Structural Impacts of Super Heavy Load (SHL) Vehicles on Transportation Infrastructure

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STRUCTURAL IMPACTS OF SUPER HEAVY LOAD (SHL) VEHICLES ON TRANSPORTATION INFRASTRUCTURE

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Dedication

To my parents, and

my beloved one, Mahsa,

for all their love, endless support, and encouragement.
STRUCTURAL IMPACTS OF SUPER HEAVY LOAD (SHL) VEHICLES ON TRANSPORTATION INFRASTRUCTURE

by

ALI MOROVATDAR, MSCE

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Abstract

Operation of non-conventional Super Heavy Load (SHL) vehicles is an ongoing challenge for transportation stakeholders and the traveling public across the nation. Despite facilitating the movement of heavy, large, and non-divisible loads, passages of such vehicles with complex loading configurations that typically weigh several folds of the permissible weight limits set forth by regulatory agencies have been adversely affecting the structural integrity of pavement facilities. This translates into accelerated damage of pavement structures, which in turn poses safety concerns for users of the transportation facilities. Accurate quantification of such detrimental impacts is the precursor to preserve the existing transportation facilities. Having an accurate account of SHL effects is also the key step for further incorporation of these non-generic loads into the mechanistic-empirical (ME) pavement design systems. Current procedures commonly deployed by state highway agencies for evaluation of SHLs are primarily empirical in nature and need major overhaul to accommodate taxing loading conditions associated with SHLs. Furthermore, there is a knowledge gap in mechanistic quantification of the damages imparted on pavements by super heavy trucks. Therefore, this study was designed to bridge this gap by identifying the distresses and damage mechanisms pertinent to SHLs.

The primary goal of this study was to mechanistically quantify the damages and structural impacts imparted on transportation facilities due to SHL applications. The secondary goal of the research was to synthesize the results to provide further insights on necessary adjustments to upgrade the current ME pavement design protocols for accommodating SHLs. To achieve the research objectives, initially, two comprehensive databases of SHLs in demanding corridors in Texas were developed based on (1) the calibrated axle load spectra data collected by P-WIM units in ten representative sites, and (2) the most recent permit records issued by TxDMV Motor Carrier Division in both Eagle Ford and Permian Basin regions. The research team then developed a methodology for clustering unconventional trucks carrying super heavy loads, and further developed an algorithm for the determination of the number of influencing tires extrapolated outside of the nucleus axle of the multi-axle trailers. Along with the traffic characterization efforts, a series of non-destructive tests such as GPR and FWD was conducted in the field to better understand the structural capacity of pavement sections. Subsequently, the relevant information on traffic loading conditions and site-specific pavement material properties were in turn
incorporated in a series of 3D finite element modeling for advanced modeling of moving SHLs to accurately calculate the induced pavement responses under extremely heavy loads.

Ultimately, a methodologically sound and robust protocol was devised for the mechanistic characterization of the SHL structural impacts on transportation infrastructure. The devised multi-level approach consists of the following analysis procedures: (1) quantification of pavement damage, (2) prediction of pavement service life, (3) characterization of loss of pavement life, (4) analysis of slow-moving nature of SHLs, (5) analysis of acceleration/deceleration forces, (6) analysis of roadway geometric features, (7) stability analysis of sloped pavement shoulders, and (8) buried utility risk assessment. The multi-tier framework developed in this study consisted of provisions to account for the non-generic nature of super heavy trucks, unique features of the transportation network, site-specific axle load spectra, and climatic factors for realistic assessment of the SHL effects.

The numerical simulation results, cross validated by field visual observations and distress records, underscored the significance of SHL vehicles and their role to jeopardize the longevity, structural integrity, and stability of pavement facilities in overload corridors with large volume of heavy truck traffic operations. The analysis of pavement life reduction indicated that operation of SHLs can impart substantial loss of service life as high as 55% for Farm-to-Market (FM) roadways, 33% for State Highways (SH), and 25% for US highways. Furthermore, the results showed that the inclusion of “slow-moving nature” and “acceleration/deceleration” of SHLs in the analysis resulted in substantially higher pavement life consumption. It was also found that the accumulated damage under SHLs was more pronounced at horizontal curves with high super-elevation rates. Accordingly, analysis and design protocols of pavements servicing the overload corridors should also manifest such sensitivity to the vehicle speed, SHL dynamics, and roadway geometric features for accurate assessment of the pavement damage and loss of service life.

Realistic simulation of tire-pavement interactions is the prelude for proper quantification of the damages imparted by SHLs. The numerical analyses revealed that overlooking the variations of tire-pavement contact stresses during vehicle maneuvering at roadway curved segments or vehicle speeding/braking scenarios at intersections leads to significant underestimation of damages imparted by super heavy loads. Consequently, case-specific contact stresses with non-uniform distribution patterns, in lieu of an all-purpose uniformly distributed load, better represent the taxing stress paths imposed by SHL trailer units.
A comprehensive sensitivity analysis showed that increasing the wheel load from 6 kips to 10 kips leads to drastic increase in the imparted damages on pavements by 5-7 times. The results also indicated that operation of SHLs adversely affected FM roads with less robust pavement profile, more than evaluated SH and US highways. The detrimental impact of SHLs was also more pronounced when combined with poor moisture management capabilities under demanding environmental scenarios such as flooding conditions. Similarly, the damages imposed by SHLs during hot summer months were found to be appreciably higher, as compared to cold winter months. Consequently, damage quantification mechanisms for pavement facilities accommodating SHLs should properly account for the synergistic influence of key components such as wheel load magnitude, SHL axle assembly, pavement profile, and climatic conditions for accurate assessment of the pavement distresses.

Another noteworthy finding of this study pertains to the potential failure risk of roadway shoulders subjected to SHLs. Based on the parametric analysis results, unpaved shoulders with narrow widths and steep slopes were sensitive to the demanding loading conditions imposed by wide multi-axle trailers. The probabilistic slope stability analysis devised in this study indicated that considerations of the complex and unique nature of SHLs, inherent aleatory variability of influencing parameters, and incorporation of epistemic uncertainties in the slope stability analyses can protect pavement shoulders from failure under SHL movement.

The synthesized results of this research can provide insights to improve current protocols for the analysis and design of pavements subjected to taxing loading conditions imposed by SHLs. Additionally, the research findings and risk management heat maps provided in this study facilitate the characterization of the detrimental impacts of non-conventional SHL units on transportation infrastructures in OW networks. The developed color-coded maps can be further instrumental for stakeholders/state DOTs to have a mechanistic means for the approval (or rejection) of SHL permits across overload corridors. The best practice recommendations presented in this study can also guide highway agencies to make provisions of safe traffic passage, and to adopt proper M&R strategies to preserve the transportation infrastructure facilities against expedited deterioration.

Keywords: Pavement Design, Pavement Structural Analysis, Super Heavy Load (SHL) Vehicles, Finite Element Analysis, Non-Destructive Testing (NDT), Portable WIM, Overload Corridors.
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Chapter 1: Introduction

Super Heavy Loads (SHLs) are heavy vehicles that exceed the thresholds set for Over-Weight (OW) vehicles. In Texas, any vehicle with Gross Vehicle Weight (GVW) above the 80,000 lb limit is categorized as an OW truck. Trucks with GVW exceeding 254,300 lb are referred to as SHLs (TxDOT Pavement Manual, 2019). Another definition based on the Texas Department of Transportation (TxDOT) Pavement Manual provides an alternative definition of SHLs. Based on the clarification provided in Chapter 13, Section 5, trucks with GVW over 200,000 lb, with less than 95 ft of axle spacing, or trucks that exceed the maximum permissible weight on any axle or axle group, could be also considered as SHLs (TxDOT Pavement Manual, 2019).

Recent traffic trends and permit issuance records indicate significant demands for the operation of the SHL vehicles in overload corridors of several states across the nation (Ashtiani et al., 2019). In Texas, records of the SHL permits issued by Motor Carrier Division indicated more than 8,000 permit issuance in the Eagle Ford Shale and Permian Basin regions during the 2017-2020 period. SHL movement is attributed to a variety of economic activities such as energy production, freight, and transportation of heavy machinery and equipment such as well servicing units, power transformers, and electric generators. Despite facilitating the movement of heavy, large, and non-divisible loads, the SHL vehicles weigh several times more than the permissible truck size/weight limits in Texas. Figure 1.1 illustrates an example of a trailer-mounted SHL unit that was allowed to operate in a specified route in San Antonio, Texas, in November 2018 to transport a generator stator. The demonstrated vehicle comprised of 45 axles and GVW exceeding 1.8 million lb. As indicated in Figure 1.1, the load magnitudes on each axle and tire in the specialized trailer units were 45.5 kips and 5.7 kips, respectively.
Passages of such non-conventional vehicles with heavy tires and complex axle arrangements can potentially affect the structural integrity of highway pavements, leading to premature failure and loss of service life of the impacted pavements. These detrimental impacts are more pronounced in Farm-to-Market (FM) roadways that were never designed to withstand the non-conventional loading conditions attributed to SHL vehicles. Due to taxing loading conditions, even one pass of SHL vehicle can consume the entire remaining service life of these pavements. In terms of the truck size-related concerns, it is also noted that the majority of the FM and State highways lack appropriate lane width to accommodate SHL vehicles. Thus, passages of wide trailers with multi-axle and multi-wheel configurations can potentially jeopardize the stability of pavement shoulders. This translates into an accelerated damage of pavement structures, which in turn pose safety concerns for the traveling public. Consequently, it is imperative to accurately quantify the structural impacts of these non-conventional SHLs on pavement facilities.
1.2. **Problem Statement**

There are several concerns associated with the detrimental impacts of the SHL vehicles operating in the OW corridors. In addition to the pavement life reduction, SHL applications can cause localized shear failure, particularly in the subgrade. Stability of the sloped pavement shoulders under the movement of the wide SHL units is another major concern that needs to be properly assessed. Moreover, these heavy vehicles pose a potential failure risk to the existing buried utilities along the SHL vehicles’ route. Consequently, the analysis of the SHL structural impacts should properly account for these nationwide challenges. The highlighted concerns are even more pronounced considering the slow-moving nature of the SHLs, acceleration/deceleration forces, turning movements at the bends, poor moisture management capabilities, and elevated temperatures in the summer season.

Considering the highlighted issues and concerns, coupled with the absence of a practical procedure, analysis and design protocols of pavements servicing the overload corridors need to be accordingly updated to account for the demanding loading conditions induced by SHL vehicles. Essentially, mechanistic characterization of SHL impacts on structural capacity, service life, and the pavement stability is the key step for further incorporation of the SHLs into the mechanistic-empirical (ME) pavement design systems. The results can be also instrumental in providing a mechanistic means for the approval (or rejection) of SHL permits in overload corridors across the nation. Therefore, there is a pressing need to properly assess the structural impacts of SHLs to preserve the existing transportation network, facilitate the SHL permit evaluation procedure, and to properly plan for future pavement design.
1.3. **Purpose of Study**

The primary goal of this study was to mechanistically quantify the damages and structural impacts imparted on transportation facilities due to SHL applications, using field-derived information and advanced numerical modeling techniques. The secondary goal of the research was to synthesize the results to provide further insights on necessary adjustments to upgrade the current ME pavement design protocols, considering the complex nature of SHLs. The following items will be considered to satisfy the objectives of the study:

- Evaluation of major concerns associated with the operation of SHL vehicles in pavement facilities.
- Development of a mechanistic approach to analyze the moving non-conventional SHLs with multi-axle trailers and numerous tires.
- Development of robust and mechanistic-based approaches to quantify the damages and the loss of pavement service life imparted by SHL vehicle movements.
- Analysis of the structural impacts associated with SHL vehicles, by deploying relevant numerical modeling techniques such as the finite element method.
- Evaluation of major factors contributing to the damages imparted on the pavement facilities due to SHL vehicle applications.
- Providing best practice recommendations to mitigate the detrimental impacts pertaining to SHL vehicle operation.

1.4. **Outline of Dissertation**

The general organization of this dissertation is provided in this section. Subsequent to the introductory chapter detailing the problem statement and research objectives, the following information was presented in succinct yet detailed manner in this study:
• Chapter 2: literature review on current methodologies to assess the SHL effects.
• Chapter 3: analysis of the current network prior to the field trials.
• Chapter 4: field-testing and evaluation of pavement sections.
• Chapter 5: deployment of the Portable Weigh-In-Motion (P-WIM) devices, calibration procedure and algorithms, and the development of axle load spectra for several representative sites.
• Chapter 6: development of comprehensive databases pertinent to SHLs, and classification of SHLs with most demanding loading scenarios.
• Chapter 7: development of a mechanistic-based approach for the analysis of specialized SHL units with multi-axle trailers.
• Chapter 8: 3D finite element analysis of moving SHL vehicles.
• Chapter 9: calculation of new site-specific axle load equivalency factors considering the unique nature of SHLs, environmental factors, and type of transportation facility.
• Chapter 10: analysis of the remaining life of pavement sections in the OW corridors.
• Chapter 11: utilizing the field-derived databases, as well as advanced numerical techniques, to mechanistically characterize the structural impacts of SHLs on transportation infrastructure facilities through an all-encompassing analysis protocol.
• Chapter 12: summary of the research, conclusions, and list of major findings, as well as recommendations for future research initiatives.
Chapter 2: Literature Review

2.1. Introduction

The primary objective of this chapter is to provide a review summary of the current practices for permitting process and assessment of the impacts of SHLs on the transportation infrastructures. A brief synthesis of the previous research studies on the evaluation of the structural impacts and the damages imparted on the pavement facilities due to the SHL truck operations follows the preceding segment of this chapter.

2.1.1. Organization of the Chapter

This chapter consists of six sections. Subsequent to the introductory section, Section 2 provides information associated with the vehicle characteristics and thresholds used to define the SHL trucks. Section 3 discusses the recent trends in energy development and freight movement activities across the U.S. and more importantly across Texas. Section 4 illustrates different types of pavement damage attributed to the SHL vehicles. Section 5 provides a review of SHL permitting practices in the United States and Texas. Section 6 provides discussions on the previous research studies relevant to the SHL truck operations and the evaluation of their impacts on the pavement structures. Ultimately, the main conclusions obtained through the review of the preceding literature are also presented in Section 6.
2.2. **DEFINITION OF SUPER HEAVY LOAD VEHICLES**

2.2.1. **Metrics Used for SHL Identification in United States**

Super heavy loads are heavy trucks that exceed the thresholds set for OW vehicles. As discussed earlier in the introduction chapter, in Texas, any trucks with GVW above the 80,000 lb. limit is categorized as OW trucks. However, SHLs are referred to as trucks with GVW exceeding 254,300 lb. (TxDOT Pavement Manual, 2019). Evidently, different states consider different weight thresholds for the definition of the SHLs, ranging from 120,000 to 254,300 lb. (Papagiannakis, 2015). Figure 2.1 shows the geographic distribution of these GVW limits set forth by different state Department of Transportations (DOTs) across the nation. As evidenced in the plot, Texas is one of the States with substantially higher threshold value defined for permitting SHLs compared to the other states across the US.

![Figure 2.1: GVW Limits for Definition of Super Heavy Loads in the United States (Papagiannakis, 2015).](image)

Another definition based on the TxDOT pavement manual in Chapter 13 provides an alternative definition of super heavy loads. Based on the clarification provided in Chapter 13
section 5, trucks with GVW over 200,000 lb. with less than 95 ft. of axle spacing, or trucks that exceed the maximum permissible weight on any axle or axle group, could be also considered as SHLs (TxDOT pavement manual, 2019). Hence, based on current state of practice, the GVW value is not the sole contributing factor included in the definition of the SHL vehicles. A relevant research effort conducted by Papagiannakis (2015) indicated that 41% of the total 39 studied states in the US define SHLs based on the maximum GVW alone, as shown in Figure 2.2. The authors reported that the majority of the states consider GVW of 200 kips as the primary metric to define SHLs. However, some questionnaire respondents stated factors other than GVW, such as axle spacing, axle loads, number of axles, truck size, etc., for the classification of SHLs. Table 2.1 provides a summary of different metrics used for the identification of the SHL vehicles by different state highway agencies in the US.

![Pie Chart](image)

Figure 2.2: Vehicle Characteristics used to Define SHLs in the United States (Papagiannakis, 2015).
Table 2.1: Summary of Metrics Used for SHL Identification in United States

<table>
<thead>
<tr>
<th>State</th>
<th>Metrics Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alaska, Kansas, New Jersey, North Carolina, Utah</td>
<td>GVW, Axle Load</td>
</tr>
<tr>
<td>Texas, Illinois, Louisiana, Wyoming</td>
<td>GVW, Axle Load, Axle Spacing</td>
</tr>
<tr>
<td>Colorado, Pennsylvania, Virginia</td>
<td>GVW, Vehicle Size</td>
</tr>
<tr>
<td>North Dakota</td>
<td>Axle Load, Number of tires per axle</td>
</tr>
</tbody>
</table>

2.2.2. Axle Configurations of Super Heavy Loads

Typically, SHL trucks consist of complex arrangement of multiple axles and tires compared to standard trucks. Figure 2.3 illustrates an example of axle configurations of an SHL vehicle that is allowed to operate in a specified route in Channelview, Texas, in October 2019. The demonstrated trailer comprised of 28 axles with 8 tires per axle and GVW that exceeds 1.2 million kips.

Table 2 provides summary information on the GVW, axle load, and axle configurations of SHL vehicles based on the compilation of permits issued by different State DOTs. As indicated in Table 2.2, the axle weights for the SHL trucks has a wide range from 25 to 131 kips. Similarly, the load magnitude on each tire ranges from 3.5 kips to 16.3 kips. As evidenced from tabulated permit data, Texas and Louisiana SHLs ranked among the highest in terms of load on tire. Based on the issued permits records, the reported SHLs consisted of complex axle and tire configurations; hence, it deems necessary to account for the non-generic nature of the loading conditions of these heavy vehicles for analysis of pavement structures subjected to SHLs.
Figure 2.3: Example of an Operated SHL in Texas with a GVW above 1.2 Million kips (from Project Technical Team, February 2020).

Table 2.2: Examples for SHL Vehicles’ Axle and Tire Configurations in different States (Synthesis of Ashtiani et al., 2019, Oh and Wimsatt, 2010, and Hajj et al., 2018)

<table>
<thead>
<tr>
<th>SHL-Vehicle Information</th>
<th>Texas</th>
<th>Arizona</th>
<th>Louisiana</th>
<th>Nevada</th>
<th>New York</th>
</tr>
</thead>
<tbody>
<tr>
<td>GVW (lb.)</td>
<td>254,300–2,550,000</td>
<td>647,855–1,180,000</td>
<td>402,240–3,660,551</td>
<td>250,041–6,215,938</td>
<td>200,000–855,000</td>
</tr>
<tr>
<td>Axle Weight (lb.)</td>
<td>30,000–120,000</td>
<td>46,305–51,687</td>
<td>25,639–130,734</td>
<td>18,000–75,000</td>
<td>28,300–52,600</td>
</tr>
<tr>
<td>Number of tires per axle</td>
<td>4 or 8</td>
<td>8</td>
<td>4, 8, or 12</td>
<td>4 or 8</td>
<td>4 or 8</td>
</tr>
<tr>
<td>Axle width</td>
<td>–</td>
<td>18 ft 4 in. to 20 ft 4 in.</td>
<td>17 ft 5 in. to 24 ft 7.3 in.</td>
<td>–</td>
<td>12 ft 10 in. to 13 ft 6 in.</td>
</tr>
<tr>
<td>Axle spacing (ft)</td>
<td>4 ft 2 in.</td>
<td>6 ft to 12 ft 1 in.</td>
<td>4 ft 7 in. to 11 ft 0.75 in.</td>
<td>–</td>
<td>4 ft 11 in. to 5 ft</td>
</tr>
<tr>
<td>Tire load (lb.)</td>
<td>5,000–15,000</td>
<td>5,000–6,460</td>
<td>7,028–16,341</td>
<td>2,580–11,500</td>
<td>3,538–6,575</td>
</tr>
<tr>
<td>Tire width (in.)</td>
<td>8 to 10 in.</td>
<td>8.25 to 11 in.</td>
<td>1 ft 0.5 in. to 1 ft 2 in.</td>
<td>–</td>
<td>1 ft 0.5 in. to 1 ft 2 in.</td>
</tr>
</tbody>
</table>
2.3. **Recent Trends on Energy Development and Freight Movement Activities**

In the past decades, several states have experienced drastic improvements in energy-related activities such as natural gas and crude oil production, as well as increased freight transportation due to improvements in the economy of States. Figure 2.4 shows the different shale plays across the continental United States and more importantly across Texas.

![Figure 2.4: United States Oil and Gas Shale Regions.](image)

Energy development operations have provided favorable economic benefits to the states and the nation. However, such activities have significantly increased the frequency of the heavy truck traffic movements in the network, resulting in expedited deterioration of ride quality and loss of pavement service life in the impacted zones. Damaged local and county roads have been a major source of inconvenience for the local residents in such states. A prime example of that is the unprecedented energy production activities during the past decade in highly active oil fields in the Permian Basin and the Eagle Ford Shale regions in Texas. The energy production activities on one hand, and the increase in freight transportation due to improvements in economic activities of the
state on the other, have resulted in unprecedented operations of OW and SHL trucks, which adversely impacted the pavement facilities in the impacted networks.

### 2.3.2. Texas Energy Sectors and Economic Impacts on State Highway Network

Advances in technology, particularly in crude oil extraction, natural gas production, wind energy farms, and other pertinent industries have yielded exponential growth in energy development. The development of these energy resources in the energy sector zones substantially contributes to the economy of the state as well as individual sectors. In Texas, it predominantly influences economically disadvantaged areas where it provides employment for thousands of Texans. In addition, such economic activities provide several forms of tax revenue due to the generation of sales taxes, hotel taxes, and severance taxes. Figure 2.5 illustrates a map of the major energy developing areas throughout Texas. It is estimated that between 12,000 to 24,000 oil and gas wells were permitted each year in Texas for the last decade (Railroad Commission of Texas, 2013).

![Figure 2.5: Texas Oil and Gas Well Permits from 2010 to 2015 (from TxDOT, 2016).](image)

The contributions of these energy development areas are significant, so several studies have been conducted to assess their socio-economic impacts in affected Districts. A 2014 study conducted by the Rawls College of Business at Texas Tech University examined the economic
impact of oil and gas industry in the Permian Basin (Ewing et al., 2014). The report indicated that in 2013, the Texas Portion of the Permian Basin’s oil and gas industry supported over 444,000 jobs, generated $113.6 billion in economic output and contributed with over $71.1 billion to the gross product of the state (Ewing et al., 2014). The detailed results of this study is tabulated in Table 2.3. Technological improvements such as horizontal drilling and hydraulic fracturing have led to drastic increases in productivity. The Permian Basin has seen such productivity increases that it has one of the greatest rig count of any basin or region in the world (Ewing et al., 2014).

Table 2.3: Economic Impact of the Permian Basin in 2013, in Millions of USD (Ewing et al., 2014)

<table>
<thead>
<tr>
<th>Impact Type</th>
<th>Employment</th>
<th>Labor Income</th>
<th>Total Value Added</th>
<th>Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direct Effect</td>
<td>0.19</td>
<td>15,706.91</td>
<td>40,086.45</td>
<td>77,880.00</td>
</tr>
<tr>
<td>Indirect Effect</td>
<td>0.13</td>
<td>6,238.98</td>
<td>11,405.09</td>
<td>21,103.16</td>
</tr>
<tr>
<td>Induced Effect</td>
<td>0.12</td>
<td>4,297.03</td>
<td>8,724.07</td>
<td>14,646.35</td>
</tr>
<tr>
<td>Total Effect</td>
<td>0.44</td>
<td>26,242.09</td>
<td>60,215.61</td>
<td>113,629.51</td>
</tr>
</tbody>
</table>

Note: Labor income, total value added and output are measured in current USD.

In another study conducted by the Center of Community and Business Research (CCBR) at The University of Texas at San Antonio, the authors concluded that the Eagle Ford Shale is the largest single oil and gas development in the world based on capital expenditures (Turnstall et al., 2013). CCBR focused their study on the impacts of the 15 most active energy producing counties and their 6 neighboring counties. This study concluded that the economic impact in 2013 for the 21-county area amounted to 155,000 jobs and $87 billion in total economic output (Turnstall et al., 2014). Total impacts for Eagle Ford Shale are summarized in Table 2.4. The Eagle Ford Shale paid nearly $5.6 billion in salaries and benefits to workers and provided over $2.2 billion for local and state governments (Turnstall et al., 2014). Additionally, the CCBR also made moderate predictions for the year 2023 in the 15-county area and the greater 21-county area that presented
staggering revenue figures (Table 2.5). The 21-county area was estimated to employ 196,660 people, provide over $4 billion for local and state governments, and generate over $137 billion in economic output (Turnstall et al., 2014).

Table 2.4: Total Impacts for Eagle Ford Shale 21-County Area in 2013 (Turnstall et al., 2014)

<table>
<thead>
<tr>
<th>Economic Impact</th>
<th>Direct</th>
<th>Indirect</th>
<th>Induced</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core 15-county area</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Output</td>
<td>$61,470,280,412</td>
<td>$7,941,100,117</td>
<td>$2,418,234,050</td>
<td><strong>$71,829,614,579</strong></td>
</tr>
<tr>
<td>Employment, full-time</td>
<td>42,607</td>
<td>52,333</td>
<td>19,375</td>
<td><strong>114,315</strong></td>
</tr>
<tr>
<td>Payroll</td>
<td>$2,027,428,721</td>
<td>$1,539,076,337</td>
<td>$584,718,872</td>
<td>$4,151,223,930</td>
</tr>
<tr>
<td>Gross regional product</td>
<td>$30,448,269,805</td>
<td>$4,333,962,004</td>
<td>$1,542,827,867</td>
<td>$36,325,059,676</td>
</tr>
<tr>
<td>Local government revenues</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>State revenue, including severance taxes</td>
<td>$2,025,968,804</td>
<td>$2,028,406,113</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Core and neighboring 21-county area

<table>
<thead>
<tr>
<th>Economic Impact</th>
<th>Direct</th>
<th>Indirect</th>
<th>Induced</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Output</td>
<td>$70,725,115,021</td>
<td>$12,896,817,708</td>
<td>$4,135,496,654</td>
<td><strong>$87,757,429,382</strong></td>
</tr>
<tr>
<td>Employment, full-time</td>
<td>51,652</td>
<td>71,648</td>
<td>31,684</td>
<td><strong>154,984</strong></td>
</tr>
<tr>
<td>Payroll</td>
<td>$2,707,017,870</td>
<td>$2,036,271,899</td>
<td>$896,394,413</td>
<td>$5,639,684,182</td>
</tr>
<tr>
<td>Gross regional product</td>
<td>$32,992,259,490</td>
<td>$7,199,851,186</td>
<td>$2,640,560,616</td>
<td>$42,832,671,293</td>
</tr>
<tr>
<td>Local government revenues</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>State revenue, including severance taxes</td>
<td>$2,218,877,342</td>
<td>$2,214,664,000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2.5: Total Impacts for Eagle Ford Shale 21-County Area in 2023 (Turnstall et al., 2014)

<table>
<thead>
<tr>
<th>Economic Impact</th>
<th>Direct</th>
<th>Indirect</th>
<th>Induced</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core 15-county area</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Output</td>
<td>$90,168,322,826</td>
<td>$10,893,464,660</td>
<td>$5,332,379,266</td>
<td><strong>$106,394,056,752</strong></td>
</tr>
<tr>
<td>Employment, full-time</td>
<td>36,785</td>
<td>71,309</td>
<td>42,699</td>
<td><strong>150,793</strong></td>
</tr>
<tr>
<td>Payroll</td>
<td>$6,311,816,751</td>
<td>$2,035,342,931</td>
<td>$1,289,319,720</td>
<td>$9,636,479,402</td>
</tr>
<tr>
<td>Gross regional product</td>
<td>$52,608,595,765</td>
<td>$5,805,086,021</td>
<td>$3,402,243,230</td>
<td>$61,815,925,016</td>
</tr>
<tr>
<td>Local government revenues</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>State revenue, including severance taxes</td>
<td>$3,741,688,868</td>
<td>$3,774,006,283</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Core and neighboring 21-county area

<table>
<thead>
<tr>
<th>Economic Impact</th>
<th>Direct</th>
<th>Indirect</th>
<th>Induced</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Output</td>
<td>$110,576,454,317</td>
<td>$19,636,284</td>
<td>$7,488,598,501</td>
<td><strong>$137,428,498,102</strong></td>
</tr>
<tr>
<td>Employment, full-time</td>
<td>38,767</td>
<td>99,786</td>
<td>58,107</td>
<td><strong>196,660</strong></td>
</tr>
<tr>
<td>Payroll</td>
<td>$6,718,204,896</td>
<td>$3,432,856,335</td>
<td>$1,927,647,160</td>
<td>$12,078,708,391</td>
</tr>
<tr>
<td>Gross regional product</td>
<td>$57,330,415,830</td>
<td>$10,686,840,880</td>
<td>$4,777,170,284</td>
<td>$72,794,426,994</td>
</tr>
<tr>
<td>Local government revenues</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>State revenue, including severance taxes</td>
<td>$4,073,239,614</td>
<td>$4,098,369,070</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.3.3. **Negative Impacts on Texas Highway Infrastructure**

Despite many positive economic impacts and benefits that energy companies generate; the
development of these energy resources has significantly affected the state and local transportation infrastructure. This is more pronounced during oil and gas fracking operations where there is a large volume of truck traffic in a short timeframe associated with the transportation of fresh water, chemicals, and sand to the sites as well as the transportation of the wastewater from the sites (Bierling et al., 2014). It is estimated that the volume of truck traffic to bring one gas well into production is equivalent to eight million cars and an additional two million cars per year to maintain one gas well (TxDOT, 2016).

