traffic loading that removes all overweight axles.

Medium Volume Interstate Flexible Sections With and Without Overweight Axles

Rehab Trigger at Predicted		Geographic Location #1		Geographic Location #2	
Mean Values, Raw		All Traffic	No OW Axles	All Traffic	No OW Axles
IRI @	160	34.250	34.890	15.170	15.296
Rutting	0.4	26.068	31.092	4.164	5.189
Fatigue Crk @	7.5%	30.853	37.835	7.107	9.201
Change (years)		-	5.024	-	1.025
Change (pct)		-	19.3%	-	24.6%

Medium Volume Interstate Rigid Sections With and Without Overweight Axles

Rehab Trigger at Predicted		Geographic Location #1		Geographic Location #2	
Mean Values, Raw		All Traffic	No OW Axles	All Traffic	No OW Axles
IRI @	160	22.657	23.976	29.317	32.396
Faulting @	0.15	28.553	31.398	27.487	30.424
Transv Crk @	7.5%	> 50	> 50	> 50	> 50
Change (years)		-	1.320	-	2.938
Change (pct)		-	5.8%	-	10.7%

Medium Volume Interstate Flexible Sections With and Without Overweight Axles

Rehab Trigger at Predicted		Geographic Location #3		Geographic Location #4	
Mean Values, Raw		All Traffic	No OW Axles	All Traffic	No OW Axles
IRI @	160	38.277	39.223	35.915	36.360
Rutting	0.4	25.212	33.128	5.059	6.398
Fatigue Crk @	7.5%	> 50	> 50	13.544	18.058
Change (years)		-	7.916	-	1.339
Change (pct)		-	31.4%	-	26.5%

Medium Volume Interstate Rigid Sections With and Without Overweight Axles

Rehab Trigger at Predicted		Geographic Location #3		Geographic Location #4	
Mean Values, Raw		All Traffic	No OW Axles	All Traffic	No OW Axles
IRI @	160	33.499	36.676	> 50	> 50
Faulting @	0.15	43.875	49.790	> 50	> 50
Transv Crk @	7.5%	> 50	> 50	> 50	> 50
Change (years)		-	3.177	-	0.000
Change (pct)		-	9.5%	-	0.0%