In addition to the substantial truck traffic operations, the transportation of heavy construction equipment and drilling rig components has also contributed to damages that local and county roads experience. These local and county roads were not designed or built to accommodate such high volumes of truck traffic or heavy loads. Therefore, even a few passages that exceed loading conditions can result in substantial damage and reduced service life of transportation facilities in the region. Damaged roads and bridges are also a major source of inconvenience for energy companies, local residents, school buses, and emergency vehicles in Texas overload corridors.

In addition to the damaged roads and bridges, the maintenance costs on severely deteriorated Farm-to-Market (FM) roads has increased significantly from $500-$1,500 per mile prior to oil and gas developments to $35,000-$45,000 post-development (Epps and Newcomb, 2016). Local governments are expected to spend around $200 million per year for maintenance and rehabilitation. At the state level, it is anticipated that TxDOT will invest $500 million per year for safety, maintenance, and capacity need on the energy sector oil and gas impacted roadways (Epps and Newcomb, 2016).

According to the Center for Community and Business Research (CCBR), it is estimated
that roadways currently require $2 billion in total maintenance per year, $1 billion for damage to state highways and $1 billion for damages to local and county roads (Turnstall et al., 2013). However, many counties do not have the funding to address these damaged roads. Rebuilding a paved road can cost local agencies more than $1 million amounting to the total annual maintenance and construction budget that most of these departments can afford to allocate (TxDOT, 2016). Damaged roads also affect the energy development industry in the forms of equipment damage and lower operating speeds due to poor road conditions amounting to $1.5 to 3.5 billion (Epps et al., 2013). Consequently, the author concluded that taking precautionary measures to address roadway conditions is an urgent need in the impacted transportation networks that can potentially protect the transportation infrastructure facilities in a more cost-effective manner (Thach et al., 2021g).

2.3.4. Safety Concerns

Texas has become one of the deadliest states in total traffic fatalities, with over 3,120 fatalities in 2015, due to the state’s ongoing drilling and fracking boom (Olsen, 2014). Table 2.6 provides information on motor vehicle crash deaths per state and highlights Texas as the state with the most fatal crashes, deaths, and deaths per 100 million vehicle miles travelled. There is no exact way to correlate deaths to energy development activities. However, the counties that experienced the largest increase in accidents were in the Permian Basin and the Eagle Ford Shale, as shown in Figure 2.6. In 2013, there were 3,430 traffic reported accidents in the energy development areas that resulted in serious injuries or fatalities. The 26 counties in energy development areas that stretch from Laredo to Huntsville accounted for 236 of fatalities of the total reported accidents (TxDOT, 2018).
Table 2.6: Motor Vehicle Crash Deaths per State (Insurance Institute for Highway Safety, 2016)

<table>
<thead>
<tr>
<th>State</th>
<th>Population</th>
<th>Vehicle miles traveled (millions)</th>
<th>Fatal crashes</th>
<th>Deaths</th>
<th>Deaths per 100,000 population</th>
<th>Deaths per 100 million vehicle miles travel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alabama</td>
<td>4,858,979</td>
<td>67,257</td>
<td>783</td>
<td>849</td>
<td>17.5</td>
<td>1.26</td>
</tr>
<tr>
<td>Alaska</td>
<td>738,432</td>
<td>5,045</td>
<td>60</td>
<td>65</td>
<td>8.8</td>
<td>1.29</td>
</tr>
<tr>
<td>Arizona</td>
<td>6,828,065</td>
<td>65,045</td>
<td>810</td>
<td>893</td>
<td>13.1</td>
<td>1.37</td>
</tr>
<tr>
<td>Arkansas</td>
<td>2,978,204</td>
<td>34,897</td>
<td>472</td>
<td>531</td>
<td>17.8</td>
<td>1.52</td>
</tr>
<tr>
<td>California</td>
<td>39,144,818</td>
<td>335,539</td>
<td>2,925</td>
<td>3,176</td>
<td>8.1</td>
<td>0.95</td>
</tr>
<tr>
<td>Colorado</td>
<td>5,456,574</td>
<td>50,437</td>
<td>506</td>
<td>546</td>
<td>10.0</td>
<td>1.08</td>
</tr>
<tr>
<td>Connecticut</td>
<td>3,590,886</td>
<td>31,592</td>
<td>253</td>
<td>266</td>
<td>7.4</td>
<td>0.84</td>
</tr>
<tr>
<td>Delaware</td>
<td>945,934</td>
<td>9,931</td>
<td>122</td>
<td>126</td>
<td>13.3</td>
<td>1.27</td>
</tr>
<tr>
<td>D.C.</td>
<td>672,228</td>
<td>3,557</td>
<td>23</td>
<td>23</td>
<td>3.4</td>
<td>0.65</td>
</tr>
<tr>
<td>Florida</td>
<td>20,271,272</td>
<td>206,982</td>
<td>2,699</td>
<td>2,939</td>
<td>14.5</td>
<td>1.42</td>
</tr>
<tr>
<td>Georgia</td>
<td>10,214,860</td>
<td>118,107</td>
<td>1,327</td>
<td>1,430</td>
<td>14.0</td>
<td>1.21</td>
</tr>
<tr>
<td>Hawaii</td>
<td>1,431,603</td>
<td>10,301</td>
<td>86</td>
<td>94</td>
<td>6.6</td>
<td>0.91</td>
</tr>
<tr>
<td>Idaho</td>
<td>1,654,930</td>
<td>16,662</td>
<td>198</td>
<td>216</td>
<td>13.1</td>
<td>1.30</td>
</tr>
<tr>
<td>Illinois</td>
<td>12,859,995</td>
<td>105,223</td>
<td>914</td>
<td>998</td>
<td>7.8</td>
<td>0.95</td>
</tr>
<tr>
<td>Indiana</td>
<td>6,619,680</td>
<td>78,819</td>
<td>756</td>
<td>821</td>
<td>12.4</td>
<td>1.04</td>
</tr>
<tr>
<td>Iowa</td>
<td>3,123,899</td>
<td>33,161</td>
<td>282</td>
<td>320</td>
<td>10.2</td>
<td>0.96</td>
</tr>
<tr>
<td>Kansas</td>
<td>2,911,641</td>
<td>31,379</td>
<td>322</td>
<td>355</td>
<td>12.2</td>
<td>1.13</td>
</tr>
<tr>
<td>Kentucky</td>
<td>4,425,092</td>
<td>48,675</td>
<td>694</td>
<td>761</td>
<td>17.2</td>
<td>1.56</td>
</tr>
<tr>
<td>Louisiana</td>
<td>4,670,724</td>
<td>48,180</td>
<td>674</td>
<td>726</td>
<td>15.5</td>
<td>1.51</td>
</tr>
<tr>
<td>Maine</td>
<td>1,329,328</td>
<td>14,629</td>
<td>144</td>
<td>156</td>
<td>11.7</td>
<td>1.07</td>
</tr>
<tr>
<td>Maryland</td>
<td>6,006,401</td>
<td>57,516</td>
<td>472</td>
<td>513</td>
<td>8.5</td>
<td>0.89</td>
</tr>
<tr>
<td>Massachusetts</td>
<td>6,794,422</td>
<td>59,257</td>
<td>291</td>
<td>306</td>
<td>4.5</td>
<td>0.52</td>
</tr>
<tr>
<td>Michigan</td>
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<td>97,843</td>
<td>893</td>
<td>963</td>
<td>9.7</td>
<td>0.98</td>
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<td>Minnesota</td>
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<td>375</td>
<td>411</td>
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<td>0.72</td>
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<td>Mississippi</td>
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<td>677</td>
<td>22.6</td>
<td>1.7</td>
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<td>Missouri</td>
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<td>71,918</td>
<td>802</td>
<td>869</td>
<td>14.3</td>
<td>1.21</td>
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<td>204</td>
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<td>21.7</td>
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<td>20,101</td>
<td>218</td>
<td>246</td>
<td>13</td>
<td>1.22</td>
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<tr>
<td>Nevada</td>
<td>2,890,845</td>
<td>25,925</td>
<td>296</td>
<td>325</td>
<td>11.2</td>
<td>1.25</td>
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<tr>
<td>New Hampshire</td>
<td>1,330,608</td>
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<td>103</td>
<td>114</td>
<td>8.6</td>
<td>0.87</td>
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<tr>
<td>New Jersey</td>
<td>8,958,013</td>
<td>75,393</td>
<td>522</td>
<td>562</td>
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<td>New Mexico</td>
<td>2,085,109</td>
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<td>269</td>
<td>298</td>
<td>14.3</td>
<td>1.09</td>
</tr>
<tr>
<td>New York</td>
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<td>127,230</td>
<td>1,046</td>
<td>1,121</td>
<td>5.7</td>
<td>0.88</td>
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<tr>
<td>North Carolina</td>
<td>10,042,802</td>
<td>111,879</td>
<td>1,275</td>
<td>1,379</td>
<td>13.7</td>
<td>1.23</td>
</tr>
<tr>
<td>North Dakota</td>
<td>756,968</td>
<td>10,036</td>
<td>111</td>
<td>131</td>
<td>17.3</td>
<td>1.31</td>
</tr>
<tr>
<td>Ohio</td>
<td>11,613,423</td>
<td>113,673</td>
<td>1,029</td>
<td>1,110</td>
<td>9.6</td>
<td>0.98</td>
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<tr>
<td>Oklahoma</td>
<td>3,911,338</td>
<td>47,713</td>
<td>588</td>
<td>643</td>
<td>16.4</td>
<td>1.35</td>
</tr>
<tr>
<td>Oregon</td>
<td>4,028,977</td>
<td>35,999</td>
<td>412</td>
<td>447</td>
<td>11.1</td>
<td>1.24</td>
</tr>
<tr>
<td>Pennsylvania</td>
<td>12,802,503</td>
<td>100,945</td>
<td>1,102</td>
<td>1,200</td>
<td>9.4</td>
<td>1.19</td>
</tr>
<tr>
<td>Rhode Island</td>
<td>1,056,298</td>
<td>7,833</td>
<td>41</td>
<td>45</td>
<td>4.3</td>
<td>0.57</td>
</tr>
<tr>
<td>South Carolina</td>
<td>4,896,146</td>
<td>51,726</td>
<td>909</td>
<td>977</td>
<td>20.0</td>
<td>1.89</td>
</tr>
<tr>
<td>South Dakota</td>
<td>858,469</td>
<td>9,324</td>
<td>115</td>
<td>133</td>
<td>15.5</td>
<td>1.43</td>
</tr>
<tr>
<td>Tennessee</td>
<td>6,600,299</td>
<td>76,670</td>
<td>884</td>
<td>958</td>
<td>14.5</td>
<td>1.25</td>
</tr>
<tr>
<td>Texas</td>
<td>27,469,114</td>
<td>258,122</td>
<td>3,124</td>
<td>3,516</td>
<td>12.8</td>
<td>1.36</td>
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<tr>
<td>Utah</td>
<td>2,995,919</td>
<td>29,604</td>
<td>256</td>
<td>276</td>
<td>9.2</td>
<td>0.93</td>
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<tr>
<td>Vermont</td>
<td>626,042</td>
<td>7,314</td>
<td>50</td>
<td>57</td>
<td>9.1</td>
<td>0.78</td>
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<tr>
<td>Virginia</td>
<td>8,382,993</td>
<td>82,625</td>
<td>711</td>
<td>753</td>
<td>9.0</td>
<td>0.91</td>
</tr>
<tr>
<td>Washington</td>
<td>7,170,351</td>
<td>59,653</td>
<td>516</td>
<td>568</td>
<td>7.9</td>
<td>0.95</td>
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<tr>
<td>West Virginia</td>
<td>1,844,128</td>
<td>19,827</td>
<td>246</td>
<td>268</td>
<td>14.5</td>
<td>1.35</td>
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<tr>
<td>Wisconsin</td>
<td>5,771,337</td>
<td>62,073</td>
<td>523</td>
<td>566</td>
<td>9.8</td>
<td>0.91</td>
</tr>
<tr>
<td>Wyoming</td>
<td>586,107</td>
<td>9,597</td>
<td>128</td>
<td>145</td>
<td>24.7</td>
<td>1.51</td>
</tr>
<tr>
<td>U.S total</td>
<td>321,418,821</td>
<td>3,095,373</td>
<td>32,166</td>
<td>35,092</td>
<td>10.9</td>
<td>1.13</td>
</tr>
</tbody>
</table>
2.4. **Impacted Damages on Pavements by SHL Vehicles**

Although the contribution of the SHLs to the total percentage of the operating trucks might not be significant, SHL vehicles have a high potential for substantial damage of the pavements due to more taxing stress path compared to the conventional trucks. For instance, based on the mechanistic quantification of the damage described in Ashtiani et al. (2019), it was found that one passage of an SHL with GVW of 364 kips on the studied pavement section of SH 123 in Corpus Christi can potentially induce 125 times of the damage imparted by an 80-kips reference vehicle. For this reason, the analysis of the truck traffic patterns in regions subjected to the SHL movements should accurately account for the heaviest loads characterized in the tail end of the load distribution histograms, as schematically shown in Figure 2.7. The loads located within the highlighted zone can adversely impact the longevity of the pavement infrastructures, since even a few passages of such super heavy loads can impart substantial damage on the pavement structures.
The major types of damages/distress imparted on the pavement sections due to the super heavy load movements include, but not limited to the following five categories (TxDOT pavement manual, 2019):

- **Shearing of the pavement surface during turning movements**, shown in Figure 2.8 (a).
- **Rutting failure**, shown in Figure 2.8 (b).
- **Peeling of fresh seal coats or AC pavement overlays**, shown in Figure 2.8 (c).
- **Bleeding of seal coats or surface layers**, shown in Figure 2.8 (d).
- **Lateral shear failure at the pavement edge**, shown in Figure 2.8 (e).
2.5. REVIEW OF SUPER HEAVY LOAD PERMITTING PRACTICES IN THE UNITED STATES

2.5.1. SHL Permit Records

The recovery of the nation’s economy, coupled with the recent energy boom in Texas, has led to significant increases in the SHL permit issuances. Table 2.7 illustrates the number of super heavy Single-Trip permits issued by the Texas Department of Motor Vehicles (TxDMV) between the fiscal year 2004 and 2020. Since the initiative of tracking this permit, the number of issued permits doubled from the previous year until 2007. Then, the permits reached an all-time high in 2008 and 2009 with 1,238 and 1,525 issued permits, respectively; followed by several years of fewer issued permits and spiking up again in 2014 through 2016 – corresponding to the most recent boom in the oil and gas industry.

Based on our recent communication with TxDMV Motor Carrier Division, the number of issued SHL permits drastically increased during 2017-2019 period by almost 100%. Moreover, during the first few months of the fiscal year 2020, TxDMV issued 832 permits for the SHL vehicles. Another noticeable finding from the review of the historical records of the SHL permits in Texas was the fact that the majority of the issued permits were attributed to the SHLs serving oil and gas industries in the Eagle Ford Shale and Permian Basin regions and less related to freight operations (Correspondence with TxDMV, February 2020).
Table 2.7: Super Heavy Load Permits Issued by the TxDMV (from Ashtiani et al., 2019, and personal communication with TxDMV, February 2020)

<table>
<thead>
<tr>
<th>Fiscal Year</th>
<th>SHL Permits Issued</th>
<th>Fiscal Year</th>
<th>SHL Permits Issued</th>
</tr>
</thead>
<tbody>
<tr>
<td>FY 2004</td>
<td>107</td>
<td>FY 2013</td>
<td>740</td>
</tr>
<tr>
<td>FY 2005</td>
<td>208</td>
<td>FY 2014</td>
<td>1,066</td>
</tr>
<tr>
<td>FY 2006</td>
<td>415</td>
<td>FY 2015</td>
<td>1,493</td>
</tr>
<tr>
<td>FY 2007</td>
<td>821</td>
<td>FY 2016</td>
<td>1,170</td>
</tr>
<tr>
<td>FY 2008</td>
<td>1,238</td>
<td>FY 2017</td>
<td>770</td>
</tr>
<tr>
<td>FY 2009</td>
<td>1,525</td>
<td>FY 2018</td>
<td>985</td>
</tr>
<tr>
<td>FY 2010</td>
<td>700</td>
<td>FY 2019</td>
<td>1,496</td>
</tr>
<tr>
<td>FY 2011</td>
<td>579</td>
<td>FY 2020</td>
<td>832</td>
</tr>
<tr>
<td>FY 2012</td>
<td>676</td>
<td>(Sep. 2019-Feb. 2020)</td>
<td>832</td>
</tr>
</tbody>
</table>

For comparison purposes, Nevada is another state suffering from the distresses imposed by the SHL in the highway transportation network. Available databases of SHL permits in Nevada indicate 1,398 permits issued between the years 2004 and 2013 (Batioja, 2017), as shown in Figure 2.9. The majority of them, i.e., 89%, corresponded to vehicles with a GVW less than 500,000 lb. Another noteworthy observation from the Nevada SHL patterns is that a noteworthy number of SHL exceeded 1 million lb., with the maximum recorded weight as high as 6.2 million lb.
As elaborated, the utilization of SHL vehicles has become more common in recent years due to higher industry demands that resulted in a high number of SHL movements in various transportation network systems. In a relevant study, Papagiannakis (2015) conducted a web-based survey to collect information on the number of SHL permits issued throughout the US. Figure 2.10 illustrates the outcome of this survey, showing the number of SHL permits issued annually. The plot shows that although the majority of the state highway agencies issue fewer than 1,000 SHL permits per year, several other states reported a very large number of issued permits, i.e., more than 10,000 permits per year, which is an alarming number of operations of such heavy vehicles.
Figure 2.10: Number of SHL Permits Issued Annually in the U.S. (Papagiannakis, 2015).

2.5.2. Pavement Analysis: An Integral Component in SHL Permitting Process

Due to extremely high load magnitude of the SHLs, as well as the alarming frequency of their operations throughout transportation networks, determination of the imparted damage on pavement facilities, and imposing permit fees in accordance with the level of induced damages are integral components of the decisions for the SHL permit. Ideally, the permit fees should at least cover the infrastructure damage associated costs due to SHL trucks plus any administrative costs such as permit processing, engineering analysis, and enforcement (Papagiannakis, 2015). Currently, the majority of the State Highway Agencies (SHAs) use some measures such as GVW, axle weight, and distance traveled for determination of the permit fees attributed to the SHL trucks. Although these parameters contribute to the total imparted damages, the permit fee structures that only consider the aforementioned characteristics of the SHLs might not be reasonably accurate. Moreover, some other SHAs collect flat fees for single-trip permits, disregarding the associated damages imparted on the pavement facilities. Consequently, SHL permitting process should
properly account for the damages imparted on the pavement infrastructures by SHL truck movements.

As part of the permit request process, a few highway agencies allow SHLs to operate subsequent to a preliminary analysis of their impacts on the pavement facilities on the selected route. In a relevant research effort, Papagiannakis (2015) reported that only 15% of the SHAs always conduct pavement analysis for permitting SHL trucks, as shown in Figure 2.11. Additionally, 40% of the studied SHAs conduct pavement damage analysis depending on the specific circumstances. A prime example of this category pertains to the criteria defined for the engineering analysis of pavements subjected to SHLs in Texas. Pavement engineers in TxDOT Maintenance Division (MNT) are typically asked to conduct pavement damage analysis when the SHL has GVW exceeding 500 kips and or tire loads above 6 kips. Another noticeable observation from Figure 2.11 is that around 45%, i.e., 18 out of 40, of the studied SHAs never perform pavement damage analysis through the permit issuance process.

![Figure 2.11: Distribution of States Performing Pavement Analysis as part of SHL Permitting (Papagiannakis, 2015).](image)

Additionally, the type and capability of the pavement damage analysis conducted by different highway agencies need to be investigated. The aforementioned study documented that the majority of the responding agencies used either their own developed state-specific pavement
analysis approaches or the methods currently deployed by the pavement design industry, such as the Asphalt Institute (AI) approach, as shown in Figure 2.12. In these approaches, the induced pavement damage under the applied loading conditions is predicted using the developed transfer functions.

Figure 2.12 also indicated that several other agencies employed the tabulated damage equivalency factors and ESAL concept presented in the 1993 AASHTO Design Guide, for characterization of the pavement damage associated with SHL vehicles. It was also reported that one agency deployed the ESAL concept, as well as the calculated mechanistic load equivalency factors, to quantify the associated pavement damage. It is noted that none of the responding agencies used the Mechanistic-Empirical Pavement Design Guide (MEPDG) for evaluation of the pavement performance under the passing super heavy loads (Papagiannakis, 2015). Using the described approaches, instead of deploying ME methods for prediction of the pavement performance, can seriously jeopardize the accuracy and reliability of the results obtained from the pavement damage analysis. Batioja et al. (2018) documented that only a few agencies request or conduct ME pavement analysis during the permit issuance process of the SHL vehicles. However, such analyses tend to preliminarily assess the potential failure risk of the pavement section, without quantification of the imparted damage.
2.5.3. **Current State of the Practice in Texas**

As stated earlier in this chapter, TxDOT uses a 254.3 kips GVW threshold for the definition of the SHL trucks. Current threshold criteria for analysis of the pavements subjected to SHLs were established based on the empirical information and the pavement distresses observed during field studies of the SHL movements. As depicted in Figure 2.13, based on empirical data and the records of the issued SHL permits, the GWV and load on tire are the primary criteria for making decisions about the permit issuance. As shown in Figure 2.13, vehicles with GVW exceeding 500 kips and load on tire above 6000 lb pose the highest probability of damage in roadways.
Figure 2.13: General Relationship between GVW and Tire Loads, based on Historical Records of Issued SHL Permits (TxDOT Pavement Manual, 2019).

Figure 2.14 shows the current SHL evaluation process followed by TxDOT Maintenance Division. In the case that pavement analysis is required and the proposed SHL route contains load zoned segments, the evaluation traditionally relies on the review of the PMIS distress and condition scores of the specified route. Furthermore, if the maximum trailer tire load surpasses the 10 kips limit, additional FWD testing or visual distress evaluation in the field might be needed to investigate the existing condition of the pavement sections subjected to the SHL movement. Ultimately, the proposed SHL route can be approved if the pavement condition is acceptable; otherwise, rerouting is requested.

Adopting such an analysis approach that is merely supported by empirical data and is based on the preliminary evaluation of the pavement distresses and conditions, fails to accurately assess the current demanding loading conditions of the SHL trucks operating in Texas overload corridors. Hence, the SHL detrimental impacts and the severity of the damages imparted on the pavement facilities might be overlooked.
Consequently, there is an urgent need to develop and adopt a comprehensive mechanistic-based approach that accounts for the accurate level of imparted damage, as well as different aspects of structural impacts on pavement systems attributed to the SHL vehicles operating in the network. Implementing such an approach in the permitting procedure and/or pavement design manual can further update the strategies in establishing permit fee structures that cover the costs associated with the SHL detrimental impacts. This can potentially protect States’ assets by reimbursement of the reconstruction and/or rehabilitation costs spent to restore the structural capacities of the deteriorated pavement facilities. Therefore, it is imperative to accurately quantify the impacts of the SHL vehicles through mechanistic characterization of the pavement performance, pavement stability, and the imparted damages.
2.6. **SHL and Pavement Analysis in Previous Studies**

Since the 1990s, a number of researchers studied different methodologies to evaluate the damages/structural impacts on the pavements due to the SHL vehicle operations. Jooste and Fernando (1994) developed a procedure for the determination of the pavement responses under SHL movements. The authors also measured the pavement deflections induced by SHL vehicles,
using the Multi-Depth Deflectometer (MDD). The researchers found that reasonable predictions of pavement response under SHL movements could be obtained using the layered elastic theory. They also used the Mohr-Coulomb yield criterion to investigate the pavement structural capacity under the SHLs. In a relevant study, Fernando (1997) evaluated the stability of pavement structures, considering the edge load condition attributed to the SHLs.

Chen et al. (2005) studied the general SHL permitting program in Texas. They analyzed 63 SHL vehicles moving in the network between 2001 and 2002. Nondestructive testing such as Falling Weight Deflectometer (FWD) and Ground Penetrating Radar (GPR) and pavement condition surveys were conducted before and immediately after SHL moves. Subsequently, using the Asphalt Institute (AI) transfer functions and Equivalent Single Axle Load (ESAL) concept, the number of load repetitions to reach failure was determined. Ultimately, the corresponding values were contrasted with the post and pre SHL movements to estimate the service life reduction. The authors also documented that there were no observed structural-related damages in the pavements during the field monitoring process. However, it was reported that SHL moves can cause pavement damages for the cases with fresh seal coat overlay, particularly under the following conditions: (1) immediately after placement, (2) high temperatures, and (3) steep grades in combination with too few drive axles. For this reason, a policy was adopted to allow the SHL vehicles to pass through pavements only after five weeks or more after reconstruction/rehabilitation activities. Later, Chen et al. (2009), through conducting a pavement condition survey in the field, qualitatively assessed six cases of pavement surface damage due to the SHL movements in Texas.

Dong and Huang (2013) evaluated the impact of SHLs on pavements in Tennessee through field measurement of the induced pavement responses. The authors deployed the Benkelman beam and total station to the field to measure the pavement deflections caused by the SHL movements,
as shown in Figure 2.15. Field measurement records indicated that the evaluated SHL did not cause considerable pavement deformation. The researchers also utilized FWD to assess the pavement structural condition before and after the SHL passage. It was found that the SHL operation resulted in a 7% reduction in the back-calculated surface layer moduli of the studied pavement section.

![Field Measurement of Pavement Deflections under SHL Movement in Tennessee, using (a) Benkelman Beam, and (b) Total Station (Dong and Huang, 2013).](image)

Figure 2.15: Field Measurement of Pavement Deflections under SHL Movement in Tennessee, using (a) Benkelman Beam, and (b) Total Station (Dong and Huang, 2013).

A few research efforts have also made to mechanistically evaluate the effect of SHL vehicles on pavement structures. Oh et al. (2010) developed a mechanistic approach to evaluate the fresh seal coat damages due to the SHL operations in low volume roads in Texas. The developed approach estimates the tensile strength of the seal coat to further characterize the associated damage potential. The authors found that the pavement surface temperature was the most critical factor associated with the seal coat damage.
Chen et al. (2013) conducted pavement structural analysis attributed to a case study of SHL vehicle with over 4 million lb. of GVW operating in Louisiana. The researchers used finite element software as well as a layered elastic analysis program, namely, BISAR, to predict the pavement responses under the described SHL move. Then, the Mohr-Coulomb yield criterion was deployed to assess the possibility of the shear failure of the pavement sections subjected to the movement of the evaluated SHL truck. Additionally, the authors proposed an approach to estimate the associated cost of repairing the deteriorated pavement. To achieve this objective, using the AI performance models for rutting and fatigue cracking, as well as the ESAL concept, loss of pavement life due to the operation of the SHL truck, was predicted. Ultimately, the researchers calculated the pavement damage associated cost, considering the estimated service life reduction. The results indicated that the rutting was the main controlling factor in the determination of the accumulated damage and the associated costs under an SHL.

Chatti et al. (2009), using the ESAL concept, calculated the damage equivalency factors based on fatigue cracking and rutting failure criteria form laboratory and mechanistic analyses to further determine the influence of Michigan SHL trucks on pavement distresses. The researchers found that the imparted damages due to excessive surface rutting were primarily associated with the passages of multi-axle heavy trucks in Michigan. Hajj et al. (2018) developed a mechanistic-based analysis approach to investigate the impacts of the SHLs on flexible pavements on a case-by-case basis. Figure 2.16 shows the flowchart of the overall approach developed by the authors. The following sections briefly describe the analysis steps and the theoretical concepts deployed by the authors to analyze the SHL trucks passing the highway pavement sections.

- **Ultimate Failure Analyses:** The authors used Meyerhof’s general bearing capacity concepts to assess the possibility of ultimate shear failure of the subgrade layer in the
pavement structure under loading conditions attributed to the SHL vehicles. Additionally, the wedge method was deployed to evaluate the stability of a sloped pavement shoulder under the SHL-vehicle movement (Hajj et al., 2018).

- **Buried Utility Risk Analysis:** The authors assessed the failure risk against buried structures beneath the pavement surface. To accomplish this, the SHL-induced vertical stress at the location of buried utilities was calculated and contrasted with the strength capacities of the evaluated utilities (Hajj et al., 2018).

- **Service Limit Analyses:** The service limit analyses consisted of localized shear failure analysis and deflection-based service limit analysis. The authors used the Drucker-Prager failure criterion to investigate the possibility of localized failure at the critical location at the top of the subgrade layer under the SHL vehicle. Additionally, the deflection-based service limit analysis assessed the magnitude of the pavement deflections induced by the SHL operation (Hajj et al., 2018).

- **Cost Allocation Analysis:** Subsequent to conducting and satisfying the above-mentioned analyses, the cost allocation analysis is performed. The developed mechanistic approach estimates the pavement damage associated costs based on the ESAL concept and the pavement life reduction due to a single passage of the evaluated SHL vehicle (Hajj et al., 2018).
Additionally, as shown in Figure 2.16, the mitigation strategies might be needed at any stage of the evaluation process when the calculated results fail to meet the particular requirements imposed (Hajj et al., 2018). The authors also deployed the nucleus approach throughout the performed analyses to estimate pavement responses under the SHL-vehicle movements (Hajj et al., 2018). Figure 2.17 shows an example of the representative nucleus. This approach is aimed to identify the components of the SHL-vehicle configuration that can represent the entire SHL
vehicle. Using the nucleus, the cumulative pavement responses can be calculated by superimposing the stresses determined under the nucleus. It should be also noted that the researchers used records of issued permits as the primary source to characterize the loading conditions due to SHL vehicles.

Figure 2.17: Example of Representative Nucleus (Hajj et al., 2018).

The researchers deployed the 3D-Move ENHANCED and SuperPACK software for the numerical simulation purposes to evaluate the impacts of SHLs on pavements (Hajj et al., 2018). Figure 2.18 illustrates a screenshot of the main window of SuperPACK software. As shown in the figure, initially, the information on pavement structure is incorporated into the software. Then, subsequent to the determination of the vehicle speed and analysis temperature, available FWD data could be also incorporated into the program. Then, the pre-analysis modules are designed to define analysis inputs associated with vehicle axle configurations, material properties, subgrade shear parameters, and representative material properties for SHL and reference vehicles. Ultimately, the primary analysis modules, i.e., bearing capacity, service limit, slope stability, buried utility, cost allocation, are developed to assess the impacts of SHLs on pavement structures.
In a recent study, Gonzalez (2021) developed an analysis methodology to quantify the damages imparted by SHLs on flexible pavements in Quebec, Canada. The researchers proposed a damage quantification parameter based on the permanent deformation accumulated in base and subgrade layers due to SHL passages (Gonzalez et al., 2021). The authors documented that excessive permanent deformation is one of the major concerns associated with SHL movements resulting in deterioration of pavement structures. The researchers further developed a procedure to calibrate and validate the proposed deformation rate model, using extensive laboratory/field tests, and finite element analysis. In the numerical simulation phases, the researchers considered uniform distribution of contact stresses to simulate the tire-pavement interactions. In the aforementioned study, the pavement structure was also evaluated as an elastic system.

2.7. **SUMMARY OF THE PREVIOUS STUDIES AND CONCLUSIONS**

Tables 4 presents a summary of the major previous research studies conducted to evaluate the impacts of SHL vehicles on flexible pavement structures. Information on the deployed software and methodology and the major findings of the conducted researches are also provided in Table 2.8.
As elaborated in this chapter, preceding studies provided insights into the SHL movements and the associated damages on the pavement structures. However, the majority of the proposed analysis approaches typically rely on general observations and measurements made in the field without mechanistic characterization of the associated damages. Essentially, several researchers in the prior studies either conducted pavement condition surveys in the field or deployed nondestructive testing equipment to preliminarily evaluate the imparted damages or the changes in structural capacity of the pavements subjected to the SHL vehicles passages.

Additionally, the majority of the mechanistic-based damage quantification studies either rely on the issued permit data as the primary source to characterize the loading conditions, or are based on limited sections and data points. The simplifying assumptions such as the use of permit records in lieu of field data collection, as well as limitation of type of pavement facilities in the study, overlooking the influence of seasonal variation material properties, and unique characteristics of the pavement structure in each location can potentially jeopardize the accuracy and reliability of the damage quantification and remaining life analyses of pavement facilities.

Another limitation persistent in the literature pertains to unrealistic simulation of the tire-pavement contact stresses using uniformly distributed load, rather than considering non-uniform distribution of the contact stresses. Relying on such simplifying assumptions can be detrimental to the accuracy of the analysis of structural impacts of SHL vehicles with demanding loading conditions. Hence, there is a lack of a well-accepted and verified ME analysis protocol for quantification of the imparted damages and the loss of service life of the pavements imparted by SHL operations, considering the site-specific Axle Load Spectra (ALS) databases, realistic tire-pavement interactions, and unique features of pavement facilities in the network.
Additionally, the structural impact of SHLs on transportation infrastructures is another important aspect of these heavy vehicles, besides the imparted pavement damages, that needs to be accurately assessed. Essentially, the non-conventional axle loading conditions, coupled with the slow-moving nature of the SHL vehicles operating under demanding environmental scenarios such as flooding conditions can substantially increase the failure risk of the pavement foundations and pavement shoulders. Evidently, this is an ongoing nationwide challenge that has been never addressed through the stability analysis approaches proposed in previous research studies. Consequently, it deems necessary to develop an all-encompassing protocol for accurate quantification of the imparted damage, and various structural impacts of the SHL vehicles on highway infrastructure facilities, considering the environmental factors, and unique traffic demands in overload corridors.
Table 2.8: Selected Research on SHL Vehicles Impacts on Flexible Pavement Systems across the Nation

<table>
<thead>
<tr>
<th>Research Description</th>
<th>Research Methodology &amp; Findings</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Researchers:</strong> Hajj et al. (2018)</td>
<td><strong>Methodology</strong></td>
</tr>
</tbody>
</table>
| **Title:** “Analysis Procedures for Evaluating Superheavy Load Movement on Flexible Pavements” | ➢ The researchers developed a comprehensive mechanistic-based analysis methodology associated with SHL movement on flexible pavements.  
➢ Subgrade bearing failure analysis, buried utility risk analysis, localized shear failure analysis, and cost allocation analysis were the major components of this research effort. |
| **State:** Nevada | **Major Findings** |
| **Software(s) Used:** 3D Move ENHANCED, SuperPACK | ➢ The developed methodology was based on available analysis and evaluation procedures.  
➢ The findings from this study revealed that the developed methodology is helpful in assessing the impacts of the SHL movements on the pavements. |

<table>
<thead>
<tr>
<th>Research Description</th>
<th>Research Methodology &amp; Findings</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Researchers:</strong> Chatti et al. (2009)</td>
<td><strong>Methodology</strong></td>
</tr>
<tr>
<td><strong>Title:</strong> “Effect of Michigan Multi-axle Trucks on Pavement Distress”</td>
<td>➢ The researchers calculated the damage equivalency factors based on fatigue cracking and rutting failure from laboratory and mechanistic analyses.</td>
</tr>
<tr>
<td><strong>State:</strong> Michigan</td>
<td><strong>Major Findings</strong></td>
</tr>
</tbody>
</table>
| **Software(s) Used:** KENPAVE | ➢ Imparted damages due to excessive surface rutting were primarily associated with the passages of multi-axle heavy trucks in Michigan,  
➢ Calculated damage equivalency factors were significantly higher than those from AASHTO, especially for thinner flexible pavements. |
Table 2.7: Selected Research on SHL Vehicles Impacts on Flexible Pavement Systems across the Nation (cont.)

<table>
<thead>
<tr>
<th>Research Description</th>
<th>Research Methodology &amp; Findings</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Researchers:</strong> Oh et al. (2010)</td>
<td><strong>Methodology</strong></td>
</tr>
<tr>
<td><strong>Title:</strong> “Mitigating Seal Coat Damage due to Superheavy Load Move in Texas Low Volume Roads”</td>
<td>➢ An M-E approach was developed to evaluate and mitigate fresh seal coat damage due to SHL movements.</td>
</tr>
<tr>
<td><strong>State:</strong> Texas</td>
<td><strong>Major Findings</strong></td>
</tr>
<tr>
<td><strong>Software(s) Used:</strong> Mechanistic-Empirical Seal Coat Damage Evaluation Program (M-E SDEP)</td>
<td>➢ The pavement surface temperature was found to be the most critical factor associated with seal coat damage,   ➢ The developed M-E approach exhibits great potential for evaluating and mitigating seal coat damage due to SHL vehicle loading on low volume roads in Texas.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Research Description</th>
<th>Research Methodology &amp; Findings</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Researchers:</strong> Chen et al. (2005)</td>
<td><strong>Methodology</strong></td>
</tr>
<tr>
<td><strong>Title:</strong> “A Review of the Superheavy Load Permitting Programme in Texas”</td>
<td>➢ FWD, GPR, DCP, and a condition survey was conducted before and immediately after SHL moves.   ➢ Using the Asphalt Institute (AI) equations, the number of load repetitions to reach failure was determined.   ➢ Corresponding values were contrasted associated with the post and pre SHL movements.</td>
</tr>
<tr>
<td><strong>State:</strong> Texas</td>
<td><strong>Major Findings</strong></td>
</tr>
<tr>
<td><strong>Software(s) Used:</strong> BISAR</td>
<td>➢ The most frequent SHL permits issued in the range of 600–700 kips of GVW.   ➢ In all SHL cases, rutting was the controlling factor.   ➢ SHLs can cause serious damages to seal coat roads under following conditions: (1) time after placement, (2) high temperatures, and (3) steep grades in combination with too few drive axles.   ➢ A policy was adopted to re-route SHL moves to 5 weeks or older pavements.</td>
</tr>
<tr>
<td>Research Description</td>
<td>Research Methodology &amp; Findings</td>
</tr>
<tr>
<td>----------------------</td>
<td>--------------------------------</td>
</tr>
<tr>
<td><strong>Researchers:</strong> Chen et al. (2013)</td>
<td><strong>Methodology</strong></td>
</tr>
</tbody>
</table>
| **Title:** “Evaluation of Superheavy Load Movement on Flexible Pavements” | ➢  3D finite element method was used to assess the pavement damage potential under an SHL.  
➢ Damage equivalency factors were calculated using the AI performance models.  
➢ The damage associated cost analysis was based on the predicted damage caused by an SHL move, considering the estimated cost for repairing the damaged pavement. |
| **State:** Louisiana | **Major Findings** |
| **Software(s) Used:** Finite Element Software and BISAR | ➢ Rutting was the controlling factor to evaluate the accumulation of incremental damage under an SHL.  
➢ BISAR program could be used to predict pavement performance under SHLs. |
| **Researchers:** Dong and Huang- (2013) | **Methodology** |
| **Title:** “Field Measurement of Pavement Responses under Superheavy Load” | ➢ Benkelman beam, total station, and FWD were deployed to the field to measure pavement deflections caused by the SHL movements. |
| **State:** Tennessee | **Major Findings** |
| | ➢ The Benkelman Beam, and total station surveys indicated that the studied SHL did not cause significant permanent deformation.  
➢ FWD test results showed that the back-calculated pavement surface moduli decreased by 7% after SHL passage. |
Chapter 3: Assessment of the Current Network

3.1. INTRODUCTION

With assistance from the project advisory panel, the research team developed a survey questionnaire to document Districts affected by energy development operations. The main objective of this task was to document the extent, severity, and location of severely distressed sites primarily affected by oversize/OW (OS/OW) vehicles. Additionally, the research team gathered information on OS/OW permits issued by the Department of Motor Vehicles (DMV). Ultimately, the representative sites in the energy development areas and OW corridors of Texas, as well as the rationale behind the site selection, are discussed in this chapter.

Survey responses were received from the following 17 Districts: Dallas, Houston, Paris, Pharr, San Angelo, Bryan, Fort Worth, Corpus Christi, Laredo, Austin, Odessa, Tyler, Abilene, El Paso, San Antonio, Yoakum, and Beaumont as noted in Figure 3.1. The research team was particularly interested in the survey responses from the following Districts in the overload corridors of south Texas with emphasis on the Eagle Ford Shale region: 1) Laredo, 2) San Antonio, 3) Corpus Christi, 4) Yoakum, 5) Austin, 6) Bryan, and 7) Pharr District. This chapter summarizes all responses to the online survey and highlights the responses from Districts with OW corridors and in the Eagle Ford Shale region. The collected information will be instrumental for selection of sites for deployment of the WIM devices and nondestructive field testing of representative pavements sections.
3.2. EXISTING OS/OW PERMIT INFORMATION

In recent years the utilization of oversize and OW (OS/OW) vehicles has become more common due to higher industry demands. The use of (OS/OW) vehicles has several benefits such as reduction of traffic congestion, reduced fuel consumption, and lower CO₂ emissions as a result of fewer vehicles traveling in the highway systems. Despite the potential benefits of using (OS/OW) vehicles, heavier vehicles detrimentally impact the transportation infrastructure by accelerating pavement damage and causing premature failure (Batioja-Alvarez et al., 2018). These deteriorated highway systems require state agencies to spend millions in reconstruction, repair and maintenance.

Figure 3.2 illustrates the annual volume of OS/OW permits issued between 1995 and 2017. Though the plot can be characterized by a fluctuating trend of increases and decreases in issued permits, the general trend of the plot shows that the issue of permits has been gradually increasing over the years. There are three noticeable peaks within each rise and fall of annual permits issued in 1998, 2008, and 2014. Expectedly, these years coincide with sharp increase in oil prices.
Figure 3.2: Annual Volume (1995-2017) of Oversize/OW (OS/OW) Permits Issued (from N. Edington, personal communication, February 12, 2018).

*Data compiled from the Texas Permitting and Routing Optimization System (TxPROS) by fiscal year and does not include Temporary Registration
Figure 3.3 shows the periods where peaks in price for the 42-gallon barrel of the West Texas Intermediate crude oil occur coincide with the years with peaks in OS/OW permits issuance by TxDMV.

![West Texas Intermediate (WTI) Crude Oil History Chart](image)

**Figure 3.3:** West Texas Intermediate (WTI) Crude Oil History Chart (Macrotrends, 2018).

Additionally, Table 3.1 illustrates the number of OS/OW permits issued for the oil and gas industry compared to the total permits. As evidenced in this table, the energy industry is responsible for slightly more than 1/3 of the total issued permits. In 2014 alone, the oil and gas industry accounted for nearly half of all issued permits. Therefore, it can be safely assumed that the annual volume of issued OS/OW permits is closely intertwined with the oil price per barrel and the energy development companies.

**Table 3.1:** Corresponding Percentage of Permits Issued to Gas and Oil Industry (from N. Edington, personal communication, February 12, 2018).

<table>
<thead>
<tr>
<th>Fiscal Year</th>
<th>Gas and Oil Industry Percentage of Corresponding Permits</th>
</tr>
</thead>
<tbody>
<tr>
<td>FY 2012</td>
<td>37%</td>
</tr>
<tr>
<td>FY 2013</td>
<td>43%</td>
</tr>
<tr>
<td>FY 2014</td>
<td>44%</td>
</tr>
<tr>
<td>FY 2015</td>
<td>41%</td>
</tr>
<tr>
<td>FY 2016</td>
<td>32%</td>
</tr>
<tr>
<td>FY 2017</td>
<td>37%</td>
</tr>
</tbody>
</table>
3.3. **Survey Results**

3.3.1. Are the Transportation Infrastructure Facilities in Your District Adversely Affected by OW Vehicles Due to Energy Development Activities?

From all the 17 Districts that responded to the survey questionnaire, 94.1% indicated that their transportation infrastructure has been adversely affected by OW vehicles due to energy development activities, as shown in Figure 3.4. The Dallas District was the only District that did not report being severely affected by OW vehicles. As expected, all respondents in the south Texas corridors and Eagle Ford Shale answered in the affirmative.

![Figure 3.4: Percentage Districts Affected by Energy Development Operations.](image)

3.3.2. The Severity of the Damages Imparted by OW Vehicles Associated with Energy Development Activities

Figure 3.5 illustrates the responses to the severity of the damages imparted by OW vehicles in each District, ranked from minimal (1) to severe (10). Dallas District once again was the only District that did not rank the severity of the damages as high. All other respondents ranked the severity at a minimum of 5 or higher. Corpus Christi, Bryan, Abilene, Beaumont, and Tyler Districts all ranked the severity at 7. While El Paso, Houston, Laredo, Yoakum, and San Antonio District ranked the severity at 8. The Districts with the highest ranked severity were San Angelo, Austin, and Odessa Districts at 9, 9, and 10, respectively. The results clearly indicate that the severity of the damages is more pronounced in Districts with active energy development operations.
3.3.3. Typical Pavement Distresses/Damages Due to Energy Production Activities in TxDOT’s Districts:

One of the main objectives of the survey was to identify the typical pavement distresses and damages that the Districts experience. As shown in Figure 3.6, the most prevalent type of distresses among all the Districts are rutting (82.4%), potholes (82.4%), and fatigue cracking (76.5%). Other common distresses indicated by the respondents are slippage cracks (58.8%), edge cracks (58.8%), raveling (52.9%), and longitudinal/traverse cracking (52.9%). Tyler District indicated that the destruction of the seal coat and pavement at the entrance of well sites is also a notable distress that was not listed in our questionnaire. More importantly, for districts strictly in the Eagle Ford Shale region the top pavement distresses were rutting and pot holes, as indicated in Figure 3.7. Based on the Districts responses, the results indicate that most of the distresses and damages caused by energy development operations are inflicted on flexible pavements. Figure 3.7 also illustrates that the damages associated with rigid pavements are not a significant issue with the Districts in the Eagle Ford Shale. This could be attributed to few lane miles of rigid pavements as compared to the flexible pavement sections in the south Texas corridors. This conclusion is in agreement with the information provided in section 3.3.1 of this chapter.
Figure 3.6: Typical Pavement Distresses and Damages among All Responding Districts.

Figure 3.7: Typical Pavement Distresses and Damages among Districts in the Eagle Ford Shale Region.

3.3.4. Availability of active Weigh-In-Motion (WIM) Station in Each District:

The majority of the Districts indicated that they do not have available active weigh-in-motion station in their District, as shown in Figure 3.8. However, Dallas, Paris, Pharr, Fort Worth, Corpus
Christi, Laredo, and Odessa Districts indicated that they do have operational WIM stations.

Figure 3.8: Availability of Operational Weigh-In-Motion (WIM) Stations in All TxDOT Districts

3.3.5. The Frequency of Over-Size/Over-Weight (OS/OW) Truck Traffic Experienced in Districts Highway Network

Figure 3.9, summarizes the District responses to question pertaining to the frequency of OS/OW truck traffic experienced in each District. El Paso, Abilene, and the Bryan Districts were the only Districts to rank the frequency of OS/OW truck traffic as relatively low. On a scale of 1 to 10, the majority of the respondents ranked the frequency of OS/OW traffic at 8 or higher in their Districts. The highest ranked frequencies were indicated by Tyler, Austin, Beaumont, Yoakum, and Odessa Districts, which are Districts in active energy development zones.

Figure 3.9: Frequency of Over-Size/OW (OS/OW) Truck Traffic Experienced.
3.3.6. Growth Patterns for the Over-Size/OW (OS/OW) Truck Traffic in Districts

According to the observations of the different Districts throughout the state, nearly all the respondents indicated an increasing pattern in the frequency of the OS/OW truck traffic in their Districts. None of the Districts indicated that the traffic pattern has been similar to the pre-energy development era in Texas. Yoakum District indicated that the frequency of OS/OW has stayed the same in recent years, post energy boom. Corpus Christi District was the only District to indicate that the traffic operations of OS/OW trucks has been declining in recent years, results are illustrated in Figure 3.10.

![Growth Trends of Over-Size/OW (OS/OW) Truck Traffic](image)

Figure 3.10: Growth Trends of Over-Size/OW (OS/OW) Truck Traffic.

3.3.7. Highways with High Volume of OS/OW Truck Traffic and Severely Distressed Roads

One of the objectives of the survey was to gather information on OS/OW corridors and information on the location of the severely damaged highways and roadways in the energy development areas with emphasis on the Eagle Ford Shale region. This information will be crucial for the selection of sites for the deployment of WIM devices and non-destructive testing of representative sites. Table 3.2 illustrates the highways indicated by each District with high volume of OS/OW truck traffic. Table 3.3 illustrates specific Interstates, US Highways, State Highways, and Farm to Market roads listed by the TxDOT personnel as severely damaged. Predictably, some of the OS/OW corridors listed in Table 3.2 are the same ones that are severely damaged by the truck traffic operations, and are cross-listed in Table 3.3.
## Table 3.2: Roadways with High Volume of OS/OW Truck Traffic.

<table>
<thead>
<tr>
<th>No.</th>
<th>District</th>
<th>Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dallas</td>
<td>No Response</td>
</tr>
<tr>
<td>2</td>
<td>Houston</td>
<td>I-10, SH3, SH 146, SH 225, SH 332, SL 8, SS 330, FM 2004</td>
</tr>
<tr>
<td>4</td>
<td>Pharr</td>
<td>UP0281, US0281, IH0002, FM1016, SH0004, FM0511, SH0048</td>
</tr>
<tr>
<td>5</td>
<td>San Angelo</td>
<td>With the exception of Real and Edwards Counties most any road in the District could be subject to OS/OW</td>
</tr>
<tr>
<td>6</td>
<td>Bryan</td>
<td>IH 45, SH 6</td>
</tr>
<tr>
<td>8</td>
<td>Corpus Christi</td>
<td>US281 and US77</td>
</tr>
<tr>
<td>9</td>
<td>Laredo</td>
<td>US 83, FM133, FM 468, SH 97, FM 469</td>
</tr>
<tr>
<td>10</td>
<td>Austin</td>
<td>All US/IH routes</td>
</tr>
<tr>
<td>11</td>
<td>Odessa</td>
<td>All of Them</td>
</tr>
<tr>
<td>13</td>
<td>Abilene</td>
<td>No Response</td>
</tr>
<tr>
<td>14</td>
<td>El Paso</td>
<td>RM 652, FM 3541, FM 2185, US 62</td>
</tr>
<tr>
<td>15</td>
<td>San Antonio</td>
<td>SH 97, SH 72, SH 16, SH 85, FM 99, FM 2924, US 87, FM 140, FM 791, FM 1344, FM 541, FM 1582, FM 624</td>
</tr>
</tbody>
</table>

* Districts highlighted in grey are within the Eagle Ford Shale Region

## Table 3.3: Severely Distressed Roadways that Need Maintenance and Reparations
<table>
<thead>
<tr>
<th>No.</th>
<th>District</th>
<th>Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dallas</td>
<td>No Response</td>
</tr>
<tr>
<td>2</td>
<td>Houston</td>
<td>US59</td>
</tr>
<tr>
<td>3</td>
<td>Paris</td>
<td>No Response</td>
</tr>
<tr>
<td>4</td>
<td>Pharr</td>
<td>FM1847, FM1732, FM0803, UP0281, US0281, IH0002, FM1016, SH0004, FM0511, SH0048, FM0507, SS0206, FM1425, SH0107, SS0115, FM0681, SH0285</td>
</tr>
<tr>
<td>6</td>
<td>Bryan</td>
<td>Various FM roadways</td>
</tr>
<tr>
<td>7</td>
<td>Fort Worth</td>
<td>SH114 US281 FM2190 FM1191 FM4 FM51 IH20 FM8 FM2491 FM219 FM 1187 RM2871</td>
</tr>
<tr>
<td>8</td>
<td>Corpus Christi</td>
<td>Most distress occurred in Karnes/Live Oak: SH 72, FM 99, SH 239, SH 123, SH 80, etc. but most have been repaired beginning 2012</td>
</tr>
<tr>
<td>9</td>
<td>Laredo</td>
<td>FM 469, US 83</td>
</tr>
<tr>
<td>10</td>
<td>Austin</td>
<td>SH 142 In Caldwell County , SH 21 in Lee County, FM 20 in Bastrop County, US 77 in Lee County</td>
</tr>
<tr>
<td>11</td>
<td>Odessa</td>
<td>Most of Them</td>
</tr>
<tr>
<td>13</td>
<td>Abilene</td>
<td>No Response</td>
</tr>
<tr>
<td>14</td>
<td>El Paso</td>
<td>RM 652, FM 3541</td>
</tr>
<tr>
<td>15</td>
<td>San Antonio</td>
<td>FM 99, FM 2924, SH 85, SH 72, FM 624, FM 1099, SH 173</td>
</tr>
<tr>
<td>17</td>
<td>Beaumont</td>
<td>The roadways mentioned in question 9 have all had some needed repairs over time. None of them have significant issues at the moment due to planning and identifying projects that have helped to preserve the pavement (overlays, sealcoats, etc.)</td>
</tr>
</tbody>
</table>

* Districts highlighted in grey are within the Eagle Ford Shale Region

### 3.3.8. The Impact of Energy Development Activities on the Transportation Infrastructure Network, State Highways (SH), and Farm to Market (FM) Roads:

Based on the survey results, all respondents in the Eagle Ford Shale ranked the severity of energy development operations at a minimum of 5 or greater. Yoakum, Laredo, San Antonio, and Austin Districts indicated that their transportation infrastructure has been severely impacted by the energy developments in the transportation network, as shown by Figure 3.11. The energy development impact on the Districts’ state highways (SH) shown in Figure 3.12, was also significant. However,
all the Districts in the Eagle Ford Shale indicated that the energy development operations are more pronounced in their Farm to Market (FM) system as evidenced in Figure 3.13. The results clearly show that the existing pavement structures along the (FM) roads and some (SH) are not sufficient to sustain the truck traffic operations by energy developing companies.

Figure 3.11: Energy Development Impact on the Transportation Infrastructure in the Eagle Ford Shale Region

Figure 3.12: Energy Development Impact on State Highways (SH) in the Eagle Ford Shale Region.

Figure 3.13: Energy Development Impact on Farm to Market (FM) roads in the Eagle Ford Shale Region.
3.3.9. **Typical Pavement Sections**

According to the survey results, the most common pavement type for Districts in the Eagle Ford Shale region is the asphalt pavement with intermediate thickness (2-1/2” to 5-1/2”), followed by a tie between thick asphaltic concrete pavement (greater than 5-1/2”), and thin surfaced flexible base pavement (less than 2-1/2”). The results suggest that flexible pavements are the most prevalent pavement sections in the Eagle Ford Shale region. Bryan, Austin, and San Antonio Districts were the only respondents to report presence of rigid pavements specifically, continuously reinforced concrete pavement (CRCP), as shown in Figure 3.14.

![Figure 3.14: Typical Pavement Sections in the Eagle Ford Shale Region.](image)

**3.4. ** **REPRESENTATIVE SITES IN THE EAGLE FORD SHALE NETWORK**

Ten representative sites in the Eagle Ford Shale Region were selected based on the survey results after extensive communication with TxDOT personnel in Districts affected by energy developing activities. The research team focused on prioritizing roadways that were severely distressed and that accommodate high volume of Over Size/Over Weight (OS/OW) truck traffic in known energy developing areas and TxDOT priority corridors. In addition, the research team incorporated information such as: 1) proximity to the oil refineries, 2) neighboring oil and gas wells, 3) number of wells in the vicinity of the energy operations areas, 4) distress and conditions scores in the PMIS database, 5) current and upcoming construction/rehabilitation plans for roads in the affected highway network, and 6) previous research conducted. The study considered U.S. Highways, State Highways (SH), and Farm to Market (FM) roads to quantify the traffic operations and to further
assess the detrimental effect of high volume/heavy truck traffic on the network. Major Interstate highways such as, I-35 and I-37 are better suited for truck traffic operations related to energy developments, and some even have operational WIM stations; therefore, these highways were not prioritized in this study. Table 3.4 shows the selected roadways accompanied by the rationale behind the selection of the roadway.
<table>
<thead>
<tr>
<th>District</th>
<th>County</th>
<th>Road Way</th>
<th>Online Survey</th>
<th>TxDOT Priority Corridor</th>
<th>Project Information</th>
<th>Well County Maps</th>
<th>Nearby Refineries &amp; Oil/Gas Companies</th>
<th>PMIS Scores</th>
<th>Literature</th>
</tr>
</thead>
</table>
| 1        | LRD          | La Salle       | US 83         | Listed as distressed road in network | Serves high volume of OS/OW traffic                      | TxDOT identified as priority corridor in Eagle Ford Shale Region. | Numerous oil/gas wells in surrounding area as of 2015                                                      | 1. Basic Energy Services  
2. Chesapeake Energy  
3. Stallion Oilfield  
4. Eastern Oil Well Services  
5. Sunbelt Oil& Gas Rentals | Low Distress and Condition Scores in PMIS database  
N/A                                                                 |
| 2        | LRD          | La Salle       | FM 469/468    | Listed as distressed road in network | Serves high volume of OS/OW traffic                      | N/A              | 1. Construction Scheduled  
2. Finalizing for Construction  
2. All American Plains  
3. NuStar  
4. Patterson 239  
5. Eog Resources  
6. Noble Energy  
7. NOV National Oilwell Varco | Low Distress and Condition Scores in PMIS database  
Referenced in Tech Memo TM-14-03. FM 468 in Cotulla in La Salle County. Experienced premature distress due to high volume & heavy traffic. Pavement was repaired by removing the existing surface treatment and placing 3” HMA layer. |
| 3        | SAT/LRD      | McMullen       | FM 624        | Listed as distressed road in network | Serves high volume of OS/OW traffic                      | N/A              | 1. Construction Scheduled  
2. Finalizing for Construction  
3. Projects Under Development | Numerous oil/gas wells in surrounding area as of 2015 | 1. All American Plains  
2. Storey Ranch | Low Distress and Condition Scores in PMIS database  
| 4        | SAT/CRP      | McMullen/Live Oak/Karnes | FM 99 | Listed as distressed road in network | Serves high volume of OS/OW traffic                      | N/A              | 1. Construction Scheduled  
2. Finalizing for Construction  
3. Projects Under Development | Numerous gas wells in surrounding area as of 2015 | 1. Coy City 1H on FM 99  
2. Buckeye McMullen | Low Distress and Condition Scores in PMIS database  
Referenced in Tech Memo TM-14-01. Construction project limits were from US 281A to the McMullen Co.Line. Researchers tested foam asphalt stabilization for 1-mile section |
| 5        | SAT/LRD      | Atascosa/McMullen | SH 16 | Listed as distressed road in network | Serves high volume of OS/OW traffic                      | TxDOT identified as priority corridor in Eagle Ford Shale Region. | Numerous oil/gas wells in surrounding area as of 2015                                                      | 1. ETS Oilfield Services  
2. Aery 1-1 | Low Distress and Condition Score in PMIS database  
N/A                                                                 |
<table>
<thead>
<tr>
<th>District</th>
<th>County</th>
<th>Road Way</th>
<th>*Online Survey</th>
<th>**TxDOT Priority Corridor</th>
<th>***Project Information</th>
<th>†Well County Maps</th>
<th>‡Well &amp; Oil/Gas Companies</th>
<th>‡‡PMIS Scores</th>
<th>‡‡‡Literature</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>CRP</td>
<td>Karnes</td>
<td>US 181/ SH123</td>
<td>Listed as distressed road in network</td>
<td>Serves high volume of OS/OW traffic</td>
<td>TxDOT identified as priority corridor in Eagle Ford Shale Region.</td>
<td>Numerous gas wells in surrounding area as of 2015</td>
<td>1. Total Safety</td>
<td>Low Distress and Condition Score in PMIS database</td>
</tr>
<tr>
<td>7</td>
<td>CRP/ YKM/ SAT</td>
<td>Live Oak/ Karnes</td>
<td>SH72</td>
<td>Listed as distressed road in network</td>
<td>Serves high volume of OS/OW traffic</td>
<td>TxDOT identified as priority corridor in Eagle Ford Shale Region.</td>
<td>Numerous gas wells in surrounding area as of 2015</td>
<td>1. Energy Transfer Plant (On FM626) 2. South Sugarloaf 3. Buckeye McMullen 4. Aery1-1</td>
<td>Low Distress and Condition Scores in PMIS database</td>
</tr>
<tr>
<td>8</td>
<td>CRP</td>
<td>Karnes</td>
<td>US 281</td>
<td>Listed as distressed road in network</td>
<td>Serves high volume of OS/OW traffic</td>
<td>TxDOT identified as priority corridor in Eagle Ford Shale Region.</td>
<td>Numerous gas wells in surrounding area as of 2015</td>
<td>1. Valero Three Rivers Refinery 2. Kinder Morgan Texas Pipeline</td>
<td>Low Distress and Condition Score in PMIS database</td>
</tr>
<tr>
<td>9</td>
<td>YKM/ CRP</td>
<td>Gonzales</td>
<td>US 183</td>
<td>Listed as distressed road in network</td>
<td>Serves high volume of OS/OW traffic</td>
<td>TxDOT identified as priority corridor in Eagle Ford Shale Region.</td>
<td>Numerous gas wells in surrounding area as of 2015</td>
<td>1. Noble Royalties Inc. 2. Original Art in Oil</td>
<td>Low Distress and Condition Scores in PMIS database</td>
</tr>
<tr>
<td>10</td>
<td>YKM</td>
<td>De Witt</td>
<td>SH 119</td>
<td>Listed as distressed road in network</td>
<td>Serves high volume of OS/OW traffic</td>
<td>N/A</td>
<td>Numerous gas wells in surrounding area as of 2015</td>
<td>1. Pipeline Construction</td>
<td>Low Distress and Condition Score in PMIS database</td>
</tr>
</tbody>
</table>

* Online survey questionnaire was answered by District Engineers and Maintenance Supervisors.
** Identified Priority Corridors where obtained from the PowerPoint presentation "Energy Sector Workshop" (2016) by Randy C. Hopmann.
*** Construction project information was obtained for each roadway from TxDOT Project Tracker.
†Well County maps where used from TxDOT provided documents. Referenced in Implementation Report IR-16-01 "Well County Maps"
‡Well & Oil/Gas Companies
‡‡ Distress Scores and Condition Scores for each roadway where obtained from the PMIS Database for years up to 2010.
‡‡‡Information obtained from the extensive literature that was reviewed.
3.5. Summary of the Major Points:

- The oil and gas industry accounts for more than 1/3 of the total OS/OW permits issued in any given year.

- From the Districts that responded to the survey questionnaire, 94% indicated that their transportation infrastructure has been adversely affected by OW vehicles due to energy development activities.

- The most prevalent type of distresses among all the Districts occur on flexible pavement, they are as follows: rutting (82.4%), potholes (82.4%), and fatigue cracking (76.5%).

- Energy development activities in the Eagle Ford Shale impacted Farm to Market (FM) roads the most adversely.

- Ten representative sites strategically distributed through the overload corridors of Texas, including San Antonio, Corpus Christi, Laredo, and Yoakum Districts, were ultimately selected for further deployment of the nondestructive testing, and the P-WIM units. The rationale behind the site selection is also provided in this chapter.
Chapter 4: Non-Destructive Field Testing of Candidate Sites

4.1. Introduction

This chapter documents the results of pavement condition surveys conducted at ten selected sites in the Eagle Ford Shale network. In addition, forensic studies using Non-Destructive Testing (NDT) were implemented to determine the pavement structure and layer properties. For this purpose, Falling Weight Deflectometer (FWD) and Ground Penetrating Radar (GPR) were deployed to the field for the determination of the layer profile and back-calculation of layer moduli in the surveyed network. The collected information will be utilized for damage quantification purposes as well as for remaining life analyses of the representative pavement sections.

4.2. Pavement Condition Evaluation

Proper evaluation of the pavement conditions is of primary concern to accurately quantify the pavement damages imparted by OW truck traffic. The pavement condition is essentially interconnected with the functional and structural performance properties during the service life of the pavement sections. In order to properly assess the pavement condition, it is necessary to identify the type and the severity of the distress-related damages and the pavement layer configurations besides the layer stiffness properties. To achieve this objective, the research team conducted visual inspection surveys, GPR, and FWD tests in ten pavement sections located in the San Antonio, Corpus Christi, Yoakum, and Laredo District. Table 4.1 indicates the selected roadways, the roadbed type, the data collection lane, the reference markers, and their exact GPS location.
Table 4.1: Location of Selected Roadways in Eagle Ford Shale Network

<table>
<thead>
<tr>
<th>District</th>
<th>County</th>
<th>Roadway</th>
<th>Road Bed</th>
<th>Lane</th>
<th>TRM</th>
<th>GPS Coordinate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laredo</td>
<td>Dimmit</td>
<td>US 83 (SB Lane)</td>
<td>Single</td>
<td>K1</td>
<td>638</td>
<td>(28.504907,-99.838659)</td>
</tr>
<tr>
<td></td>
<td>La Salle</td>
<td>FM 468 (WB Lane)</td>
<td>Single</td>
<td>K6</td>
<td>440</td>
<td>(28.531170,-99.398022)</td>
</tr>
<tr>
<td>San Antonio</td>
<td>McMullen</td>
<td>FM 624 (WB Lane)</td>
<td>Single</td>
<td>K6</td>
<td>500</td>
<td>(28.125868,-98.525511)</td>
</tr>
<tr>
<td></td>
<td>McMullen</td>
<td>FM 99 (SB Lane)</td>
<td>Single</td>
<td>K1</td>
<td>588</td>
<td>(28.465600,-98.440487)</td>
</tr>
<tr>
<td></td>
<td>Atascosa</td>
<td>SH 16 (SB Lane)</td>
<td>Single</td>
<td>K1</td>
<td>642</td>
<td>(28.784422,-98.540370)</td>
</tr>
<tr>
<td>Corpus Christi</td>
<td>Live Oak</td>
<td>US 281 (NB Lane)</td>
<td>Multiple</td>
<td>L1</td>
<td>622</td>
<td>(28.452511,-98.183444)</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>SH 72 (WB Lane)</td>
<td>Single</td>
<td>K1</td>
<td>536</td>
<td>(28.739827,-97.940206)</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>BU 181/SH 123 (SB Lane)</td>
<td>Single</td>
<td>K1</td>
<td>552</td>
<td>(28.878125,-97.893333)</td>
</tr>
<tr>
<td>Yoakum</td>
<td>Gonzales</td>
<td>US 183 (SB Lane)</td>
<td>Single</td>
<td>K6</td>
<td>580</td>
<td>(29.459768,-97.435360)</td>
</tr>
<tr>
<td></td>
<td>Dewitt</td>
<td>SH 119 (WB Lane)</td>
<td>Single</td>
<td>K1</td>
<td>544</td>
<td>(29.036632,-97.572325)</td>
</tr>
</tbody>
</table>

4.3. Visual Inspection of Selected Sites in the Eagle Ford Shale

Visual inspection surveys were performed for all ten selected highways in the Eagle Ford Shale network listed previously in Table 4.1. The visual inspections of pavement sections were conducted under lane closure by examining 100 ft. before and 100 ft. after the portable WIM station. The research team documented and reported the different distress types present in each of the inspected sections per TxDOT’s 2018 PMIS Pavement Rater’s Manual. In addition, Figure 4.1 shows an illustration of the visual inspection plan the research team incorporated.
Table 2 lists a summary of the different pavement distresses associated with each of the inspected highways. As indicated in Table 4.2, rutting and flushing are the predominant pavement distresses related to highways in the Eagle Ford Shale. However, these distresses are expected as these highways service high volumes of heavily loaded truck traffic and OW vehicles related to the oil-gas industry and heavy equipment transportation. Moreover, it was also found that the severe rutting and flushing were more pronounced in FM and SH roads because in addition to the heavy traffic, these highways also have less robust structural layers due to their nature of initial design. The safety of such pavement sections during wet seasons can become a concern as segments with severe rutting coupled with flushing can become extremely slippery due to accumulation of rainwater in the wheel path.
Table 4.2: Pavement Distresses in Representative Roadways in Eagle Ford Shale Network

<table>
<thead>
<tr>
<th>District</th>
<th>Laredo</th>
<th>San Antonio</th>
<th>Corpus Christi</th>
<th>Yoakum</th>
</tr>
</thead>
<tbody>
<tr>
<td>County</td>
<td>La Salle</td>
<td>Dimmit</td>
<td>Atascosa</td>
<td>McMullen</td>
</tr>
<tr>
<td>Rutting</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Patching</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Block Cracking</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alligator Cracking</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Cracking</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse Cracking</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Raveling</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flushing</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Failures</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Potholes</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.4. **NON-DESTRUCTIVE TESTING**

Non-destructive testing of the pavement section is the main procedure of determining the pavement structural characteristics. In recent years vast variety of nondestructive testing methods have been developed, which can provide critical information pertaining to the pavement structure. The UTEP research team deployed GPR and FWD devices as the non-destructive testing methods for further pavement condition evaluation.

4.4.1. **Ground Penetrating Radar (GPR)**

4.4.1.1. **Background**

GPR was implemented by TxDOT in the mid-1990s (FHWA, 2007). GPR testing is used to determine the layer thickness, detect changes in the pavement structure, and identify subsurface moisture. This nondestructive testing system consists of the antenna, data acquisition system and Distance Measuring Instrument (DMI), as shown in Figure 4.2. GPR technology has proven to be
highly effective in Texas due to simplicity and efficiency of the operation. Specifically, the air-coupled GPR is normally operated at highway speed and does not require traffic control, which is extremely important in the studied areas where lane closures are difficult and can potentially pose a safety issue for the traveling public.

The principle of the GPR system operation consists on sending an electro-magnetic (EM) pulse through an antenna to the pavement surface. The pulse reflections and arrival times are detected by a receiver, where there is a contrast in the dielectric properties, as depicted in Figure 4.3. Changes in the dielectric properties are used to assess layer interfaces and layer thicknesses of the pavement sections.

Figure 4.2: GPR Testing in Laredo District.
The thickness of the $i^{th}$ layer could be computed according to the following equation:

$$d_i = \frac{ct_i}{2 \sqrt{\varepsilon_{r,i}}} \quad (4-1)$$

where $d_i$ is the thickness of the $i^{th}$ layer, $t_i$ is the EM wave two-way travel time through the $i^{th}$ layer as shown in Figure 4.3, $c$ is the speed of light in free space ($c=10^8 \text{ m/s}$), and $\varepsilon_{r,i}$ is the dielectric constant of the $i^{th}$ layer (Al-Qadi, 2005). The dielectric constant of a material is an electrical property that is most influenced by moisture content and density.

4.4.1.2. GPR Testing in the Eagle Ford Shale Region

The research team conducted GPR testing on the selected pavement sections (Table 5.1) in overload corridors of the Eagle Ford Shale region. An air-coupled GPR unit equipped with a 2 GHz antenna was deployed in this study to properly evaluate the pavement layer configuration.

Using RADAN computer software, the research team analyzed and interpreted the GPR collected data associated with the 10 selected roadways in the Eagle Ford Shale Districts. It should be noted that the research team conducted GPR surveys two times in each roadway under similar conditions in order to ensure the accuracy of the collected data. Therefore, the reported pavement
layer thicknesses are based on the average values between the two operated tests. Additionally, in order to properly assess the pavement layer thicknesses on one hand, and acquire comprehensive information regarding the type and material of the layers on the other, it deems necessary to validate the GPR testing data by comparing the collected results with other available TxDOT databases and interviews with District engineers.

4.4.1.3. Validation of the GPR Data Collected in the Network

The research team conducted data mining from available databases (i.e. PMIS, intranet resources), pavement design plans, and communication with TxDOT personnel to confirm the pavement profiles of the studied sections.

To further explain this process, the GPR measurements from SH 72 are used as an example of measurements where post-processing required further verification. Figure 4.4 shows the GPR collected data for the specified location. Analysis of the GPR image showed a shallow interface corresponding to a 1.5 in. overlay on top of an HMA layer with a thickness that varies between 4 in. to 6 in., with an average of 5.1 in. However, from such image, the base-subgrade interface was not discernible. To verify the available and complement the missing information, the research team proceeded to contact the Districts to request pavement design plans and conducted an extensive search within the available databases.
Figure 4.4: GPR Data from State Highway 72, Corpus Christi.

- **Pavement Design Plans**

For the described case, the research team thoroughly reviewed the pavement design plans, corresponding to Reference Marker 536 of SH 72, located in Karnes County. Figure 4.5 shows the proposed design plans and specifications as of November 2017. From the design plans, State Highway 72 was designed to have a 1.5 in. asphalt overlay, 5 in. of HMA, 16 in. of 1% cement treated base (CTB) constructed over 8 in. of 3% cement treated subgrade (CTS). Thus, the overlay and HMA layer thicknesses obtained from post-processed GPR data were found to be in good agreement with the pavement design. In this case, the design plan further supplemented the base thickness missing from the GPR measurements.
The research team also consulted the PMIS database to verify and cross-validate the pavement profile information. For the case used as example, the PMIS data records associated to SH 72, RM 536, in Corpus Christi District are shown in Figure 4.6. The PMIS records indicate that such section corresponds to Pavement Type 5. According to the PMIS Rater’s Manual (TxDOT, 2016), Pavement Type 5 is defined as a pavement with medium thickness of AC, i.e. 2.5 - 5.5 in. This was found to be in line with our obtained GPR results. Appendix B also provides the information extracted from the PMIS database for all ten selected overload corridors.
• **TxDOT’s Personnel Interview**

The research team also communicated with TxDOT’s District personnel to verify and/or supplement layer configuration information. For the case explained here, a shallow layer as seen by the GPR could be attributed either to an overlay or to the presence of moisture underneath the pavement surface. Communicating with TxDOT’s maintenance supervisor confirmed the presence of an asphalt overlay in the SH 72 pavement structure.

• **TxDOT’s Intranet Resources**

TxDOT provided us access to a number of Intranet resources, which were considered as integral components to tackle the project tasks. For this reason, the research team comprehensively reviewed the available TxDOT intranet resources to gather information pertaining to the pavement structures. For the case described in this section, it was found in Technical Memorandum 14-06 (Sebesta, 2014) that a section along SH 72 in Karnes County was redesigned in 2014 to address the severe distresses it experienced due to the energy development traffic. The previous pavement structure consisted of 3.5 in. of HMA over 10.5 in. of a lightly cement stabilized base over a moderate to high PI clay subgrade.

Figure 4.6: PMIS Information associated with the Pavement Type of SH 72 in Corpus Christi (from PMIS Database, 2010).
The severe pavement failure on this section of SH 72 was primarily attributed to disintegration of the upper part of the existing treated base by the heavy energy related truck traffic. The tech memo recommended SH 72 a 5 in. (at least) HMA surface layer in combination with a cement stabilized base layer. Moreover, Research Report RR-16-02 (Gurganus, 2016), recommended the use of an 8-in. base layer in Corpus Christi District. This information is in agreement with the GPR data obtained by the research team on SH 72.

Ultimately, Table 4.3 indicates the pavement layer thicknesses obtained from GPR results and validated by available information and databases. The pavement layer thicknesses and configurations obtained from GPR were further incorporated to back-calculation layer moduli from FWD data. Following section provides the information pertaining to the FWD testing results.
<table>
<thead>
<tr>
<th>Roadway Information</th>
<th>Pavement Design Plans</th>
<th>PMIS Database</th>
<th>TxDOT's Personnel Interview</th>
<th>TxDOT's Intranet Resources</th>
<th>GPR Results (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PMIS Definition</td>
<td></td>
<td></td>
<td></td>
<td>Overlay</td>
</tr>
<tr>
<td>District</td>
<td>County</td>
<td>Roadway</td>
<td>TRM</td>
<td>Information</td>
<td>Pavement Type</td>
</tr>
<tr>
<td>Laredo</td>
<td>Dimmit</td>
<td>US 83 (SB Lane)</td>
<td>Near 638</td>
<td>1.5 in. Overlay</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>La Salle</td>
<td>FM 468 (WB Lane)</td>
<td>440</td>
<td>2.5 in. HMA, 6 in. Base</td>
<td>10</td>
</tr>
<tr>
<td>San Antonio</td>
<td>McMullen</td>
<td>FM 624 (WB Lane)</td>
<td>500</td>
<td>7.5 in. Base layer</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>McMullen</td>
<td>FM 99 (SB Lane)</td>
<td>588</td>
<td>-</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>Atascosa</td>
<td>SH 16 (SB Lane)</td>
<td>642</td>
<td>-</td>
<td>10</td>
</tr>
<tr>
<td>Corpus Christi</td>
<td>Live Oak</td>
<td>US 281 (NB Lane)</td>
<td>622</td>
<td>2 in. Overlay 4-6 in. HMA 18 in. Base</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>SH 72 (WB Lane)</td>
<td>536</td>
<td>1.5 in Overlay 5 in. HMA 16 in. CTB 8 in. Subgrade Cement Treated</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>SH 123 (SB Lane)</td>
<td>Near 552</td>
<td>5.5 in. HMA 15 in. Base</td>
<td>5</td>
</tr>
<tr>
<td>Yoakum</td>
<td>Gonzales</td>
<td>US 183 (SB Lane)</td>
<td>Near 580</td>
<td>6 in. HMA 8 in. Base</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Dewitt</td>
<td>SH 119 (EB Lane)</td>
<td>Near 544</td>
<td>4 in. HMA 12 in. Base</td>
<td>5</td>
</tr>
</tbody>
</table>

<sup>a</sup>Cement Treated Base (CTB) layer
<sup>b</sup>Cement Treated Subgrade
4.4.2. Falling Weight Deflectometer (FWD)

4.4.2.1. Background

The FWD is the most commonly used equipment for evaluating structural capacity and to back-calculate the pavement layer moduli. FWD is a trailer-mounted device that delivers a transient force impulse to the pavement surface. Figure 4.7 shows the components of the FWD testing device used for the forensic studies conducted in the test sites. The testing unit consisted of a Dynatest FWD with a load pulse of ~28 ms. Seven deflection sensors (geophones) were used to measure the deflection bowl caused by the impulse load, as shown in the Figure 4.7 (c) and (d). The deflections obtained from the seven geophones were input into a back-calculation program to determine the layer moduli of the pavement structure.
4.4.2.2. **FWD Testing in the Eagle Ford Shale Region**

The research team collected FWD data associated with the studied pavement sections to identify the layers moduli. Figure 4.8 shows the FWD setup that the research team arranged for data collection. Essentially, it was 3 drops per 3 test spots (at a 25’ distance) in the right wheel path to consider the load-induced damage on back-calculated material properties. Same testing pattern followed in between both wheel-paths to mitigate the effect of possible systematic errors incurred in the back-calculation procedure. A variable load level scheme ranging from 6,000 lb. to 12,000 lb. imparted through a 12 in. diameter spring loaded plate on pavement surfaces.
To back-calculate the modulus value, an iterative scheme is used to calculate theoretical deflections by varying the material properties until an acceptable match of measured deflection is obtained. This is achieved using MODULUS 7 program in this research effort.

It should be noted that the accuracy of the back-calculated layer modulus are highly linked with three main input parameters as follows:

- **Pavement Layer Thicknesses:** Accurate assessment of the layer thicknesses attributed to the multi-layer pavement systems is of outmost importance for the proper determination of the back-calculated layer modulus. This information was incorporated in the software as the initial step of the back-calculation procedure.

- **Seed Modulus:** The results of the iterative back-calculation procedure heavily depends on the initial seed value provided by the user. This makes the back-calculation process complex, since the programs cannot think for the user, and the process of arriving at a rational moduli value requires engineering judgment and comprehensive evaluation of all available data (Mehta et al., 2003). In practice, engineers and researchers select seed values...
considering the particular region, pavement design, material type, and climatic condition.

- **Depth to Bedrock and Subgrade Modulus:** Under TxDOT Research Project 0-1175 (Chang et al., 1992), the researchers investigated the importance of depth to bedrock in the accuracy of the FWD back-calculation procedure. The study attested that the value of depth to bedrock can significantly influence on the back-calculated layer modulus values. It was also found that a small error in the estimation of the subgrade elastic modulus would lead to large errors in the back-calculated elastic moduli of other pavement layers, as the subgrade can contribute up to 60% of the surface deflection under the center of the applied load.

The FWD measurements from FM 624 roadway, are used as an example to showcase the back-calculation procedure. Figure 4.9 shows the deflection basins diagram attributed to FM 624 roadway (for different loads) obtained from the seven FWD geophones.

![Deflection Basins from FM 624, San Antonio.](image)

Figure 4.9: Deflection Basins from FM 624, San Antonio.

Figure 4.10 (a) illustrates the back-calculation program environment used to analyze the FWD deflection data. The GPR post-processed results were incorporated to the program, which considered as the main input parameters. Analysis of the GPR data indicates that the Farm to
Market 624 is consisted of 1.3 in. of seal coat as the surface layer and 8.9 in. of granular base layer. Subsequently, using engineering judgment considering the roadway type, visual inspection of the sites, and reviewing the quoted typical range of moduli in the literature, the research team defined the seed modulus range in the program, as shown in Figure 4.10 (a). Furthermore, the depth to bedrock was automatically calculated by the program. Ultimately, running the software, the back-calculated layers modulus were obtained and a summary report was provided. Figure 4.10 (b) shows the snapshot of FM 624 back-calculation output results.

(a)

(b)

Figure 4.10: Back-calculation Layer Moduli for FM 624 in San Antonio (a) Modulus Program Input, and (b) Back-calculation Results.

Based on the results, it was found that the 8.9 in. granular base layer in FM 624 is relatively weak as all of the stiffness modulus values for this layer hit the lower allowed threshold of 20 ksi.
in the back-calculation procedure. It was also found that the FM 624 consisted of subgrade layer with low structural capacity (6 ksi) and surface layer with 358 ksi.

It should be noted that in order to properly evaluate the effect of seasonal variation and the climatic conditions on pavement stiffness properties, the research team conducted FWD test in both summer and winter seasons for selected corridors in the network and the obtained results are presented in the following section.

4.4.2.3. *Seasonal Climate Variation Effect on Modulus Value*

Temperature and moisture content are the prominent factors that affect the stiffness and strength properties of multi-layer pavement systems. Considering the fact that the HMA is viscoelastic in nature, its strength is greatly dependent on the temperature (Farahi et al., 2021; and Cloutier et al., 2021). Additionally, for granular base and subgrade layers the stiffness properties are highly connected with the moisture condition. Figure 4.11 illustrates the effect of moisture on soil particles. The change in stiffness is related to the state of moisture tension in unsaturated soils, also known as soil suction (Chandra et al., 1989). Soil suction is made up of two components:

1. Osmotic suction due to dissolved contaminants in the pore water, and
2. Matric suction due to the attraction between water soil particles.

The latter component is a negative pressure that exists in the soil water as a result of capillary tension. Soil suction is a measure of the soil’s affinity for water and indicates the intensity with which it will attract water. The drier the soil, the greater the soil suction (Chen, 1988) and the stiffer the material owing to the greater capillary tension holding the soil particles together.
Figure 4.11: Effect of Moisture on Soil Particles.

Figure 4.12 illustrates the annual average temperature in San Antonio located in the Eagle Ford Shale region. As evidenced in this plot, the average temperature in summer season is 38% more than in winter. Moreover, as shown in Figure 4.13, average monthly precipitation, which is connected with potential moisture ingress in pavement layers, in summer season is approximately 138% higher than the winter season. Due to the significant differences in precipitation patterns and temperature regimens in summer and winter seasons, the materials properties are also appreciably different. For this reason, the research team devised a plan to perform FWD testing in both summer and winter times to capture the variation of the damage factors in different seasons of the year.

Figure 4.12: Annual Average Temperature in San Antonio, Texas (WeatherSpark Website).
Figure 4.13: Average Monthly Precipitation in San Antonio, Texas (RSS Weather Website).

The site-specific back-calculated pavement layer modulus of all ten selected sites attributed to the summer and winter seasons are presented in Table 4.4. Additionally, to further clarify the seasonal variations of the back-calculated modulus values of pavement sections, the analysis results were contrasted and classified for the various roadway types, as indicated in Table 4.5. The results show that the summer-based layer modulus values are significantly lower than the winter-based values. This is primarily attributed to the viscoelastic behavior of the asphalt layers and the variations of stiffness properties of granular layers due to changes in the saturation state of the unbound granular layers due moisture infiltration or evapotranspiration during the service life of pavements, leading to softening of the surface layers due to elevated temperatures in summer seasons.
### Table 4.4: Pavement Layer Modulus attributed to the Representative Roadways in the Eagle Ford Shale Network

<table>
<thead>
<tr>
<th>Selected Roadways in Eagle Ford Shale Network</th>
<th>Pavement Layer Modulus in Summer Season (ksi)</th>
<th>Pavement Layer Modulus in Winter Season (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>District</td>
<td>TRM</td>
<td>Overlay</td>
</tr>
<tr>
<td>Laredo</td>
<td>Dimmit</td>
<td>US 83  (SB Lane)</td>
</tr>
<tr>
<td></td>
<td>La Salle</td>
<td>FM 468  (WB Lane)</td>
</tr>
<tr>
<td>San Antonio</td>
<td>McMullen</td>
<td>FM 624  (WB Lane)</td>
</tr>
<tr>
<td></td>
<td>McMullen</td>
<td>FM 99  (SB Lane)</td>
</tr>
<tr>
<td></td>
<td>Atascosa</td>
<td>SH 16  (SB Lane)</td>
</tr>
<tr>
<td>Corpus Christi</td>
<td>Live Oak</td>
<td>US 281  (NB Lane)</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>SH 72  (WB Lane)</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>SH 123  (SB Lane)</td>
</tr>
<tr>
<td>Yoakum</td>
<td>Gonzales</td>
<td>US 183  (SB Lane)</td>
</tr>
<tr>
<td></td>
<td>Dewitt</td>
<td>SH 119  (EB Lane)</td>
</tr>
</tbody>
</table>

<sup>a</sup>: Cement Treated Base, <sup>b</sup>: Cement Treated Subgrade, and <sup>c</sup>: Subgrade Soil
Table 4.5: Back-calculated Pavement Layer Modulus attributed to the Summer and Winter Seasons for Different Roadway Types

<table>
<thead>
<tr>
<th>Roadway</th>
<th>Layer Modulus in Winter (ksi)</th>
<th>Layer Modulus in Summer (ksi)</th>
<th>Percent Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC Base Subgrade</td>
<td>AC Base Subgrade</td>
<td>AC Base Subgrade</td>
</tr>
<tr>
<td>FM</td>
<td>565 24 9</td>
<td>419 21 6</td>
<td>26 12 27</td>
</tr>
<tr>
<td>SH</td>
<td>718 45 12</td>
<td>522 42 10</td>
<td>27 7 14</td>
</tr>
<tr>
<td>US</td>
<td>895 57 12</td>
<td>550 50 8</td>
<td>39 12 33</td>
</tr>
</tbody>
</table>

4.5. **Summary of the Major Points**

This chapter presented the post-processed results of Non-Destructive Tests (NDT) and pavement condition surveys conducted at representative sites in the Eagle Ford Shale region. Evaluation of the GPR measurements validated by TxDOT databases and available historical information provided the site-specific pavement layer configurations and layer thicknesses. It was found that in the studied network, FM roadways, on average consisted of only 1.6 in. of treated surface layer, however, representative pavement sections of SH and especially US highways were found to be more robust, with an approximate average asphalt layer thicknesses of 4.8 in. and 6.3 in., respectively.

FWD testing was also conducted to determine the back-calculated layer moduli in the surveyed network. The research team devised a plan to perform FWD testing in two different seasons (summer of 2018 and winter of 2019), to account for the seasonal variations of the back-calculated modulus values of pavement sections. Analyzing the FWD data for summer and winter seasons indicated that the back-calculated layers modulus values in summer were substantially lower compared to winter time modulus values from nondestructive testing of pavement sections.
Chapter 5: Characterization of Traffic Distribution and Loading Conditions

5.1. INTRODUCTION

The main objective of this chapter is to outline the instrumentation efforts to collect the WIM data, and the development of the Axle Load Spectra (ALS) data for representative sites in the energy development areas and OW corridors of South Texas. The developed ALS, in combination with the recent SHL permit records, will be synthesized to compile comprehensive databases on traffic distributions and loading conditions for the selected sites in the surveyed network, which will be discussed further in the succeeding chapters.

In order to develop the axle load spectra, the research team collected the necessary traffic information by deploying Portable Weigh-In-motion (P-WIM) devices to ten selected roadways. Utilizing the P-WIM devices, the research team collected information pertaining to the Gross Vehicle Weight (GVW), axle weights, vehicle classification, axle configuration, traffic volume, and vehicle speed. Additionally, the research team focused on identifying the truck traffic, its distribution in the highway network, and the detrimental effect of overload traffic on the transportation infrastructure. Figure 5.1 illustrates a map of the locations of the ten selected sites throughout the different Districts in the Eagle Ford Shale region.

Figure 5.1: Selected sites in overload corridors for field testing.
5.2. **BACKGROUND**

5.2.1. **Weigh-In-Motion (WIM) Systems**

The primary purposes of weigh-in-motion (WIM) systems are (1) to record truck weights or axle loads for road analysis, (2) to screen trucks as a part of commercial vehicle weight enforcement operation and (3) to use weight information to calculate tolls on toll roads, bridges or tunnels. For research purposes, the collected data can be used for planning of roadways, road repairs, and maintenance, and to reduce traffic and its consequences (traffic congestion, accidents, etc.). A typical WIM system consists of four components: a processor and data storage unit, vehicle classification system, user communication unit, and relevant weight sensors. WIM technology allows measuring the dynamic tire forces of a moving vehicle to estimate the corresponding tire loads of the static vehicle.

WIM devices are commonly categorized based on their portability by three categories: permanent, semi-permanent, and portable systems. Permanent WIMs collect and analyze data exclusively at a single, fixed location. Semi-permanent systems have sensors built into the pavement but their data acquisition system can be disconnected and used at a different instrumented location. Portable devices can be moved as a whole for use at different locations. Weigh-in-motion systems can be categorized based on use and speed as described in Table 5.1. There are several factors that contribute to the accurate measurement of the traffic information using WIM devices.
Table 5.1: Classification of WIM Technology Based on Speed

<table>
<thead>
<tr>
<th>WIM Category</th>
<th>Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Speed Weigh-in-Motion (HS-WIM)</td>
<td>Data collection performed under normal traffic speed. No disturbance of traffic flow. Accuracy =15%. Overloaded vehicles diverted to the checkpoint.</td>
</tr>
<tr>
<td>Low-Speed Weigh-in-Motion (LW-WIM)</td>
<td>Speed restriction to minimize dynamic effects.</td>
</tr>
<tr>
<td>Bridge Weigh-in-Motion (B-WIM)</td>
<td>Use existing bridge to weigh vehicles via measurement of the structural response of the bridge while vehicle crossing.</td>
</tr>
</tbody>
</table>

5.2.1.2. Permanent WIM Stations

Permanent Weigh-In-Motion (WIM) stations are typically used for collecting accurate weight data and traffic volume. Axle load sensors are embedded in the pavement perpendicular to the direction of the traffic flow. Installation of WIM sensors in permanent WIM stations are divided into two groups from the installation method point of view (Burnos and Rys, 2017):

- Sensors installed in a small cut in the pavement at a depth of 1 to 4 in. (2 to 10 cm). In this case, the sensor does not have direct contact with the vehicle wheel and the axle load is transmitted to the sensor by the pavement and installation grout (which is used to fill up the groove). Polymer and piezo-ceramic sensors are mounted using this installation method.

- Sensors installed in the pavement, flush with the pavement surface. In this case, the sensor has direct contact with the vehicle tire. Bending plate, quartz, and capacitive sensors are mounted using this installation method.

Figure 5.2 illustrates different types of sensors and their corresponding installation method for instrumenting permanent and semi-permanent WIM stations.
As the fulcrum for the sensor is the pavement regardless of the mounting method, Burnos and Rys (2017) evaluated the effect of pavement properties on the WIM system. The researchers assumed that the pavement, by itself, is an integral part of the weighing system; as the structural integrity of the pavement and the installation grout influence the weight measurements.

Despite the comprehensive data that can be extracted from the stationary WIM stations, the upfront installation funds and prohibitive maintenance costs are major challenges of such systems. According to Refai et al. (2014), permanent WIM installation could cost more than $200,000 per site, while static weight stations cost could exceed $800,000 per site. Figure 5.3 shows the active permanent WIM stations across Texas.
5.2.1.3. **Virtual WIM (V-WIM) stations**

This system is the combination of WIM technology and Automatic Vehicle Identification (AVI) systems with a camera and Optical Character Recognition (OCR) software (Walubita et al., 2014). Similar to permanent WIM stations, the instrumentation of V-WIM stations involves axle load sensors embedded in the pavement perpendicular to the traffic flow direction. The technology for measuring axle loads is identical to the permanent WIM stations. According to Walubita et al. (2014), there are typically two setup options associated with this system as shown in Figure 5.4 (1) the system is connected with a digital warning signpost that instructs vehicles in violation to exit the highway, and (2) the system wirelessly transmits the data to an enforcement agency/agent.
5.2.1.4. Portable Weigh-in-Motion

Portable WIM systems as shown in Figure 5.5, are often preferred over the traditional permanent WIM stations due to the convenience, and the flexibility that they provided for the characterization of the traffic information. The lower cost associated with the temporary installation and maintenance of portable WIM systems has made them a viable option for the traffic data collection. There are several challenges for the use of portable WIM devices in the field. The most noteworthy anomaly is associated with the pavement roughness. The reliability of the collected data is greatly influenced by the vibrations generated due to the slope variance, surface cracks, rut depths, and patches on the pavement surface. Moreover, the WIM systems mounted above the surface also results in an additional dynamic motion. Such noises can potentially compromise the reliability of the static weight estimations based on dynamic measurements (Sridhar, 2008). Additionally, the flexible nature of the tire results in the adsorption horizontal force, which further adversely impact the accuracy of the results. Nonetheless, lower operating costs and ease of use makes the portable WIM systems a useful means for the collection of traffic information.
5.2.2. Sensor Technologies in WIM Systems

5.2.2.1. Strain Gauge Bending Plates

The bending plate device is used for traffic monitoring applications, overload detection, and data collection. It is a scale that is composed of two steel platforms that typically measured 2 ft by 6 ft and cover a 12-ft lane (Mohammad et al., 2005). These bending plates shown in Figure 5.6 use strain gages that measure tire load-induced strains that are analyzed to determine the tire load (Mohammad et al. 2005). Based on the manufacturer’s information, such bending plates can weigh vehicles traveling between 5 km/h to 200 km/h and have a typical lifespan of more than 10 years.

Figure 5.6: IRD-Pad Bending Plate System (from International Road Dynamics Inc., n.d).
1.1.1. Hydraulic Load Cell

These type of certified load cells shown in Figure 5.7 are essentially large scales primarily used by truckers for self-weighting. They are one of the most accurate systems for collecting weight data however, they cannot record dynamic weight, are very expensive, require high maintenance cost, and have to be overhauled every 5 to 6 years.

Figure 5.7: Hydraulic Load Cells (from Cardinal Scales. n.d.)

1.1.2. Piezoelectric Sensors

Typical piezoelectric sensors consist of a copper strand that is surrounded by piezoelectric material and that is usually covered by a copper sheath or other material. Piezoelectric sensors measure the deformation induced by tire loads on the pavement and convert it to a charge that is equivalent to deformation. These piezoelectric sensors shown in Figure 5.8 can be affixed to pavement surface with conveyor belts, high strength tape, or metal fixtures. However, it is more common to embed them in the pavement by making a small groove on the surface, 1 to 2 in deep by 1 to 2 in wide, and cover them with resin (Mohammad et al., 2005). The installation procedures usually take less than a day however, once installed the piezoelectric sensors are left permanently in pavement. In addition, these sensors are able to record vehicles traveling at normal highway speeds.
As elaborated in this section, the P-WIM systems are preferred over WIM stations due to their convenience, cost-efficiency, and flexibility for continuous collection of the traffic data without interrupting the traffic flow in heavily trafficked highways. Additionally, the P-WIM units are capable of collecting reliable and accurate traffic data provided that a verified calibration procedure is implemented upon installation of the piezoelectric sensors in the field. Hence, several researchers favored P-WIM systems over the traditional stationary WIM devices in order to collect the traffic information. (1, 2). This provides the rationale behind deploying the P-WIM systems for proper characterization of the traffic loading conditions in this study.

5.3. **PORTABLE WEIGH-IN-MOTION (P-WIM) OBTAINED DATABASE**

The P-WIM devices were deployed to collect the site-specific traffic database in the representative sites during summer and winter months to capture the seasonal effect of traffic variations. The following sections provide the relevant information on the field data collection efforts, PWIM-system setup, P-WIM calibration procedure, development of the axle load spectra, and characterization of the OW and SHL vehicles captured in the field trials.
5.3.1. P-WIM Equipment

For the primary data collection equipment, the research team selected the portable traffic recording system (TRS) unit from International Road Dynamics (IRD) to be in compliance with previous research efforts conducted by TxDOT. The TRS unit consists of a controller, piezo input box, piezo-electric sensors, and their protective cover, as seen in Figure 5.9. The TRS unit is the main data logger that records the traffic information from the sensors placed on the road. The type of sensors used in this study were the Roadtrax BL Class I piezoelectric sensors which were installed on the road using a specialized pocket tape. This tape is used to affix the sensors to the pavement surface and allows the sensors to be easily removed and reused at another site if still serviceable.

Table 4.2 shows the equipment details and layout of the sensors. The equipment utilized was the most advantageous in this study due to its cost-effectiveness, minimum installation time, and portable convenience. In contrast, permanent WIM stations typically have higher installation cost and maintenance requirements that makes them financially challenging to operate in a continuous manner. Furthermore, they require extensive installation efforts due to the small trenches that must be cut in the pavement to permanently place the sensors, inductive loops, or weight pads on the roads. The pavement damage on one hand and the traffic control requirements as well as the user delays are other disadvantages of such systems. Additionally, there are favorable scholarly publications by researchers in other states, such as Faruk et al. (2016) and Lubinda et al. (2019), regarding the reliability of the acquired traffic distribution and classification data using the portable TRS WIM units.
Figure 5.9: Field Equipment from Left to Right: TRS Controller, Piezo Input Box, 4in. Pocket Tape, Piezo-electric Sensors, Splice Protective Cover.

Table 5.2: Portable WIM Equipment Details

<table>
<thead>
<tr>
<th>WIM Equipment Used in Data Collection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data Logger</td>
</tr>
<tr>
<td>Type of Sensor</td>
</tr>
<tr>
<td>Sensor Placement</td>
</tr>
<tr>
<td>Lane Coverage</td>
</tr>
<tr>
<td>Sensor Length</td>
</tr>
<tr>
<td>Sensor Layout</td>
</tr>
<tr>
<td>Additional Devices Used</td>
</tr>
</tbody>
</table>

5.3.2. P-WIM Field Installation

The P-WIM devices were deployed in selected FM, SH, and US roadways to collect the site-specific traffic information. The P-WIM units were temporarily installed at each location and were left to continuously record traffic information for at least two weeks in each site. The process was repeated for both summer and winter times to capture the seasonal effect of traffic variations. The field installation consisted of two piezoelectric sensors inserted into specialized 4 in. pocket tapes that are adhered to the pavement surface. The tapes with the inserted sensors are placed a predetermined distance, in our case 8 ft apart from one another, and connected to the main data acquisition system as shown in Figure 5.10. The sensors are installed to essentially just register
one-wheel path; nonetheless, they nearly extend the entire length of the lane to account for wheel wander. The WIM unit automatically converts the data for 1-wheel path and translates it into the total axle weights and GVW by using an internal subroutine.

For highways with multiple lanes in one direction, the piezo-sensors were installed in the outside lane of all highways, where the majority of the truck traffic travels. Moreover, during summer installations the research team had no problem installing the sensors because of the high temperature of the pavement surface. However, for winter installation the research team had to heat up the tapes and the roads surface with a heat torch to allow proper adherence between the tape-pavement interfaces, as show in Figure 5.11.

![Figure 5.10: Typical Portable WIM Equipment Setup.](image)

![Figure 5.11: Portable WIM Field Installation in the Winter Time.](image)
5.3.3. P-WIM Field Calibration

The research team successfully developed an effective method to properly calibrate the piezoelectric sensors to obtain accurate and reliable weigh-in-motion readings. The following steps describe the calibration algorithm implemented in this research project:

1. Using static scales, weigh the axles and GVW of the reference Class 6 and Class 9 trucks.
2. Drive the reference vehicles over the installed sensors.
3. Vehicle passes are conducted at different speeds. Usually a minimum of 5 passes at 3 speeds (+/- 15 mph) the Speed Limit.
4. Adjust the calibration factors until the recorded dynamic weight matches the static weight.
5. Continue adjusting the Calibration Factor until the recorded weights are within an acceptable tolerance.

This process was implemented at the time of installation and removal of the WIM devices. Calibration of the WIM systems were conducted at every test site before and after the collection period to optimize the accuracy and reliability of the WIM data. For this purpose, Class 6 trucks were selected as calibration vehicles due to their accessibility across all TxDOT Districts. Whereas Class 9 trucks were also selected due to their high frequency in the highway network. Therefore, both Class 6 and Class 9 trucks were used in the calibration procedure. Initially, the gross vehicle weights (GVW) and axle weights of these reference vehicles were measured using portable static axle scales and recorded as illustrated in Figure 5.12. The static weight of a fully loaded Class 6 dump truck typically ranged between 40 and 55 kips, and the loaded Class 9 truck typically ranged between 70 and 88 kips. The recorded static weights were then used as the target weight during the system calibration. The calibration runs were then conducted using with both reference vehicles while changing the vehicle speeds as shown in Figure 5.13. Finally, a calibration factor
was then applied to the data until the target weight was within an acceptable tolerance. In addition to the pre-calibration procedures performed, the research team also conducted post-calibration on the piezo-sensors to ensure sensor functionality and WIM data quality.

Figure 5.12: Static Axle Weight Measurements: (a)Class 6 Dump Truck (b) Class 9 Water Truck (c) Class 9 Belly Dump (d) Static Axle Weight Using Portable Scales.
5.3.4. P-WIM Sensor Life

The reliability and quality of the traffic data collection is paramount to the accuracy of the predicted damages in the proposed framework. Several factors contribute to the accuracy and reliability of the WIM data collections such as pavement condition, surface distresses, surface temperature, environmental conditions, and the field calibration procedure. However, based on the research team’s experience in relevant projects the operational service life of the piezo-sensors greatly influence the quality of the WIM achieved traffic data. One way to assess the performance of the sensors is by analyzing the deterioration of the calibration factors over the operational life of the installed piezo-electric sensors in the field. Figure 5.14 shows the variation of the calibration factors for several sites in San Antonio, Laredo, Corpus Christi, and Yoakum districts. The results pertaining to the sensors installed for over 50 days in the State Highway 123-80 in Corpus Christi
(CRP-123-80) provides valuable insights on the longevity and service life of the piezo-electric sensors in OW corridors of Eagle Ford Shale region.

Figure 5.14: Site-Specific calibration factors applied.

5.4. **AXLE LOAD SPECTRA**

The developed axle load spectra database was the compilation of P-WIM data collected at ten sites strategically distributed throughout the Eagle Ford Shale Region. The data collection was conducted in two-time intervals, summer and winter, to capture the effect of seasonal traffic variations, as well as the effect of environmental conditions on the damage quantification. Moreover, the WIM units were left in the field continuously collecting data for a two-week time period per site. After sensor installation and data collection, the raw traffic data were compiled and analyzed to produce the following general traffic information and traffic inputs listed in Table 5.3.
Table 5.3: Collected Traffic Data Using Portable WIM

<table>
<thead>
<tr>
<th>Traffic Volume</th>
<th>Weight</th>
<th>Others</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Daily Traffic (ADT)</td>
<td>Gross Vehicle Weights (GVW)</td>
<td>Vehicle speed</td>
</tr>
<tr>
<td>Average Daily Truck Traffic (ADTT)</td>
<td>Axle Weights</td>
<td>Axle Type</td>
</tr>
<tr>
<td>Percent Truck</td>
<td>Axle Load Distribution</td>
<td>Axle Spacing</td>
</tr>
<tr>
<td>Vehicle Class Distribution</td>
<td>OW Vehicles</td>
<td>Wheelbase</td>
</tr>
<tr>
<td>Truck Class Distribution</td>
<td>Super Heavy Load Vehicles</td>
<td>Axle Config.</td>
</tr>
<tr>
<td>Hourly Distributions</td>
<td>Super Heavy Load Axles</td>
<td>Time</td>
</tr>
<tr>
<td>Daily Distributions</td>
<td>Average of Ten Heaviest Wheel Load (ATHWL)</td>
<td>Date</td>
</tr>
</tbody>
</table>

To develop the axle load spectra database the research team first extracted the raw traffic data from the TRS units and applied post-calibration factors. Then the data was processed and classified by axle type (Steering, Single, Tandem, Tridem, and Quad) and load intervals and then formatted as axle load distribution (ALD) input files to be compatible with the TxME and Pavement ME software. This information not only serves for ME pavement design purposes but also aids in traffic characterization, highway planning data, OW/oversize documentation, SHL identification, and pavement damage quantification. Ultimately, the collected information was established to create a comprehensive traffic database for US highways, State highways, and Farm to Market roads of energy developing areas in the Eagle Ford Shale Region. The developed database provides the primary source for characterization of the SHL vehicles operating in the surveyed network.

5.4.2. Truck Class Distribution

The vehicle class distribution function was one of the most essential pieces of traffic information collected by the P-WIM units. This information is a direct input to the remaining service life analysis, and has a significant influence on the final result. From the collected traffic data, the most prevalent truck classes identified in all ten highways were Class 5 and Class 9 trucks as highlighted.
in Figure 5.15. Class 5 trucks have a steering axle and rear axle with dual tires, they are typically associated as small delivery trucks such Penske and U-Haul trucks. While Class 9 trucks have a single steering axle and two tandem axles in the back. The types of Class 9 trucks can range significant as they can be used to transport finished goods, oil, gasoline, equipment, and etc.

Figure 5.15: FHWA Vehicle Classifications.

Figure 5.16 illustrates the truck class distributions comparisons versus percent trucks for all the different highway types (FM, SH, US). In all different roadway types, Class 9 was the most common truck followed by Class 5. Farm-to-Market (FM) roads generally tend to have a higher number of Class 5 delivery trucks compared to the state highways (SH) and US highways. In contrast US highways had the highest Class 9 and fewest Class 5 trucks. Moreover, Figure 5.17 illustrates the truck class distributions among all the P-WIM sites in the Eagle Ford for comparison purposes. These results are in agreement with the trends found in the literature.
Figure 5.16: Truck Class Distributions: (a) FM Highways (b) SH Highways (c) US Highways in the Eagle Ford Shale.
Figure 5.17: Truck Class Distributions of All P-WIM Sites in the Eagle Ford Shale.

5.4.2.2. **Truck Misclassifications**

Portable WIM units are characteristically very reliable at collecting and classifying vehicular traffic. Figure 5.18 illustrates the typical error classifications associated with each type of truck class. As illustrated, these misclassifications errors tend to be very low. The classes of trucks with the highest error percentage are Class 4 and Class 7. Class 4 is a predominant characterized by buses, and therefore, new bus configurations could be the source of that error. While Class 7 trucks are typically characterized by dump trucks with multiple rear axles that can be deployed when needed; they often have a lift axle that could mislead the algorithm the WIM unit uses. Other sources of errors can also be attributed to pavement surface imperfections, traffic driving over the sensor splice, damaged sensors, or unconventional truck configurations.

Figure 5.19 illustrates the truck class distribution of US 83 in the Laredo District. This plot shows the truck class distribution of the summer and winter collection period and the differences are almost imperceptible. Despite two different data collection times, the portable WIM unit
classified the incoming traffic appropriately.

![Figure 5.18: Truck Class Distributions Misclassification Error.](image)

![Figure 5.19: Seasonal Variation of Truck Class Distributions.](image)

### 5.4.3. Gross Vehicle Weight (GVW) Distributions

Two types of GVW distributions were primarily observed throughout all the WIM sites. The first type of distribution is illustrated by Figure 5.20, which shows the gross vehicle weight distributions with respect to truck percentage for all classes of trucks in US 281 of the Corpus Christi District. This plot can be characterized by a bimodal distribution that is attributed to unloaded trucks in the 20,000 lb. to 44,000 lb. range and loaded trucks in the 76,000 lb. to 100,000 lb. range. These
GVWs coincide with historical WIM data throughout Texas and the current LTPP data. However, it is important to note that the GVW distributions are shifted past the 80,000 lb. weight limit due to the large number of overloaded trucks this highway accommodates. Another metric that is often employed is the minimal gross vehicle weight analysis, in this check little to no trucks should be present at the 10,000 lb. or less weight interval. Figure 5.20 shows only 4% of the data in that range. Nonetheless, truck percentages in these range are acceptable because it is known that Class 5 trucks tend to be light vehicles that peak at that the 10,000 lb.-12,000 lb. weight interval.

![Figure 5.20: GVW Distributions of All Trucks in US 281.](image)

Furthermore, the second type of GVW distribution that was observed was located in areas with heavy traffic operations and OW traffic. Usually, the bimodal distribution is significantly less pronounced as illustrate by Figure 5.21. Where a small peak is visible at the 12,000 lb. interval; due to the high number of Class 5 trucks. While a second peak is visible at the 36,000 lb. interval that is linked with unloaded trucks. Despite the weight distributions peaking at a lower weight range, the plot also shows the GVW distribution extending all the way to the 180,000 lb. weight interval. The typical trucks that are the most prevalent in the energy development highway network can be seen in Figure 5.22.
5.4.4. Axle Load Distributions

Tandem axles typically follow similar weight distributions to that of Class 9 GVW trucks. This information coincides, since Class 9 trucks are the most common truck in almost any highway network, and it is composed of two tandem axles. In most highways, tandem axles are also typically characterized by a bimodal distribution that is attributed the unloaded and loaded tandem axles. Figure 5.23 illustrates this bimodal distribution of unloaded tandem axles in the range of 14,000lbs to 26,000lbs, while the loaded tandem axles in the 32,000lbs to 42,000lbs range for SH 72. Figure 5.24 illustrates the tridem axle weight distributions for SH 16 which show the
distributions substantially shifted to the loaded and overloaded side. In addition, for heavy loaded highways such as the one illustrated in Figure 5.25, the quad axle distributions increase and then peaks in the 70,000lbs range, essentially showing significantly overloaded quad axles (79%).
Figure 5.23: Tandem Axle Weight Distributions for All Trucks in SH 72.

Figure 5.24: Tridem Axle Weight Distributions for All Trucks in SH 16.

Figure 5.25: Quad Axle Weight Distributions for All Trucks in SH 123 / BU 181.
5.4.5. OW (OW) Vehicle Distributions

One of the primary objectives of this study was to capture GVW and axle weight distributions of OW vehicles operating in the overloaded corridors and energy sector zones of South Texas. In the process, the researchers captured very interesting results that are quite surprising. Eight out of the ten selected highways experienced a significantly high number OW truck traffic (>10%). The Districts with the heaviest truck traffic operations were the Corpus Christi District and the Laredo District. However, despite having numerous oil/gas wells in the surrounding areas, pipeline construction, and equipment movement the highways in the Yoakum District were not as significantly affected.

Figure 5.26 illustrates the OW (OW) truck distributions of US 281 for both the summer and the winter time. The plot shows an OW distribution of 17% in the winter time and an astounding 45% OW distribution in the summer time. That is nearly half of all trucks traveling on this highway were OW, this is a significant number of OW trucks that detrimentally impacts the pavement structure especially in the summer when the stiffness properties of the asphalt layer are the weakest. The reason for the heavy truck traffic operations is due to a nearly oil refinery in Three Rivers, Texas and due to the transportation of heavy equipment as illustrated in Figure 5.27.
Meanwhile, Figure 5.28 also illustrates the OW truck distributions for FM 468 for both summer and winter time. Similarly, this plot also shows a smaller OW truck distribution for the winter at 12% compared to a 32% distribution for the summer time. The main distinction is that the summer distributions extend all the way to the 180,000 lb weight interval which is alarming for any roads, specially for FM roads that are not designed to sustain such heavy truck weights. FM 468 also had some of the heaviest truck traffic in terms of GVW. The portable WIM unit deployed at this site captured trucks weighting in excess of 250,000 lb. Evidently, this site was the most damaged and distressed site showing multiple distress types such as rutting, fatigue cracking, flushing, and pot holes among others.
Figure 5.28: OW Vehicle Distributions in FM 468.

FM 99 is a small load zoned road with a weight limit set at 58,420lbs however, as shown in Figure 5.29 that limit does not prevent the oil and gas industry from driving over this road. Figure 5.30 illustrates the OW vehicle distributions plot of FM 99 in the San Antonio District for both the summer and winter collection time. The portable WIM unit deployed at this site captured data that characterized the OW truck distributions as high as 56% for the summer time. While in the winter time as much as 63% of the truck traffic was OW. These are incredibly high percentages of OW truck traffic that detrimentally impact the pavement structures and bridges, the impacts of these heavy truck traffic operations are discussion in future sections.
Figure 5.29: Oversize/OW Loads traveling on FM 99 of the San Antonio District.

Figure 5.30: OW Vehicle Distributions in FM 99.

Figure 5.31 illustrates OW truck distributions in SH 123/ BU 181 of the Corpus Christi District for both the summer and the winter time. The P-WIM data collected looked very consisted for both time intervals, characterizing the OW distribution for the winter time at 35% while the summer truck distribution at 36%. That is nearly a third of all truck traffic in this highway being OW at any given season. In addition, GVW in excess of 360,000lbs have been recorded in this highway during the summer time. To verify the validity of the collected information the research team contacted the local TxDOT office and it was confirmed that they had issues SHL permits for super heavy trucks weighting in excess of 300,000lbs. In terms of damage quantification, a single passage of this load can be enough to impart significant damage to the pavement structure, culverts, and nearby bridges.
5.4.6. Temporal Variations of OW Truck Distributions

ALS provides hourly, daily, weekly, monthly, seasonal, and annual distribution of the classes of vehicles for mechanistic analysis and design of pavement structures. This section provides the post-processed results on the temporal variations of the OW truck traffic with respect to time (hourly, daily, and seasonal variations). Figures 5.32 and 5.33 show the hourly and daily distributions of OW trucks, respectively, for different seasons in the representative US highways. According to the plots, the OW trucks operating in the studied US highways in the summer time tend to peak at the 7:00 AM - 12:00 PM timeframe, while the traffic makeup in the winter season indicate considerably high frequencies of OW trucks in the afternoon between 12:00 PM and 3:00 PM. This could be attributed to the safety issues and cold weather conditions in the winter months that caused heavy vehicles to be moved in the afternoon.

Based on the results provided in Figure 5.33, OW trucks showed almost the same trend during the weekdays in the summer with minimal decrease during the weekends; however, there is a notable variation with respect to the operation of the OW trucks for various days in the winter.
months. According to the recorded traffic data, Wednesday accommodated the highest percentage of OW trucks throughout the week.

Figure 5.32: Hourly OW Truck Distribution in US Highways.

Figure 5.33: Daily OW Truck Distribution in US Highways.

Figure 5.34 illustrates the OW truck distributions among all FM, SH, and US highways in the Eagle Ford Shale. The trend that can be observed is that generally the OW truck distributions
are highest in the summer time with the exception of US 83 and FM 99. This trend can be attributed for a number of different reasons however; the researchers link this trend to the seasonal variation of crude oil price per barrel. The price of oil generally tends to be more expensive in the summer time therefore energy companies are more enticed to produce more barrels of oil since they have a higher return on their investment. As a result, there are more wells being drilled that generate a plethora of truck traffic even for just one drilling site. In addition, the results show that Farm-to-Market roads can have OW distributions up to 64%, state highways (SH) of up to 36%, and US highways up to 45% for highways in the surrounding network.

![Figure 5.34: Seasonal Variation of OW Trucks for All Ten Representative Sites.](image)

5.4.7. Verification of the Axle Load Spectra Databases Using Stationary WIM

5.4.7.1. Introduction

It is imperative to properly validate the reliability and accuracy of the ALS, because the ME pavement analysis is highly sensitive to the traffic data inputs. Historically proven stationary (permanent) WIM units are typically used by many state Departments of Transportation (DOTs)
to collect accurate ME traffic data. (Buchanan, 2004, Prozzi and Hong, 2005, Jiang et al., 2008, Papagiannakis et al., 2008, Mai et al., 2013, Turochy et al., 2015, Walubita et al., 2019). Despite the accurate and reliable traffic data that can be extracted from the stationary WIM stations, the upfront installation funds and prohibitive maintenance costs are major challenges of such systems (Refai et al., 2014). For this reason, considering the limited financial resources, the vast majority of the stationary WIM units are located in the vicinity of the interstates and major highways. A prime example of that can be found in Texas with 39 stationary WIMs that are predominantly located within the interstate highway’s transportation network.

Conversely, the lower costs associated with the temporary installation and maintenance of the Portable Weigh-In-Motion (P-WIM) systems have made them a viable option for collecting the site-specific traffic data, even in the rural and arterial roads, besides the major highways. Hence, the P-WIM systems are commonly preferred over the traditional stationary WIM devices due to the convenience, cost-efficiency, and the flexibility for continuous data collection without interrupting the traffic flow in heavily trafficked highways. However, obtaining accurate and reliable traffic data is the major challenge associated with P-WIM systems.

The primary objective of this subsection is to accurately assess the reliability and quality of the prominent ME traffic parameters and the ALS databases that were directly derived from deployments of the P-WIM devices in the overload corridors of Texas.

5.4.7.2. Approach to Verify the P-WIM Obtained Data

Validating P-WIM data can be cumbersome at times because researchers often don’t have the means to justify the weight readings; thus, the reliability of the data becomes questionable. The most promising approach is to compare the P-WIM data with the corresponding data recorded by an adjacent stationary WIM unit. Additionally, there are a few metrics that can give an indication
of the accuracy and reliability of the P-WIM data being collected, such as using the steering axle weight of typical Class 9 trucks.

Figure 5.35 shows the procedure established to assess the validity of the traffic data collected by the P-WIM devices. The research team comprehensively reviewed the available online databases to extract and analyze the traffic data collected by the stationary (permanent) WIM units. The major traffic parameters from P-WIMs such as Tandem axle load distributions of Class 9 trucks, vehicle class distributions, and general traffic information were juxtaposed with those values quantified by the adjacent stationary WIM units, to assess the validity of the ME traffic inputs obtained from P-WIM units. This was accomplished in US 281 highway as a representative site. It should be noted that the traffic information associated with the simultaneous data collection for a period of two weeks in August 2018 was incorporated into the described comparative analysis.

The authors also conducted a variability analysis, considering the variations of the recorded weights for the steering-axle load of Class 9 trucks as a reference traffic parameter to further evaluate the quality and accuracy of the data collected by P-WIM units. The following sections provide the relevant information on the highway site selection, traffic data collection by both WIM systems, and the rationale for comparisons of the selected traffic parameters. Ultimately, the corresponding results are presented and synthesized.
5.4.7.3. Highway Site Location for Traffic Data Verification Purposes

US 281 highway was selected as a case study, which is an extremely trafficked highway in Texas energy developing areas. The traffic data obtained from the installed P-WIM unit at US 281 was compared with the data recorded by the nearest permanent WIM station located at the same studied highway. The analysis of the number of in-service oil and gas wells, as a significant contributing factor in traffic distribution patterns in the region, showed that productions of the energy-related resources had generated relatively similar truck traffic operations, in terms of traffic volume and frequency, at the two studied sites. For this reason, traffic data captured by these two WIM stations in US 281 was evaluated for further comparative purposes in this study.

5.4.7.4. Stationary (Permanent) WIM Data

Currently, TxDOT Transportation Planning and Programming (TPP) division operates the permanent WIM stations to collect the truck traffic information in several highway sites across Texas. The obtained traffic data is incorporated into an online database, namely Traffic Count
Database System (TCDS). Hence, the authors used TCDS to extract and post-process the raw traffic data captured by the permanent WIM station in US 281 highway. Although the raw data provided extensive traffic information including 365 days of continuous data collection, the research team studied the data associated with the same period as P-WIM data collection’s to compare the traffic information obtained from the two WIM stations.

5.4.7.5. **Selected Traffic Parameters for Comparative Analysis**

Axle load spectra, axle load distributions, axle configurations, vehicle class distributions, and general traffic parameters were the most relevant traffic information that was captured/derived by the WIM units. Due to the fact that the performance of a pavement section mainly relies on the distributions of the axle weights and axle types of the vehicles passing, axle load distributions, derived from axle load spectra, provide the most desirable traffic data input for ME pavement analysis. Additionally, field observations and the post-processed traffic data indicated that the Class 9 and Tandem axles are the most prevalent truck type/axle configuration operating in the energy sector corridors of Texas. Therefore, the Tandem-axle load distribution of Class 9 trucks, which is a key component in the ALS databases, was incorporated into the comparative analysis performed in this study. Other major traffic parameters that greatly contribute to the accurate incorporation of the traffic data into the ME pavement analysis protocol, include the average annual daily traffic (AADT), average annual daily truck traffic (AADTT), percent truck, percent OW, and vehicle class distributions. Consequently, the following traffic parameters, as representatives for the entire axle load spectra, were comparatively assessed for further verification of the P-WIM obtained data:

- *Tandem-axle load distribution of Class 9 trucks*,
- *General traffic information: AADT, AADTT, percent truck, percent OW, and*
Vehicle class distribution.

5.4.7.6. Class 9 Tandem Axle Load Distributions

Figure 5.36 draws comparisons of tandem-axle load distributions of Class 9 trucks characterized using the two WIM systems in US 281 highway. As evidenced in the plot, the P-WIM and stationary WIM data indicated a similar distribution trend. Essentially, the patterns in the plots make apparent that the tandem-axle loads in both cases were characterized by a bimodal distribution, which is attributed to the peaks in the unloaded (18 kips) and loaded axles (36 kips). This is in line with our expectations since based on the research team’s observations in the field, it was concluded that in most highways, tandem axles are typically characterized by a bimodal distribution, representing the unloaded and loaded tandem axles (Ashtiani et al., 2019).

Another noteworthy observation from the plot was that the P-WIM unit tends to capture a marginally higher number of trucks with heavy axles compared to the stationary WIM unit. Considering the Texas permissible weight limits for tandem axles as 34 kips, the P-WIM device recorded an OW truck frequency of 51.2% for tandem axles of Class 9 trucks, while the corresponding OW percentage obtained from stationary WIM measurements was found to be 46.4%. Consequently, the P-WIM unit was capable of characterizing the OW tandem axles of Class 9 trucks operating in US 281 with an accuracy of approximately 89.66%, in comparison with the stationary WIM system.
5.4.7.7. **Vehicle Class Distributions**

Figure 5.37 presents the comparative results attributed to vehicle class distributions. As demonstrated in the plot, vehicle class distributions captured by the P-WIM unit were in reasonable agreement with the corresponding measurements from the stationary WIM. Although the P-WIM unit recorded slightly lower percentages for Class 5 and higher percentages for Class 9 trucks compared to the stationary WIM, in general, the frequencies associated with all truck classes captured by the two WIM systems are nearly comparable with a maximum discrepancy of 8%.

![Comparison of Vehicle Class Distributions](image)

Figure 5.37: Comparative Results associated with Vehicle Class Distributions.
5.4.7.8. General Traffic Information

A comparison of the general traffic information achieved by the WIM systems is provided in Table 5.4. The results showed that the P-WIM unit correctly recorded the traffic parameters, including AADT, AADTT, and percent truck with an absolute arithmetic difference of 2.8%, 5.0%, and 2.1%, respectively, compared to the stationary WIM unit. These such slight differences truly validate the quality and reliability of the P-WIM collected data, suggesting accuracy of up to 97.9% (i.e., 100-2.1%) of traffic data collection using the P-WIM devices. Additionally, in terms of the OW trucks violating the maximum GVW limit of 80 kips, the overall average difference between measurements of the two WIM systems was equal to 12.5%. Consequently, the comparative analyses indicated that it is practically feasible to obtain reliable traffic data with an accuracy of at least 87.5 % (i.e., 100-12.5%) with the deployment of the P-WIM units to the field.

Table 5.4: General Traffic Information: P-WIM data vs. Stationary WIM data

<table>
<thead>
<tr>
<th>Traffic Parameter</th>
<th>Portable WIM Data</th>
<th>Stationary WIM Data</th>
<th>Absolute Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADT</td>
<td>8,355</td>
<td>8,125</td>
<td>2.8 %</td>
</tr>
<tr>
<td>AADTT</td>
<td>2,022</td>
<td>1,925</td>
<td>5.0 %</td>
</tr>
<tr>
<td>Percent Truck</td>
<td>24.2 %</td>
<td>23.7 %</td>
<td>2.1 %</td>
</tr>
<tr>
<td>Percent OW Trucks</td>
<td>38.8 %</td>
<td>34.5 %</td>
<td>12.5 %</td>
</tr>
</tbody>
</table>

5.4.7.9. Variability Analysis of the Steering Axle Load Weights

As stated earlier in the methodology section, the authors also assessed the variability of the steering axle weight of typical Class 9 trucks, which is a standard metric used industry-wide that gives an indication of the accuracy of the traffic data captured by P-WIM units. Typically, the industry-standard weight for steering axles of Class 9 trucks ranges between 8 and 12 kips, with an average
of 10.5 kips as the reference value, regardless of the truck’s GVW. This information was instrumental through data validation process. Figure 5.38 illustrates the distributions of the steering axle weights of Class 9 trucks operating in US 281 highway. The results showed that vast majority of the characterized weight distributions, i.e., 93%, fall within the expected manufacture-specified range with an overall average of 10.3 kips. Additionally, the results provided in Figure 5.38 indicated that the Coefficient of Variance (COV) value of the collected data was equal to 10.5 %, which is congruent with ±15 % error percentage for axle weight measurements indicated by the equipment manufacturer. Accordingly, the variability analysis conducted in this study showed that collecting repeatable and consistent P-WIM data with an accuracy of approximately 89.5% is achievable.

Figure 5.38: Recorded Class 9 Steering Axle Weights in US 281 Highway.

Figure 5.39 illustrates the average steering axle weight of Class 9 trucks for all portable WIM sites collected in the Eagle Ford Shale. A small spread of steering axle weights can be seen in the plot which can be attributed to a number of different reasons. Nonetheless, the vast majority of the steering axle weights are within the desired range. Accordingly, collecting reliable and accurate P-WIM data is practically possible.
5.5. LIMITATIONS AND DRAWBACKS

The major limitations and drawbacks of this study are listed below:

- Monthly adjustment factors were not able to be derived from WIM data collected.
- Since the data collection was only conducted in two time periods, summer and winter, for a length of two weeks, the researchers made the assumption that the time period measured gives an accurate measurement of weights for the entire year.
- The time interval in which the data was collected could have underestimated or overestimated the current traffic and vehicle loads.
- Since the big energy boom in the Eagle Ford Shale already occurred in the 2008-2012 era, the P-WIM data collected did not contain the heaviest GVW distributions the highway could have experienced.

5.6. SUMMARY OF THE MAJOR POINTS

- The P-WIM system is a simple and inexpensive method to collect site-specific traffic data in rural locations and produce reliable and accurate results.
The results confirmed the reliability, quality, and consistency of the traffic information obtained from the P-WIM units. Accordingly, the P-WIM system is a verified and viable alternative to costly and labor-intensive stationary WIM systems for collecting accurate and reliable site-specific traffic data required for ME pavement design and analysis.

Continuous assessment of the performance of the piezo-sensors over time, as well as the implementation of a proper calibration procedure pre- and post-installation of the P-WIMs, are the key factors, among others, that contribute to obtaining reliable readings from the P-WIM systems.

The P-WIM units collected satisfactory data in this research study that is suitable for the development of the axle load spectra for energy developing areas in the Eagle Ford Shale.

The heaviest traffic operations were located in the Laredo, Corpus Christi, and San Antonio Districts with OW distributions up to 33%, 46%, and 64% respectively.

GVW greater than 360,000lbs were recorded in certain highways of the Eagle Ford Shale.

FM Highways carried significantly heavy OW traffic from the energy industry and they are the most adversely affected highways because of their overall thin pavement structure that was not designed to sustain such heavy loads.

GVW distributions fluctuate significantly according to seasonal variations and the oil price.

Due to the inherent variability of the traffic distributions patterns during different time of the day, month, and season, accurate assessment of the temporal variations of the OW truck traffic with respect to time is of outmost importance for proper quantification of the truck traffic in overload corridors.
Chapter 6: Characterization of the SHL Operations in Texas Overload Corridors

6.1. INTRODUCTION

The primary objective of this chapter is to elaborate on the traffic distributions and loading conditions specifically tailored towards the non-conventional SHL vehicles operating in overload corridors of Texas. In this subtask, the researchers provide information associated with the development of two comprehensive databases, as the primary SHL catalogues in this research project. The demanding loading conditions and the vehicle characteristics of SHL units, as extracted from the ALS database, as well as the pertinent information obtained from the most recent SHL permit records in the network, is presented in this chapter. Ultimately, a brief synthesis of the clustered SHL configurations relevant to the most critical loading scenarios follows the preceding segments of this chapter.

6.2. SHL DATABASE (I): PORTABLE WIM DATA

Based on the calibrated P-WIM data recorded under the Tx-0-6965 project, our research team developed a comprehensive database of super heavy loads in demanding corridors of the Eagle Ford Shale region. Several SHLs with GHW ranging from 250 kips to 364 kips were collected using the P-WIM deployments in the summer of 2018 and winter of 2019, as shown in Figure 6.1. Figure 6.2 also shows the truck class distributions of SHLs, as captured by P-WIM devices. Based on the plot, the most predominant SHLs were categorized in Class 9 trucks, with 55% of total SHL vehicles recorded in the network, while 28% and 17% of the SHLs were attributed to the Class 10, and Class 13 trucks, respectively.
Figure 6.1: GVW Distributions of SHLs Collected by P-WIM Devices.

Figure 6.2: Truck Class Distributions of SHLs Collected by P-WIM.

Regarding the heavy axle loads, Table 6.1 shows the SHL axle ranges for different axle group types, based on field data. Legal and the maximum permissible weight limits for trucks operating in Texas are also provided for comparison purposes. Axle weights heavier than legal limits are primarily considered as OW, while the axles that exceed the maximum allowable weight limits are essentially regarded as super heavy loads. As evidenced in Table 6.1, the maximum recorded axle loads substantially deviate from the maximum permissible limits. The heaviest axle
load was found to be 140 kips for quad axles, as recorded during the winter data collection in SH 123 in Corpus Christi.

Table 6.1: Super Heavy Axle Loads Collected by P-WIM Devices

<table>
<thead>
<tr>
<th>Axle Type</th>
<th>Axle Weight (lb.)</th>
<th>Legal Weight Limit (OW Threshold)</th>
<th>Maximum Permissible(^{1,2}) (SHL Threshold)</th>
<th>Maximum Recorded Axle Weights in Field Trials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Axle</td>
<td>20,000</td>
<td>25,000</td>
<td>42,000</td>
<td></td>
</tr>
<tr>
<td>Tandem Axle</td>
<td>34,000</td>
<td>46,000</td>
<td>120,000</td>
<td></td>
</tr>
<tr>
<td>Tridem Axle</td>
<td>42,000</td>
<td>60,000</td>
<td>114,000</td>
<td></td>
</tr>
<tr>
<td>Quad Axle</td>
<td>50,000</td>
<td>70,000</td>
<td>140,000</td>
<td></td>
</tr>
</tbody>
</table>

TxDOT Pavement Manual, 2019

SHL vehicles typically consisted of more axles and tires compared to standard trucks; hence, their weights are distributed among numerous axle and tires. Therefore, performance of a pavement section is affected not only by the GVW and axle weight values but also, more importantly, by load magnitudes on tire. For this reason, damage quantification algorithms should properly account for the demanding loading conditions induced by heavy wheel loads. Figure 6.3 provides the results pertaining to the average of ten heaviest wheel load (ATHWL) collected by P-WIM devices in all ten representative sites. Based on the analyzed results, the majority of the studied roadways accommodate SHL vehicles with significantly heavy tires, as the maximum recorded ATHWL was as high as 14.1 kips. Moving such super heavy tires in the network can be detrimental to the longevity and structural integrity of the pavement sections.

Another noteworthy observation from the plot pertains to the fact that the calculated ATHWL in several studied roadways, exceeds the maximum 10 kips limit set forth by TxDOT for analysis of the pavement sections subjected to SHL vehicles. Therefore, the current procedure of
SHL permit evaluation in Texas is not capable of handling such cases with taxing loading conditions imposed on the pavement facilities.

Figure 6.3: Average of Ten Heaviest Wheel Load (ATHWL) Collected by P-WIM Devices.

Accordingly, analysis of the developed SHL database indicated that the majority of the evaluated roadway sections were subjected to substantial repetitions of SHL movements. Figure 6.4 illustrates distributions of all SHL vehicles among different FM, SH, and US highways in ten sites in east Texas. The analyzed results provided in the plot showed that nearly 25% of the traffic mix was attributed to passages of SHL vehicles. The average of the traffic makeup for SH and US highways in the studied network were 20%, and 7%, respectively. Considering the fact that these vehicles carry extremely heavy axle and wheel loads that are primarily characterized in the tail end of the load spectrum, operations of such heavy vehicles can jeopardize the longevity of transportation infrastructures.
Despite obtaining prominent information associated with heavy truck traffic using P-WIM units, these systems are not capable of capturing non-conventional SHL vehicles consisted of complex axle arrangements that are not classified into FHWA Class 4-13 trucks. For this reason, in addition to the P-WIM obtained database, our research team established a supplementary database of the SHLs with multi-axle trailers based on the most recent permit records of SHLs in both Eagle Ford Shale and Permian Basin regions. Analyzing the pertinent data obtained from our communication with TxDMV Motor Carrier Division indicated that more than 8,000 SHL permits with GVW in excess of 250 kips issued during the 2017-2020 period, as indicated in Figure 6.5. The plot also shows that significant number of SHLs exceeded 500 kips, with the maximum recorded as high as 1.8 million lb. Furthermore, Figure 6.6 provides the relevant information on the number of axles for non-conventional SHL units. Based on the information retrieved from available permit records, SHL units can comprised of specialized trailers with significant number of axles as high as 45, while the most common SHL trailers consisted of 15 axle lines, as indicated in Figure 6.6.

Figure 6.4: Super Heavy Load Operations in All Representative Sites within the Eagle Ford.

6.3. **SHL DATABASE (II): PERMIT RECORDS DATABASE**

Despite obtaining prominent information associated with heavy truck traffic using P-WIM units, these systems are not capable of capturing non-conventional SHL vehicles consisted of complex axle arrangements that are not classified into FHWA Class 4-13 trucks. For this reason, in addition to the P-WIM obtained database, our research team established a supplementary database of the SHLs with multi-axle trailers based on the most recent permit records of SHLs in both Eagle Ford Shale and Permian Basin regions. Analyzing the pertinent data obtained from our communication with TxDMV Motor Carrier Division indicated that more than 8,000 SHL permits with GVW in excess of 250 kips issued during the 2017-2020 period, as indicated in Figure 6.5. The plot also shows that significant number of SHLs exceeded 500 kips, with the maximum recorded as high as 1.8 million lb. Furthermore, Figure 6.6 provides the relevant information on the number of axles for non-conventional SHL units. Based on the information retrieved from available permit records, SHL units can comprised of specialized trailers with significant number of axles as high as 45, while the most common SHL trailers consisted of 15 axle lines, as indicated in Figure 6.6.
Figure 6.5: Super Heavy Permits Issued in the Eagle Ford and Permian Basin Regions between 2017 and 2020 (from Correspondence with TxDMV, March 2020).

The developed databases will be further utilized for the identification of the most prevalent/critical loading conditions associated with the super heavy vehicles operating in Texas OW corridors, as discussed in the subsequent section.
6.4. **SHL Categories and Critical Loading Scenarios**

The loading characteristics, such as magnitude, axle type, tire configuration, wheel base, etc. greatly impact the service life of pavement structures. Our research team developed a database of traffic information pertaining to SHLs in this project. Based on relevant data mining techniques, the SHLs can be categorized into three distinct groups, as elaborated in the following.

6.4.1. **Category (I)**

The first category pertains to the super-heavy Class 4-13 trucks, based on the FWHA vehicle classification. These SHL trucks were characterized based on our field testing of ten sites with history of OW/over-load trucks. Figure 6.7 illustrates an example of such demanding loading conditions in State Highway 123 in Corpus Christi for Class 13 truck with GVW as high as 364 kips in the winter of 2019. As shown in the schematic plot, this SHL truck comprised of heavy tandem and tridem axle groups with load magnitudes of 120 and 114 kips, respectively. The heaviest wheel load was also equal to 15 kips. Such high magnitudes of load on tires contribute to the expedited deterioration of the pavement structures.

![Diagram](image)

*Figure 6.7: Loading Conditions of an Operated SHL Vehicle in Texas with GVW of 364 kips.*
6.4.2. Category (II)

The second category covers the non-conventional trailers with numerous axles and tires that are similarly distributed along the entire length of the SHL unit. In this category, tire arrangements, in terms of the number of tires per axle, tire spacing, tire size, and tire load, follow the same pattern among the consecutive axles. Figure 6.8 provides a relevant example of the axle and tire arrangements of a SHL trailer with evenly distributed axles. The illustrated trailer consists of 28 identical axles with eight tires per axle. The corresponding load magnitudes on each axle and tire were found to be approximately 45.9 kips and 5.7 kips, respectively.

![Figure 6.8: Loading Conditions of an SHL Vehicle with Multi-axle Trailer and a GVW above 1.2 Million lb. (from Correspondence with Project Technical Team, February 2020).](image)

6.4.3. Category (III)

The SHL units with separate dollies of multi-axle trailers fall under the third category. Figure 6.9 schematically shows an example of axle configuration and loading conditions of such SHL vehicles, retrieved from the TxDMV permit records. The demonstrated SHL unit comprised of two dissimilar dollies in terms of the number of tires per axle, which are 20 ft apart from one another. The total GVW for this SHL unit exceeds 960 kips. Additionally, since the axle loads are distributed among multiple tires, i.e., 12 and 16 tires per axle, the maximum wheel load was as low as 3.5 kips in this case.
6.4.4. **The Most Prevalent Tire Configurations**

Based on the relevant data mining techniques, five significant groups of tire configurations were identified, as indicated in Figure 6.10. As shown in the schematic plots, the SHL vehicles moving in Texas include 4, 8, 12, and 16 tires per axle. Essentially, the information on the tire arrangement, tire spacing, and tread width are the main contributing factors in the analysis of the non-conventional SHL vehicles.
Figure 6.10: The Most Prevalent Tire Configurations in Texas OW Corridors with different Tires per Axle: (a) 4 Tires, (b) 8 Tires, (c) 8 Tires, (d) 12 Tires, and (e) 16 Tires.
6.4.5. **The Most Critical Loading Scenarios**

Table 2 provides a summary of the most critical loading conditions associated with different categories of SHL vehicles operating in Texas OW corridors. The synthesis of the available data such as P-WIM databases, SHL plans, and the permit records, etc., were the basis for the clustering information provided in Table 6.2. As indicated in Table 6.2, the SHLs classified as category (I) carry more demanding axles and tire loads compared to the SHL vehicles with multi-axle trailers in category (II) and category (III). This is primarily attributed to the fact that the heavy loads carried by conventional trucks are distributed among comparatively lower number of tires and axles.

Heavier wheel loads, coupled with lower spacing between adjacent axles in the SHL trucks that fall under the first category, can potentially result in demanding loading scenarios detrimental to the service life of pavement structures. Another noteworthy observation from Table 6.2 is the fact that the majority of the SHL vehicles operating in Texas serve the oil and gas and other pertinent industries in the Eagle Ford Shale and Permian Basin regions.
Table 6.2: Summary of the Most Critical Loading Scenarios due to SHL Vehicle Operations in the Project

<table>
<thead>
<tr>
<th>SHL Category</th>
<th>Vehicle Designation</th>
<th>FHWA Truck Classification/No. of Dollies</th>
<th>No. of Axles</th>
<th>No. of Tires per Axle</th>
<th>GVW (lb.)</th>
<th>Max. Axle Weight (lb.)</th>
<th>Max. Wheel Load (lb.)</th>
<th>Min. Axle Spacing (in.)</th>
<th>Load Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category (I)</td>
<td>C13-8A-4T</td>
<td>Class 13</td>
<td>8</td>
<td>4</td>
<td>364,000</td>
<td>60,000</td>
<td>15,000</td>
<td>50</td>
<td>Serving Oil Refineries</td>
</tr>
<tr>
<td></td>
<td>C9-5A-4T</td>
<td>Class 9</td>
<td>5</td>
<td>4</td>
<td>282,000</td>
<td>56,000</td>
<td>14,000</td>
<td>55</td>
<td>Serving Oil Refineries</td>
</tr>
<tr>
<td></td>
<td>C10-6A-4T</td>
<td>Class 10</td>
<td>6</td>
<td>4</td>
<td>264,000</td>
<td>38,000</td>
<td>9,500</td>
<td>55</td>
<td>Serving Oil Refineries</td>
</tr>
<tr>
<td></td>
<td>C13-8A-4T</td>
<td>Class 13</td>
<td>7</td>
<td>4</td>
<td>310,000</td>
<td>35,000</td>
<td>8,750</td>
<td>54</td>
<td>Serving Oil Refineries</td>
</tr>
<tr>
<td>Category (II)</td>
<td>28A-8T</td>
<td></td>
<td>1</td>
<td>28</td>
<td>1,216,164</td>
<td>45,969</td>
<td>5,746</td>
<td>55</td>
<td>Crane Mat</td>
</tr>
<tr>
<td></td>
<td>30A-8T</td>
<td></td>
<td>1</td>
<td>30</td>
<td>1,385,550</td>
<td>46,185</td>
<td>5,773</td>
<td>60</td>
<td>Well Servicing Unit (Self-Propelled)</td>
</tr>
<tr>
<td></td>
<td>24A-8T</td>
<td></td>
<td>1</td>
<td>24</td>
<td>1,162,992</td>
<td>48,458</td>
<td>6,050</td>
<td>60</td>
<td>Well Servicing Unit (Self-Propelled)</td>
</tr>
<tr>
<td></td>
<td>26A-8T</td>
<td></td>
<td>1</td>
<td>26</td>
<td>983,354</td>
<td>48,186</td>
<td>6,023</td>
<td>59</td>
<td>Power Transformer</td>
</tr>
<tr>
<td>Category (III)</td>
<td>2D-18A-8T</td>
<td></td>
<td>2</td>
<td>18</td>
<td>1,809,000</td>
<td>45,500</td>
<td>5,688</td>
<td>59</td>
<td>1075 MVA Generator Stator</td>
</tr>
<tr>
<td></td>
<td>2D-14A-12T</td>
<td></td>
<td>2</td>
<td>14</td>
<td>1,120,000</td>
<td>40,000</td>
<td>3,300</td>
<td>60</td>
<td>T-100 Reactor</td>
</tr>
<tr>
<td></td>
<td>2D-11A-12T</td>
<td></td>
<td>2</td>
<td>11</td>
<td>962,046</td>
<td>48,651</td>
<td>3,040</td>
<td>60</td>
<td>Well Servicing Unit (Trailer Mounted)</td>
</tr>
<tr>
<td></td>
<td>4D-6A-16T</td>
<td></td>
<td>4</td>
<td>6</td>
<td>1,152,000</td>
<td>48,000</td>
<td>3,000</td>
<td>57</td>
<td>T-100 Reactor</td>
</tr>
</tbody>
</table>
Chapter 7: Analysis of the SHLs with Multi-Axle Trailers

7.1. INTRODUCTION

The review of the loading characteristics of SHL vehicles from the P-WIM data and available TxDOT databases underscored the complexity of the axle arrangements, tire configurations, and other relevant features of non-conventional vehicles. For this reason, it might not be practically feasible to follow conventional analysis procedures that relied on the incorporation of the high number of axles and tires into the simulation process. Hence, it deems necessary to develop a practical and optimized analysis approach for efficient simulation of the non-conventional loading conditions of SHL vehicles. This is the precursor for mechanistic quantification of the damages and the structural impacts induced on the pavements due to the SHL vehicles.

To clarify the complex nature of the SHLs with multi-axle trailers, a SHL vehicle with 28 axle lines and eight tires per axle, was selected as a case study, as shown in Figure 7.1. Evidently, since the entire SHL unit consisted of 224 tires, it is not computationally efficient to simulate each individual tire for calculation of the pavement responses induced by the SHL operations. This underscores the need for the development of an approach, which is based on simulation of the representative axles and tires, instead of simulating the entire assembly of axles for non-conventional SHLs.

Distribution pattern of the induced responses in the pavement structure is the key for determination of the SHL representative elements. For instance, the vertical stresses induced by each individual tire partly overlap within the pavement depth. However, due to the considerably large size of the SHL vehicles and distantly arranged axles and tires, only a few number of tires influence the overlapped zone at a particular evaluation point. The hyperbolic effect and the geospatial influence zones are the primary basis for the selection of critical response locations.
Figure 7.1 schematically shows the overlapped stress distributions at Point A, i.e., top of the subgrade layer, as well as the influential tires in the determination of the vertical stresses under the described SHL vehicle with eight tires per axle.

Figure 7.1: Example of Overlapped Stress Distributions at the Top of Subgrade under the SHL Movement.

Subsequent to this introductory section on the basic concepts and the pertinent rationales, the following section elaborates on the proposed approach for the determination of the SHL representative segments, through a relevant case study. A brief synthesis of determination and incorporation of the critical loading conditions associated with different SHL categories into the structural analysis of pavements is also provided in this chapter. Eventually, the pavement responses induced by the most critical SHL cases with taxing loading arrangements will follow the preceding segment of this chapter.

7.2. PROPOSED METHODOLOGY

The UTEP research team designed a framework to determine the SHL representative segments that are influential in the mechanistic calculation of the pavement responses under any arbitrary
SHL configuration. The analysis is founded on incrementally adding the SHL tires in both traffic and transverse directions, as illustrated in Figure 7.2. The variations of the maximum induced pavement responses are monitored accordingly. The SHL representative tires are then determined when any further increase in the number of tires has a negligible effect on the maximum responses within the pavement structure. The following three critical pavement responses, as key parameters in the pavement damage and stability analyses, were investigated using the finite element models:

- **Vertical deflection at the pavement surface,**
- **Compressive strain at the top of the subgrade,**
- **Vertical stress at the top of the subgrade.**

Figure 7.2: Incremental Addition of Tires for Determination of SHL Representative Elements.

For the SHL case used as an example in this section, eight tires in both directions were incrementally added, and the critical pavement responses were accordingly calculated. Figure 7.3 shows the simulated SHL tire footprints for the calculation of the corresponding pavement responses using 3D finite element modeling. The pavement profile of SH 123, consisted of 5.5 in. asphalt concrete (AC) layer and 15 in. of flexible base layer constructed over the subgrade soil, was modeled for the performed sensitivity analysis in this section.
Figure 7.3: Simulation of the SHL Tires (8×8) Using 3D FE Models.

Figure 7.4 illustrates the variation of the pavement responses with incremental increases in the number of SHL tires. The demonstrated panel on the left side of the figure pertains to the pavement response variations by adding tires in the transverse direction that is perpendicular to the direction of traffic; while, the right-side panel shows the corresponding responses by adding tires in the traffic direction, i.e., longitudinal direction. As evidenced in the plots, the maximum values for three evaluated pavement responses showed negligible sensitivity when the simulated axle configuration exceeds 4 number of tires in the transverse direction.

The changes in the slope of bi-linear trend lines in Figure 7.4 identifies the number of influencing tires on the calculate responses. For instance, Figure 7.4 (a) shows that the deflection change along the centerline of load, beyond the 4th tire is negligible. Therefore, efficient simulation requires no more than 4 tires in longitudinal and 4 tires in transverse directions. Consequently, the representative arrangement of the SHL axles and tires includes a set of four by four tires that are influential in the determination of the maximum pavement responses. Therefore, the entire SHL unit with 224 tires is then easily split into the small units, as shown in Figure 7.5.
Figure 7.4: Influence of Adding SHL Tires on the Induced Pavement Responses: (a) Surface Deflection, (b) Compressive Strain at the Top of Subgrade, and (c) Vertical Stress at the Top of Subgrade.
It should also be noted that the results presented in this section pertain to the described loading configurations of SHL case 28A-8T simulated over the pavement structure of SH 123 with the summer-based layers material properties. Essentially, incorporation of different loading conditions or different pavement structural properties into the developed analysis algorithm might lead to a different arrangement of the SHL influential tires. This is due to the fact that the pavement responses directly influenced by loading conditions, layers material properties, layer configurations, and layer thicknesses.

Figure 7.5: Segmentation of the SHL with 224 Tires into Representative Sets of 4×4 Tires.

7.3. SYNTHESIZED ANALYSIS FLOWCHART

Figure 7.6 provides a synthesized flowchart of the proposed procedure for the determination and incorporation of critical loading conditions associated with different SHL categories into the pavement structural analysis. Initially, the research team conducted an extensive data mining of the P-WIM databases, records of the SHL permits, as well as the SHL-vehicle plans, in order to obtain the field-derived and state-specific characteristics of the SHL vehicles operating in Texas OW corridors.
Based on the vehicle characteristics and arrangement of the axles and tires, each individual SHL vehicle was further classified into a particular category. The SHL vehicles under the first category consisted of distinct axle groups; hence, it is imperative to separately analyze different axle groups to determine the most critical loading conditions. It is also noted that due to the closely-spaced heavy tires of such vehicles, adjacent tires within each axle group can significantly contribute to the imposed maximum pavement responses. For this reason, all the tires within each axle group should be considered in the analysis of the super heavy FHWA trucks.

The primary step for the analysis of the SHL cases under the second category with evenly distributed multi-axle trailers is to determine the representative arrangements of the axles and tires that are influential in demanding loading conditions. In these cases, the entire SHL unit is regarded as one axle group subdivided by representative sets of tires. The third category includes the SHL vehicles with distantly-spaced dollies of multi-axle trailers. In such cases, each individual dolly is considered as one axle group; hence, it is necessary to determine the SHL representative elements associated with each dolly. The most critical loading conditions among different dollies can be further identified by comparing the maximum induced pavement responses.

Ultimately, the SHL representative elements imposing the most critical loading conditions within the pavement structure will be incorporated into the advanced numerical modeling procedure for further analysis of the structural impacts of SHLs on the pavement infrastructures.
Figure 7.6: Synthesized Flowchart for Characterizing and Incorporating the SHL Representative Elements into the Pavement Structural Analysis.
7.4. Sensitivity Analysis

A sensitivity analysis was performed to investigate the influence of maximum wheel load, roadway type, and environmental factors on the determination of the number of SHL influencing tires. To accomplish this objective, initially, the research team incrementally increased the load magnitude on each individual tire from 6 kips to 10 kips. Then, by adopting the proposed algorithm, the influencing tires that contribute to the maximum pavement responses were determined. It should be noted that all other analysis parameters remained the same as the preceding analysis presented in section 4.2. Similar analyses were performed by changing the type of roadway facility. Therefore, different roadway types, i.e., FM, SH, and US highways, were considered in the analysis representing thin, intermediate, and thick AC pavements, respectively. The FWD back-calculated material properties of the layers associated with the summer and winter months were incorporated into the analysis to assess the influence of environmental factors on the selection of the number of influencing tires.

Table 3 provides the relevant information on the several evaluated case scenarios considering different wheel loads, roadway types, and seasons of the year. The corresponding results, i.e., $N_L$ and $N_T$, representing the number of influencing SHL tires in longitudinal and transverse directions, respectively, are also provided in Table 7.1. As expected, for the range of input parameters considered in this analysis, the influential tires increased with the increase in the maximum heaviest wheel load of the SHL vehicle. In other terms, more tires can contribute to the maximum pavement responses induced by a SHL vehicle with heavier wheel loads. As indicated in Table 7.1, more tires need to be considered when the SHL is applied to the pavement structure of the FM roadways, compared to the corresponding influential tires in SH and US highways. This is due to the fact that FM roadways with less robust pavement profiles are less efficient in
dissipating the stresses induced by multiple tires in the pavement layers. Therefore, these roadways are more sensitive to the number of SHL tires influencing the critical pavement responses.

Another noteworthy observation from Table 7.1 pertains to the influence of seasonal variations of the material properties on the selection of the axle assemblies. As indicated in Table 7.1, the number of influencing tires with the simulation of the pavement structure with summer-based material properties were higher than the corresponding number of tires with incorporation of the winter-based back-calculated material properties. This is primarily attributed to the viscoelastic nature of the asphalt layer, and softening of the surface layers due to elevated temperatures in summer season, leading to lower structural capacity of pavements during summer months. Therefore, a higher number of tires contributes to the demanding loading conditions induced by SHL operation during summer time.

Consequently, the analysis algorithm for the determination of the SHL influencing tires should properly account for the simultaneous effects of the heaviest wheel load, structural capacity of the pavements, and seasonal variations of the layers material properties to improve the accuracy of the analysis results.
Table 7.1: Summary of the Effect of Contributing Factors in Determination of the SHL Influencing Tires

<table>
<thead>
<tr>
<th>Evaluated Factor</th>
<th>Case Scenario</th>
<th>Wheel Load (kips)</th>
<th>Roadway Type</th>
<th>Season</th>
<th>( N_L )</th>
<th>( N_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heaviest Wheel Load</td>
<td>Case 1</td>
<td>6</td>
<td></td>
<td></td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Case 2</td>
<td>7</td>
<td></td>
<td></td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Case 3</td>
<td>8</td>
<td>State Highway</td>
<td>Summer</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Case 4</td>
<td>9</td>
<td></td>
<td></td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Case 5</td>
<td>10</td>
<td></td>
<td></td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>Roadway Type</td>
<td>Case 6</td>
<td>6</td>
<td>Farm-to-Market</td>
<td></td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Case 7</td>
<td>6</td>
<td>State Highway</td>
<td>Summer</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Case 8</td>
<td></td>
<td>US Highway</td>
<td></td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Season</td>
<td>Case 9</td>
<td>6</td>
<td>State Highway</td>
<td>Summer</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Case 10</td>
<td>6</td>
<td></td>
<td>Winter</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

\( N_L \) = number of influencing tires in longitudinal direction

\( N_T \) = number of influencing tires in transverse direction

7.5. **ANALYSIS OF SHL UNITS WITH CRITICAL LOADING SCENARIOS**

This section provides the results pertaining to the SHL representative elements as well as the pavement responses imposed under the most critical loading conditions associated with different SHL categories.

7.5.1. **Determination of the Representative Axle Assembly**

The research team implemented the developed approach to determine the number of influencing axles and tires in the analysis of SHLs with multi-axle trailers. Table 7.2 provides a summary of
the representative axle assembly associated with different categories of SHL vehicles that impose
the most critical loading conditions in the network. It is also noted that the pavement profile of SH
123 with summer-based layers material properties, as provided in chapter four, was modeled for
the numerical simulations in this section.

The SHL axle groups with demanding loading conditions, as well as the information on
the number of SHL influencing tires in both longitudinal and transverse directions, i.e., $N_L$ and $N_T$,
respectively, for different SHL cases are provided in Table 7.2. This information is instrumental
in facilitating the calculation procedure of the critical pavement responses under the moving SHL
vehicles with complex loading configurations and numerous axles and tires. This is primarily due
to the fact that instead of simulating the entire assembly of axles for non-conventional SHLs, the
influencing tires as representative elements were incorporated into the finite element program for
further characterization of the structural impacts on pavement facilities.
### Table 7.2: Representative Axle Assembly Imposing Critical Loading Conditions associated with Different SHL Categories

<table>
<thead>
<tr>
<th>SHL Category</th>
<th>SHL Case</th>
<th>Vehicle Designation</th>
<th>GVW (kips)</th>
<th>Max. Axle Load (kips)</th>
<th>Max. Wheel Load (kips)</th>
<th>Axle Load Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category (I)</td>
<td>Case 1</td>
<td>(C13)(^1)-(8A)(^2)-(4T)(^3)</td>
<td>364</td>
<td>60.0</td>
<td>15.0</td>
<td>Tandem Axle</td>
</tr>
<tr>
<td></td>
<td>Case 2</td>
<td>C9-5A-4T</td>
<td>266</td>
<td>56.0</td>
<td>14.0</td>
<td>Tandem Axle</td>
</tr>
<tr>
<td></td>
<td>Case 3</td>
<td>C10-6A-4T</td>
<td>264</td>
<td>38.0</td>
<td>9.5</td>
<td>Tridem Axle</td>
</tr>
<tr>
<td></td>
<td>Case 4</td>
<td>C13-7A-4T</td>
<td>310</td>
<td>35.0</td>
<td>8.8</td>
<td>Tandem Axle</td>
</tr>
<tr>
<td>Category (II)</td>
<td>Case 5</td>
<td>28A-8T</td>
<td>1,216</td>
<td>45.9</td>
<td>5.7</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Case 6</td>
<td>30A-8T</td>
<td>1,385</td>
<td>46.2</td>
<td>5.8</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Case 7</td>
<td>24A-8T</td>
<td>1,162</td>
<td>48.5</td>
<td>6.1</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Case 8</td>
<td>26A-8T</td>
<td>983</td>
<td>48.2</td>
<td>6.0</td>
<td>4</td>
</tr>
<tr>
<td>Category (III)</td>
<td>Case 9</td>
<td>(2D)(^4)-18A-8T</td>
<td>1,809</td>
<td>45.5</td>
<td>5.7</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Case 10</td>
<td>2D-14A-12T</td>
<td>1,120</td>
<td>40.0</td>
<td>3.3</td>
<td>3</td>
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<td></td>
<td>Case 11</td>
<td>2D-11A-16T</td>
<td>962</td>
<td>48.7</td>
<td>3.0</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Case 12</td>
<td>4D-6A-16T</td>
<td>1,152</td>
<td>48.0</td>
<td>3.0</td>
<td>3</td>
</tr>
</tbody>
</table>

*Truck class*

\(^2\) Number of axles

\(^3\) Number of tires per axle

\(^4\) Number of dollies/trailers

\(^5\) \(N_L\) = number of influencing tires in longitudinal direction

\(^6\) \(N_T\) = number of influencing tires in transverse direction

7.5.2. **Determination of the Critical Pavement Responses**

Figure 7.7 shows the calculated pavement responses under the evaluated SHL representative elements imposing the most critical loading scenarios in the network. Based on the numerical simulations performed in this subtask of the project, the evaluated pavement responses, i.e.,
maximum deflection at the pavement surface, compressive strain at the top of subgrade layer, and the tensile strain at the bottom of AC layer, were found to be significantly higher under category (I) of SHLs.

In other terms, operation of the SHL vehicles that fall under category (I) can induce substantially higher pavement responses within the pavement structures, and therefore higher level of imparted damages, compared to the evaluated SHL cases under category (II) and (III). This is primarily due to the fact that, despite having lower GVW ranges, the SHLs classified as category (I) carry significantly heavier axles and tires compared to the SHL vehicles with multi-axle trailers in category (II) and category (III). This is because the heavy loads carried by conventional trucks are distributed among comparatively lower number of tires and axles. Consequently, heavier wheel loads, coupled with lower spacing between adjacent axles in the SHL trucks that fall under the first category, can potentially result in demanding loading scenarios being detrimental to the service life of pavement structures.
Figure 7.7: Pavement Responses under the Critical Loading Scenarios for Different SHL Categories: (a) Maximum Surface Deflection, and (b) Subgrade Compressive Strain, and AC Tensile Strain.
7.5.3. Selection of the Most Critical SHL Case under Similar Wheel Load Magnitude

There are several load-related factors and parameters that greatly contribute to the analysis of the pavement responses induced under SHL vehicles. Loading characteristics, such as axle load magnitude, axle type, axle arrangement, tire configuration, and load magnitude on individual tires, are the primary factors, among others, in the analysis of SHL units and their impact on the pavement facilities. More specifically, the wheel load is one of the major components that substantially contributes to the loss of pavement life and the damages imparted over the pavement design life.

As described earlier, the SHL cases evaluated in this section consisted of trailer units with various levels of load magnitude on their wheels. Hence, selection of the most critical SHL case with demanding loading configuration should also account for the wheel load magnitude. For this reason, and to simulate identical conditions, the authors conducted another series of comparative analysis to investigate the pavement performance under such SHL cases but with the same wheel load magnitude. The results are instrumental in identifying the axle assembly and tire configuration of the SHL units that impose the most critical loading conditions, with inclusion of the same wheel load magnitude in the analysis.

Figure 7.8 provides the numerical simulation results associated with the maximum surface deflection induced under the described SHL cases. The authors assigned 6 kips as load magnitude on tire for all the SHL cases to simulate identical conditions. As evidenced in the plot, with inclusion of 6 kips as wheel load, operation of the SHLs with multi-axle and multi-wheel trailer units that fall under category (II), resulted in the highest pavement responses on average, followed by category (III) and ultimately category (I) of SHL vehicles. This is mainly due to the complex axle assembly and higher number of tires within one axle group, for category (II) of SHLs that
resulted in more critical loading conditions on the pavement facilities, under the same wheel load magnitude. This underscored the significance of unique axle arrangement and tire configuration of the SHL units and their impact on pavement responses. Consequently, damage quantification procedures should properly take into consideration not only the load magnitudes on the vehicle axles and tires, but also the non-generic axle assembly and tire configuration of non-conventional SHL units.

Figure 7.8: Maximum Surface Deflection under Different SHL Case Scenarios with Identical Load Magnitude on Tire as 6 kips.

Another major observation from the plot pertains to the 28A-8T SHL case, with 28 axle lines and eight tires per axle that induced the highest deflection at the pavement surface, among all other case scenarios evaluated in this section. Consequently, the described vehicle is selected as the most critical SHL case, and will be further incorporated into the damage quantification procedure to account for the most critical loading scenarios in the analysis. It will be also included in the numerical simulations and several sensitivity analyses that will be conducted to investigate
the influence of major parameters such as pavement structure, traffic loading conditions, and climatic factors, on the damages imparted on the pavement sections.

7.6. SUMMARY OF THE MAJOR POINTS

In this chapter, our research team elaborated on the methodology for determination of the number of influencing tires for category II and category III in the analysis of SHL loading groups. The selection of the number of tires and the axle assemblies are primarily based on the hyperbolic effect and the geospatial influence zones in structural analysis of pavements.

There are several factors that contribute to the selection of the number of influencing tires. In addition to the axle and tire arrangement, the type of roadway facility is a major contributing factor in the selection of the axle assemblies. For instance, state highways with thick layers and good quality materials dissipate the stresses in more efficient manner compared to the Farm to Market roads with less robust layers. Therefore, the algorithms for the selection of the axle assemblies should have the flexibility to adjust when the transition of the SHL from SH or an interstate highway to a FM road occurs.

In addition to the sensitivity of the approach to the type of transportation facility, the algorithm developed and presented in this section accounts for the variation of the material properties at different times of the year. Conceptually, the damages imparted by same vehicle during the hot summer months are substantially higher compared to cold winter months. Similarly, intrusion of moisture in granular layers due to poor drainage conditions in FM roads can be the culprit for the premature failure of pavements subjected to SHL vehicles. Therefore, the algorithm should be able to account for the cyclical nature of material properties due to environmental conditions.
Furthermore, the results pertaining to the SHL representative elements as well as the pavement responses imposed under the most critical loading conditions associated with different SHL categories are provided in this chapter. The SHL case with the most critical axle loading arrangement is also selected and will be further incorporated into the damage quantification procedures to account for the most critical loading scenarios in the analysis.
Chapter 8: Advanced Modeling of the Moving SHL Vehicles

8.1. INTRODUCTION

Damage quantification and service life prediction algorithms, using numerical modeling approaches, should properly account for the realistic simulation of the pavement structural properties and the induced loading conditions. Such necessity is even more pronounced for the analysis of the pavement structures subjected to SHL movements that impose demanding loading conditions. Hence, the simplifying assumptions such as ignoring the time-dependent nature of the viscoelastic asphalt layers, or unrealistic simulation of the tire-pavement contact stresses using uniformly distributed load, rather than considering non-uniform distribution of contact stresses can potentially jeopardize the accuracy and reliability of damage quantification and remaining life analyses of pavement facilities in overload corridors.

The primary objective of this chapter is to provide a finite element code that makes adequate provisions for advanced modeling of moving SHL vehicles in order to accurately calculate the induced pavement responses. The finite element program will be used for determination of the critical pavement responses, with considerations of the viscoelastic nature of the asphalt concrete layer, complex traffic loading conditions, non-uniform distribution of tire-pavement contact stresses, various SHL-vehicle operational speeds, and acceleration/deceleration forces.

8.2. ADVANCED NUMERICAL SIMULATION

Realistic simulation of the moving SHL vehicles greatly influences the accuracy of the assessment of the pavement responses imposed by demanding loading conditions. To realistically simulate the SHL movements in this subtask of the project, the research team developed a FE code that incorporates adequate provisions for advanced modeling of the moving SHL vehicles. For this
reason, the ABAQUS FE program was used for the determination of the SHL-induced pavement responses under various vehicle loading scenarios, with considerations of the viscoelastic nature of the AC layer, complex traffic loading conditions, and tire-pavement interactions. Subsequent sections provide relevant information on the 3D FE models developed in this study for simulation of the pavement structures, material behavior models, SHL vehicles at steady rolling condition, and tire-pavement interactions.

8.2.1. Simulation of the Pavement Structure

Based on the non-destructive testing results provided in chapter four, pavement structures consisting of AC, base, and subgrade layers were modeled in the software. The site-specific information relevant to the pavement layer thicknesses, layer configurations, as well as layer moduli, were incorporated into the developed FE code to simulate the structural characteristics of the representative pavement sections.

8.2.1.1. AC Viscoelastic Behavior Modeling

The structural capacity of multi-layer pavement structures is not monolithic under different loading frequencies and different temperatures. This is primarily attributed to the time and temperature-dependent behavior of the viscoelastic AC layers. In this research effort, the viscoelastic properties of the AC surface layers were incorporated in the analysis to account for the influence of temperature and frequency of loading. Hence, the generalized Maxwell model was used to characterize the viscoelastic properties of the AC layers through FE numerical simulations in this study, as presented in the following equations:

\[ s = \int_{-\infty}^{t} 2G(t - \tau) \frac{de}{d\tau} d\tau \]  

\[ p = \int_{-\infty}^{t} K(t - \tau) \frac{d(tr[e])}{d\tau} d\tau \]
where $s$ is deviatoric stress, $e$ is deviatoric strain, $p$ is volumetric stress, $tr[e]$ is trace of volumetric strain, $t$ is relaxation time, and $K$ and $G$ are the bulk and shear moduli of AC layer, respectively. Bulk ($K$) and shear ($G$) moduli of AC were obtained from laboratory dynamic modulus tests. The Prony series were then used to calculate the corresponding modulus values in the time domain, as indicated through Equations 9-3 and 9-4 (ABAQUS, 2014):

$$G(t) = G_0 \left[ 1 - \sum_{i=1}^{n} G_i \left( 1 - e^{-t/\tau_i} \right) \right]$$ (9-3)

$$K(t) = K_0 \left[ 1 - \sum_{i=1}^{n} K_i \left( 1 - e^{-t/\tau_i} \right) \right]$$ (9-4)

where $G_0$ and $K_0$ are instantaneous shear and volumetric elastic moduli; and $G_i, K_i,$ and $\tau_i$ are the Prony series parameters. The information on the dynamic modulus tests, as well as the Prony series parameters, were obtained from a relevant study on TxDOT asphalt mixes conducted by Hu et al. (2017).

### 8.2.1.2. Model Geometry and Dimension

Proper characterization of the dimensions and geometry of the model is of outmost importance to mitigate the systematic errors associated with boundary effect problems (Morovatdar et al., 2021f; 2020a & d). The research team carried out a comprehensive sensitivity analysis to determine the adequate subgrade depth and model longitudinal and transverse widths for simulation purposes based on the specific pavement structure of the selected sites. To achieve this objective, the height of the model was incrementally increased from 50 inches to 150 inches, the variations of the calculated responses were monitored accordingly. As shown in Figure 8.1, the sensitivity of the critical pavement responses are negligible when the simulated model height exceeds 80 inches. Therefore, the research team selected 100 inches as representative height of model for further numerical simulations.
Similar sensitivity analyses were conducted to determine the model longitudinal and transverse widths. Based on the sensitivity analysis results, the optimum value of the transverse width of the model was calculated as 150 inches. Additionally, a relatively large value, i.e., 1,000 inches, for the pavement length was considered to allow movement of the SHL vehicles at different operational speeds and rolling scenarios to reduce the adverse impacts of model boundaries. Therefore, a 3D block of pavement structure with dimension of 80×150×1,000 m (height, width, and length) was modeled using 3D continuum elements.

Figure 8.2 shows the model dimension as well as different pavement layers consisting of surface layer, base and subgrade layers simulated in the FE program. The representative axle arrangement of the SHL case 28A-8T including a set of four by four tires was also incorporated into the numerical modeling as an example in this section. However, the computational time required for model convergence was considerably high due to the extremely large size of the model with huge number of elements and the interacting effects in dynamic analysis. For this reason, only a half size of the model was simulated in the FE program in order to optimize the computational efficiency of FE analysis in symmetric problems, as shown in Figure 8.2.
8.2.1.3. Boundary Conditions

Boundary Conditions (BC) are essentially defined as the displacement or rotation constrains to avoid the movement of the selected degrees of freedom, or to prescribe the displacement or rotation for each selected degree of freedom (Beyzaei et al., 2019; ABAQUS Manual, 2015). Since boundary conditions play a critical role in the FE modeling, attention should be paid to defining appropriate boundary conditions to assure a realistic model. Figure 8.3 shows different types of BCs defined in the simulation of pavement structure models. ENCASTRE boundary condition was used at the bottom of the subgrade layer to simulate the bedrock. ENCASTRE is a specific type of BCs that restraints the displacement and rotation in all directions. Moreover, two other BCs were defined in the FE models to restrict the displacement in the orthogonal directions to the indicated surfaces.
8.2.1.4. **Meshing**

Considering the fact that the most critical response points in the procedure of pavement performance analysis are located under the wheel path, the research team defined a finer mesh in this region for more accurate results. However, to expedite the computation time and reduce the output files size, a coarser mesh in the regions far from the loading areas was used. Therefore, as shown in Figure 8.4, three-dimensional continuum elements C3D10 (ten-node quadratic tetrahedron) with finer meshing size under the wheel path, and coarser meshing size for further regions, were defined in the model. Additionally, a transition zone was defined between fine and coarse mesh areas for gradual change of element size to ensure improved accuracy.
8.2.2. Simulation of the Moving SHL Vehicles

Simulation of the moving vehicles at steady rolling condition involves several aspects of complexity in terms of the realistic tire simulation, analysis approach, boundary conditions, element constraints, tire movement incorporation, and tire-pavement interactions. The following sections provide the relevant information on the procedure adopted in this study to simulate the moving SHL vehicles using advanced numerical modeling techniques.

8.2.2.1. Simulation of the Tire Element

The tire elements, including tire ribs and grooves, were explicitly modeled with consideration of the characteristics and tread patterns of SHL-vehicle tires operating in Texas overload corridors. Figure 8.5 illustrates the cross-sectional and 3D views of the modeled tire. Based on the information extracted from the review of the SHL vehicle plans and permit records in Texas, it was found that the SHL tire’s height ranged from 28 in. to 35 in., while tread width of the tires varied between 6 and 10 in., depending on the tire configuration and axle assembly of different SHL vehicles. The elastic material properties of the tire element were also defined based on the relevant information provided in the literature (Wang et al., 2012). Additionally, 3D hybrid
elements (C3D10H) were assigned to the SHL vehicle tires, as using hybrid elements is recommended for simulation of vehicle tires that behave similar to the incompressible materials.

Figure 8.5: Simulation of the SHL Tire using 3D FE Models.

8.2.2.2. Steady Rolling Condition

The research team used the dynamic analysis offered in ABAQUS to simulate the tire rolling procedure. Constant translational speed \( (v) \), as well as the corresponding rotational speed \( (\omega) \), were assigned to the SHL tires moving in the traffic direction to simulate the steady rolling condition. The BCs were also assigned to the centroid of the SHL tires to restrict the displacement and rotation in all other directions. Additionally, the kinematic coupling constraint method was used to allow movement of the tires along the traffic direction.

A friction coefficient of 0.5, as recommended by Wang et al. (2012), was also incorporated in the numerical simulations to account for the frictional effects at the tire-pavement interface. To consider the frictional interactions in this subtask, the penalty method, as well as the Coulomb friction law, were used to simulate the normal and tangential interactions, respectively.

8.2.2.3. Different Vehicle Speeds and Tire Rolling Scenarios

Proper incorporation of the vehicle operational speed and tire rolling conditions is the key component in dynamic analysis of the moving SHL vehicles. In this project, the research team incorporated different translational speeds as well as the corresponding rotational speeds in a
comprehensive sensitivity analysis to account for the influence of operational speed of the SHL vehicles in this study. Furthermore, various tire rolling scenarios, i.e., steady rolling, acceleration, and deceleration (braking), were simulated to investigate the acceleration/deceleration effects in this research study.

8.2.2.4. Tire-Pavement Contact Stresses

Damage quantification and service life prediction algorithms for the pavements subjected to SHLs with taxing loading conditions should properly account for the realistic tire-pavement contact stresses. Figure 8.6 provides an example of the simulated tire-pavement contact stresses under a SHL tandem-axle with 120 kips axle weight and 15 kips load magnitude on each individual tire. The described load pertains to a SHL vehicle recorded by the P-WIM devices during the winter of 2019 in Corpus Christi District (Ashtiani et al., 2019). As shown in Figure 8.6, the simulated tire elements in this study properly captured the non-uniform distribution of the tire-pavement contact stresses under each individual tire rib. The characterized convex-shape distribution pattern of contact stresses along the tire contact length is consistent with the experimental measurements and observations made by previous researchers (Douglas et al., 2000; Wang and Al-Qadi 2009). Based on the contact stress distributions provided in Figure 8.6, the maximum calculated contact stress was 180 kips in the mid-ribs of the tire; while, the average value was calculated as 125 psi. This average value for contact stresses is in line with our research team's field measurements of tire pressure in Texas overload corridors (Ashtiani et al., 2019).
Proper incorporation of the non-uniform tire pavement contact stresses in the finite element numerical simulations is the primary step in the dynamic analysis of the moving SHL vehicles. Figure 8.7 provides a schematic view of the simulated moving SHL vehicle, with consideration of the contact stresses induced along the tire footprint length, including the departing and approaching zones. As evidenced in the plot, with movement of the SHL vehicle along the traffic direction, the non-uniform contact stresses induced at the pavement surface accordingly shift forward to more realistically simulate the dynamic nature of the moving SHL vehicles.

Figure 8.7: Simulation of the Moving SHL Vehicle with Non-uniform Tire-Pavement Contact Stresses.
8.2.3. **Influence of Non-Uniform Distribution of Contact Stresses on Pavement Responses**

The authors performed a series of comparative analyses to investigate the influence of tire-pavement contact stresses on the critical pavement responses, i.e., maximum deflection at the pavement surface, compressive strain at the top of subgrade layer, and tensile strain at the bottom of the AC layer. To accomplish this, two types of contact stress distributions, namely non-uniform distribution and the equivalent uniform distribution with the same contact area, were incorporated into the FE numerical models. To consider the loading conditions in this subtask, two different axles, including the 18-kips reference axle that represent a conventional loading conditions, and 120-kips tandem axle load with more demanding loading conditions representing a SHL vehicle were incorporated into the numerical simulations. Ultimately, the corresponding responses induced within the pavement structure were calculated for further comparisons.

Figure 8.8 and 8.9 provide the results associated with the simulated contact stresses and the corresponding pavement responses, respectively, calculated under the 18-kips reference axle with 4.5 kips wheel load magnitude. Based on the contract stress distributions provided in Figure 8.8a, the maximum calculated contact stress under the tire was 108 kips in the mid-ribs of the tire; while, the average value was calculated as 80 psi. Therefore, a uniform contact stress distribution with one constant value of 80 psi was also defined in the numerical simulation for comparison purposes, as shown in Figure 8.8b. Figure 8.9 provides the results associated with the analysis of pavement responses under various types of contact stresses for 18-kips reference axle. The results indicated that inclusion of the non-uniform distribution of contact stresses in the analysis led to considerably higher pavement responses under the studied reference axle, compared to the other counterpart. Based on the results provided in Figure 8.9, incorporation of non-uniform distribution in lieu of uniform distribution of contact stresses into the numerical simulations resulted in an
increase in the monitored pavement responses, i.e., surface deflection, subgrade strain, and AC strain, by 5%, 3%, and 4%, respectively. Such percent changes when incorporated into the exponential transfer functions for fatigue and rutting performances can lead to a significant increase in the accumulated pavement damage predicted during the pavement design life. Therefore, it deems necessary to properly account for the realistic simulation of the non-uniform contact stress distributions in the numerical simulation phase to accurately assess the pavement performance.

Figure 8.8: Contact Stresses under 18-kips Reference Axle Tire: (a) Non-uniform Distribution, and (b) Uniform Distribution.
Figure 8.9: Influence of Non-Uniform Distribution of Contact Stresses under 18-kips Reference Axle on the Pavement Responses: (a) Surface Deflection, (b) Compressive Strain at the Top of Subgrade, and (c) Vertical Stress at the Top of Subgrade.

The second series of the analyses aim to evaluate the influence of type of contact stress distributions on the pavement responses induced under a SHL axle with taxing loading conditions. Figure 8.10 and 8.11 show the simulated contact stresses and the corresponding pavement responses, respectively, calculated under the 120-kips SHL axle with 15 kips load magnitude on each individual tire. Based on the contract stress distributions provided in Figure 8.10a, the maximum calculated contact stress was 180 kips; while, the average value was calculated as 125 psi. Hence, the equivalent uniform contact stress with one single value of 125 psi was also included in the comparative analysis, as shown in Figure 8.10b.
Figure 8.11 provides the results associated with the analysis of pavement responses under various types of contact stresses for the described SHL case. Similar to the preceding analysis for standard axle, the SHL axle results indicated that inclusion of the non-uniform distribution of contact stresses in the analysis led to substantially higher pavement responses, compared to the corresponding pavement responses calculated under the uniform contact stresses. This is mainly due to the concentration of the contact stresses in the mid-ribs of the tire that tend to be significantly higher than the average values along the tire footprint contact area. This essentially translates into higher level of responses within the pavement structure, which in turn leads to higher predicted damages.

Evidently, the influence of contact stress distributions is more manifest on the calculated surface deflection and subgrade compressive strain, in comparison with the observed effect attributed to the AC tensile strain. Based on the results provided in Figure 8.11, incorporation of non-uniform distribution in instead of uniform distribution of contact stresses in the analysis resulted in an increase in the monitored pavement responses, i.e., surface deflection, subgrade strain, and AC strain, by 25%, 15%, and 5%, respectively. Such high percent variations, i.e., 25% and 15%, in the calculated responses due to inclusion of the non-uniform distribution of contact stresses can greatly affect the pavement performance prediction and damage quantification for pavements subjected to SHL vehicle operations.
Figure 8.10: Contact Stresses under 120-kips SHL Axle Tire: (a) Non-uniform Distribution, and (b) Uniform Distribution.
Figure 8.11: Influence of Non-Uniform Distribution of Contact Stresses under 120-kips SHL Axle on the Pavement Responses: (a) Surface Deflection, (b) Compressive Strain at the Top of Subgrade, and (c) Vertical Stress at the Top of Subgrade.

The numerical simulation results in this section underscored the significance of realistic simulation of the non-uniform contact stress distributions for proper assessment of the pavement performance under the operating truck traffic. This was even more pronounced for analysis of the SHL vehicles with demanding loading conditions, as the pavement responses under such SHL cases were found to be more sensitive to the non-uniform distribution of contact stresses, compared to the corresponding responses under standard trucks with conventional loading configurations. Consequently, the analysis and design of pavements accommodating OW and SHL vehicles should properly account for the realistic simulation of the non-uniform contact stress distributions, rather than relying on simplifying assumptions such as using uniformly distributed loads.

8.3. **Major Advantages of the Proposed Numerical Modeling Approach**

The FE numerical modeling code developed in this study makes adequate provisions for advanced modeling of moving SHL vehicles to accurately calculate the induced pavement responses under complex loading conditions. The proposed numerical modeling approach accounts for the viscoelastic nature of the AC layer, non-uniform distribution of tire-pavement contact stresses,
various SHL-vehicle speeds, and acceleration/deceleration forces to realistically simulate the
dynamic nature of the moving SHL vehicles; while several of these analysis components were
often overlooked in the literature.

In addition to the elaborated advantages, the developed FE code is instrumental in
simulating several prominent features associated with the roadway geometric characteristics such
as super-elevation and curve radius, sloped pavement shoulders, utilities buried beneath the
pavement surface with proper soil-structure interactions, and environmental conditions. Table 8.1
summarizes the major improvements achieved by our proposed approach in the numerical
simulation stage over the preceding analysis approaches to bridge the persistent gap for analysis
of the SHL vehicle structural impacts.
### Table 8.1: Major Advantages of the Proposed Numerical Modeling Approach

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Proposed Approach</th>
<th>Previous Approaches in Literature</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Loading Conditions</strong></td>
<td>• Considering the site-specific Axle Load Spectra (ALS) database</td>
<td>• Not capable of working with Axle Load Spectra database</td>
</tr>
<tr>
<td></td>
<td>• Realistic simulation of the SHL tires, considering the frictional interactions</td>
<td>• Not capable of simulating the individual SHL tires, and the frictional interactions</td>
</tr>
<tr>
<td></td>
<td>• Realistic simulation of the non-uniform tire-pavement contact stresses</td>
<td>• Simulating a circular uniformly distributed load</td>
</tr>
<tr>
<td></td>
<td>• Considering the acceleration/deceleration of SHLs</td>
<td>• Not capable of evaluating the acceleration/deceleration effects</td>
</tr>
<tr>
<td><strong>Evaluated Pavement</strong></td>
<td>• Surface deflection</td>
<td>• Surface deflection</td>
</tr>
<tr>
<td><strong>Responses</strong></td>
<td>• AC tensile strain</td>
<td>• SG vertical stress</td>
</tr>
<tr>
<td></td>
<td>• SG vertical stress</td>
<td></td>
</tr>
<tr>
<td><strong>Environmental Factors</strong></td>
<td>• Considering the influence of environmental factors such as: precipitation rate, Ground Water Table, and seasonal climate variations</td>
<td>• Overlooking the influence of environmental factors</td>
</tr>
<tr>
<td><strong>Roadway Characteristics</strong></td>
<td>• Considering the super-elevation and roadway curvature</td>
<td>• Overlooking the geometric characteristics of roadway curved segments</td>
</tr>
<tr>
<td></td>
<td>• Realistic simulation of the sloped pavement shoulders</td>
<td>• No capable of simulating the sloped shoulders</td>
</tr>
<tr>
<td><strong>Stability Analysis</strong></td>
<td>• Probabilistic approach to account for the uncertainties of Mohr-Coulomb shear strength parameters</td>
<td>• Overlooking the uncertainties associated with Mohr-Coulomb shear strength parameters</td>
</tr>
<tr>
<td><strong>Buried Utility Risk</strong></td>
<td>• Realistic simulation of the buried utilities</td>
<td>• Overlooking the interactions between buried pipe and subgrade soil</td>
</tr>
<tr>
<td><strong>Analysis</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Chapter 9: Development of Framework for Pavement Damage Quantification

9.1. **Introduction**

As stated earlier, several states have experienced significant increase in energy-related activities such as natural gas and crude oil productions over the last decade. Despite many positive economic impacts, these energy development activities have created large volumes of OW and SHL truck traffic operations in the network, which adversely affected the longevity of transportation infrastructure systems such as pavements and bridges. Damaged local and county roads have been a major source of inconvenience for the local residents in such states. Due to the sudden explosion of drilling activities, the local government agencies and the TxDOT were not able to ramp up their pavement preservation and maintenance efforts to meet the unexpected demand. Lack of funding resources coupled with unclear guidelines are among the many elements that contribute to the delay of the pavement maintenance and repair in several counties across the state. The quantification of the energy development impacts and taxing loading conditions on the highway network is the prelude to adopting proper rehabilitation strategies to meet the future growth of traffic in energy corridors. This can effectively protect the taxpayers’ resources spent on transportation systems each year. Consequently, there is a pressing need to accurately quantify the damages imparted by the OW and SHL truck operations in the affected networks.

The primary step in the quantification of the pavement damages is to have an accurate account of the loading conditions in the network. Traditionally, once the traffic loads and the distributions are determined, the results are converted into the standard 18-kip single axle using the Equivalent Axle Load Factors (EALF). EALFs essentially allow for the quantification of the pavement damages per pass relative to a standard 18-kip single axle. The EALF values are primarily dependent on the type of pavement, pavement layer profile, and material properties of
the pavement layers in the network. The traditional industry-standard EALFs are based on the AASHTO formulations with several simplifying assumptions for generalization across the nation. The major problem that pavement designers and other professionals face is that the analysis procedures rely on experimental information that was developed from field measurements in the late 1950s and early 1960s, with revisions later in 1993. Additionally, the majority of the road tests were conducted in regions with distinct climatic conditions, limited pavement profiles, and unrepresentative traffic characteristics which ultimately jeopardizes the generalization of the EALFs for nationwide use. Therefore, the industry-standard EALF tables might be relevant for similar circumstances under which the tests were conducted; however, the pavement design engineers should proceed with caution if they intend to use the default EALF values in cases with major departure from the original assumptions under which the EALF tables were developed. This anomaly is more pronounced in energy corridors with taxing loading conditions. Considering the fact that each network has its own specific characteristics, this was the motivation for the authors to develop a mechanistic framework for the determination of site-specific EALF tables that is commensurate with the traffic characteristics such as loading magnitude, frequency, vehicle characteristics such as axle configurations and tire footprint, environmental conditions, pavement type, and layer configurations in this study.

9.2. Literature Review

Several researchers studied different methodologies to quantify the damages imparted by heavy vehicles. Batioja-Alvarez et al. (2018) developed a probabilistic approach to quantify the pavement damages due to OS/OW trucks in Nevada. The authors determined the EALF values based on fatigue and rutting criteria. The authors reported that fatigue-based EALFs attributed to the OW vehicles were higher than rutting-based EALFs. Subsequently, the authors determined the
pavement damage associated costs as a means to quantify the financial ramifications of the OW/OS truck operations in Nevada. In the aforementioned study, the researchers used records of issued permits as the primary source to characterize the traffic distributions in the network. Banerjee and Prozzi (2015) proposed a mechanistic approach to estimate the deterioration of the pavement structure subjected to traffic loads. The authors developed a methodology for the determination of axle group equivalency factors and individual axle load factors. Evidently, they reported that EALFs were significantly affected by axle configuration and the distribution of loads on an axle group. The authors in turn used this information to propose a permit fee structure for OW trucks. In a relevant study, Banerjee et al. (2013) developed a methodology to determine axle-specific damage equivalency factors associated with rutting, fatigue cracking, and roughness criteria. The authors concluded that except for the case of rutting, damage equivalency factors were independent of the material properties and structural capacity of the studied pavement sections.

Sadeghi et al. (2007) evaluated the influence of overload truck traffic on the service life of flexible pavements. The authors developed a deterioration model to serve as a precursor to a cost recovery algorithm. They found that in most cases, the asphalt layer tensile strain, that controls the fatigue performance of the flexible pavements, was the critical parameter that contributes to the deterioration model. Wu et al. (2017) proposed a GIS-based routing assistance tool to optimize OW routes based on the historical data, projected heavy traffic level, pavement condition, and climatic effects in Texas. The authors indicated that in short term analysis, pavement sections in cold and dry climates performed better compared to sections located in hot and humid climates. Some researchers adopted the Pavement Condition Score (PCS) in lieu of axle load equivalency factors to assess the performance deterioration in OW corridors (Robinson et al., 1996). The pavement score takes into consideration both the Distress Score (DS) and the International
Roughness Index (IRI), to provide a qualitative means to describe the ride quality and pavement conditions (Gharaibeh et al., 2012). Though not intended as a mechanistic indicator of the distress progressions in pavements, some researchers have preferred its use as an acceptable procedure to quantify pavement damages when traffic distributions and pavement structural data are available.

Zhao and Wang (2015) developed a methodology to evaluate the costs associated with the pavement deteriorations based on the mechanistic determination of the load equivalency factors in New Jersey. The authors used WIM data and incorporated axle load spectra in routine pavement design software to determine the mechanistic load equivalency factors. The authors indicated that axle configurations had major contribution to the calculations of the pavement damages in the overload zones. Chatti et al. (2009) performed a study in Michigan to evaluate the effect of heavy multi-axle trucks and their influence on pavement damages. The authors determined the EALFs from laboratory characterization of pavement layer materials and further analysis of representative pavement sections to calculate Truck Factors (TF). Subsequent to the calculation of truck factors, the authors indicated that the damages due to excessive surface rutting are primarily associated with the passages of multi-axle OW trucks in Michigan. They also reported that the calculated EALFs, were higher than those from AASHTO, especially for less robust pavement profiles.

The majority of the proposed damage quantification approaches either assume traffic distributions based on available data, use default values in pavement design software, or tend to predict it based on a secondary means such as issued permits or citation records. The inaccuracies pertaining to unrealistic traffic characteristics can potentially induce a systematic error for the determination of the damage equivalency factors and remaining life analysis of transportation facilities.
In addition to the highlighted shortcomings, issues such as inherent limitation of the preceding methodologies for calculation of the EALF for SHLs with non-conventional axle arrangements, as well as unrealistic simulation of tire-pavement contact stresses, motivated the authors to develop a novel damage quantification algorithm. The proposed all-encompassing protocol is based on the field-derived ALS in several FM roads, State Highways, and US Highways in Texas, for calculation of site-specific damage factors and realistic assessment of pavement deteriorations imparted by OW and SHL trucks in corridors with taxing truck operations.

9.3. **Objective**

The primary objective of this chapter is to develop a mechanistic framework for the quantification of the pavement damages associated with OW and super heavy truck operations in overload corridors. The proposed approach allows for the calculation of the damage equivalency factors tailored towards the specific characteristics of three different categories of SHL vehicles operating in the network with consideration of environmental conditions and unique features of the transportation systems. The modified axle load factors were further contrasted with traditional damage equivalency factors to highlight the underestimation of the pavement damages when default values are employed for the remaining life analysis of pavement structures.

9.4. **Current State of Practice**

The EALF for flexible pavements, as established by AASHTO, is defined as follows:

\[
EALF = \frac{W_{t18}}{W_{tx}}
\]  

(10-1)

where:

\[W_{tx} = \text{Number of } x\text{-axle load repetitions after time } t, \text{ and}\]

\[W_{t18} = \text{Number of 18-kip axle load repetitions after time } t, \text{ calculated from Equation 10-2},\]
\[
\log\left(\frac{W_{tx}}{W_{t18}}\right) = 4.79 \log(18 + 1) - 4.79 \log(L_x + L_2) + \log L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}}
\]  

(10-2)

where \(G_t\) and \(\beta_x\) are defined as:

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

(10-3)

\[
\beta_x = 0.4 + \frac{0.081(L_x + L_2)^{3.23}}{(SN + 1)^{5.19}L_2^{3.23}}
\]  

(10-4)

where:

- \(L_x\) = Load in kips on one single axle, one set on tandem axles and one set of tridem axles,
- \(L_2\) = Axle code, 1 for single axle, 2 for tandem axle, and 3 for the tridem axle,
- \(SN\) = Structural number,
- \(p_t\) = Terminal Serviceability,
- \(G_t\) = Function of terminal serviceability, and
- \(\beta_{18}\) = Value of \(\beta_x\) when \(L_x\) is equal to 18-kip and \(L_2\) is one.

As is evident in Equation 10-1, the EALF for each axle load group is a function of the structural number (SN), which in turn is related to stiffness properties of layers, drainage conditions, and the pavement layer thicknesses. For generalization purposes, Asphalt Institute (AI) assumed SN as 5 and terminal serviceability \((p_t)\) as 2.5 for the development of EALF tables for nationwide use. The axle load equivalency tables essentially provide a means to characterize the damages imparted by \(i^{th}\)-axle load group relative to the standard 18-kips single axle on the pavements (10).

There are several sources of inaccuracies and systematic errors associated with such assumptions. The major shortcomings of using the current industry-standard load equivalency factors to quantify relative damages are as follows:
The EALF tables were originally developed for specified SN and terminal serviceability ($p_t$) values based on equations proposed and later modified in the AASHTO road test. Considering the fact that many of the in-service pavements, particularly roadways in the energy developing areas have been subjected to heavy loads for several years and already show visible signs of distresses and deteriorations, the assumption of $SN = 5$ is inherently flawed and will result in underestimation of the damages in the network.

The effect of terminal serviceability and structural number on the value of the EALF in the AASHTO equation is erratic and is not consistent with the theory. Super heavy wheel loads are expected to have significantly higher EALF than unity to indicate more damages compared to the standard 18-kip axle; however, the AASHTO equations predict less damage with lower SN values.

SN is a function of the layer thicknesses, drainage conditions, and the stiffness properties of the layers. This basically indicates that EALF is not a single value and should be different based on the seasonal variations of the material properties and structural features of the pavement systems in the network. Considering the fact that the passage of an OW truck over a thinly surfaced Farm-to-Market (FM) road will potentially induce more damage compared to the passage of the same truck over a well-designed and well-maintained State Highway (SH), using the same EALF for both cases will compromise the accuracy of damage analysis.

In addition to these limitations, issues such as general methodology for the calculations of EALF using AASHTO, coupled with the change in the material properties and loading conditions since the last modification of AASHTO equations, motivated the authors to explore a mechanistic framework for the determination of the axle load equivalency factors in this study.
9.5. PROPOSED APPROACH

Figure 9.1 provides a flowchart of the developed procedure for determination of the modified EALF values based on the parameters and properties directly derived from extensive field data collection efforts. Accurate and realistic characterization of the traffic information and pavement layer properties are integral components of the damage quantification algorithms. The site-specific and field-derived information on traffic loading conditions, as described in Chapters 5 and 7, as well as pavement layer thicknesses and back-calculated layer moduli, discussed in Chapters 4, were utilized to obtain the pavement responses using the 3D FE numerical simulations. Subsequently, critical pavement responses due to passage of SHLs, namely tensile strain at the bottom of Asphalt Concrete (AC) layer, compressive strain at the top of the subgrade layer, as well as the cumulative surface deflections under multiple load groups were determined. The pavement responses were further used in the damage quantification algorithm to calculate EALFs based on fatigue, rutting, and cumulative surface deflection criteria. Ultimately, the highest EALF value for each permutation was selected as the site-specific modified EALF value.
Figure 9.1: Flowchart for the Proposed Mechanistic Approach for the Determination of the Modified Equivalent Axle Load Factor (EALF).

Damage equivalency factors attributed to the various roadway types with similar characteristics, in terms of the functionality and structural pavement profile, were also classified to better represent the damages imparted by overload truck operations. In order to account for the influence of climate on the material properties of the pavement layers, the FWD testing and surface deflection measurements were conducted in both summer and winter months for all field test sections.

9.5.2. Equivalent Axle Load Factor (EALF) Equations

This section provides the rationale for the calculations of the axle load equivalency criteria in this study. Three different approaches were employed in this study to calculate the axle load equivalency factors based on the rutting, fatigue cracking, and cumulative surface deflections.
9.5.2.1. **EALF based on Fatigue Criteria**

In the AI approach, the tensile strain at the bottom of the asphalt layer was selected as the critical response that controls the fatigue performance of the flexible pavements, as shown in Equation 10-5:

\[ N_f = 7.96 \times 10^{-2}(\varepsilon_t)^{3.29}(E_{AC})^{0.85} \]  \hspace{1cm} (10-5)

where the \( N_f \) is the allowable number of load applications to fatigue failure, \( \varepsilon_t \) is the tensile strain at the bottom of the asphalt layer, \( E_{AC} \) is the modulus of the asphalt layer. The equivalent axle load factor for axle load group \( x \) compared to standard 18-kip axle based on the fatigue criteria can be calculated from Equation 10-6 as:

\[ EALF_{fatigue} = \left( \frac{W_{c18}}{W_{c_x}} \right) = \left( \frac{\varepsilon_{c_x}}{\varepsilon_{c_{18}}} \right)^{3.29} \]  \hspace{1cm} (10-6)

9.5.2.2. **EALF based on Rutting Criteria**

The AI rutting model assumes that the asphalt layer and the base layer will not experience any permanent deformation; therefore, all rutting is associated with subgrade permanent deformation (Bahia, 2000). Hence, the compressive strain \( \varepsilon_c \) at the top of the subgrade is assumed to be the controlling factor for the determination of the rutting performance of the flexible pavements. Equation 10-7 defines the rutting performance as:

\[ N_f = 1.37 \times 10^{-9}(\varepsilon_c)^{4.47} \]  \hspace{1cm} (10-7)

where \( N_f \) is the allowable number of load applications to rutting failure. The axle load factor based on the rutting criterion can be calculated from Equation 10-8 as:

\[ EALF_{rutting} = \left( \frac{W_{c18}}{W_{c_x}} \right) = \left( \frac{\varepsilon_{c_x}}{\varepsilon_{c_{18}}} \right)^{4.47} \]  \hspace{1cm} (10-8)
9.5.2.3. **EALF based on the Cumulative Surface Deflection Criteria**

The AI rutting criterion assumed that the asphalt and base layer remains intact during the service life of the pavement, and all the deformation is due to subgrade rutting. This is often not true, as there will be rutting in the asphalt and base layers during the service life of flexible pavements. Therefore, an additional criterion was proposed to incorporate individual layers deformations, manifested as cumulative surface rutting to calculate the axle load equivalency factors. To achieve this objective, the cumulative deflection determined at the surface of the pavement from numerical simulations were used to calculate the deflection-based criteria. Equations 10-9 through 10-11 provide the process for the determination of the deflection-based EALF categorized based on the type of axles as:

\[ N = \left( \frac{1}{D} \right)^{3.8}, \quad (10-9) \]

**Single Axles:**

\[ EALF_{deflection} = \left( \frac{W_{18}}{W_{tx}} \right) = \left( \frac{D}{D_b} \right)^{3.8}, \quad (10-10) \]

**Multiple Axles:**

\[ EALF_{deflection} = \left( \frac{W_{18}}{W_{tx}} \right) = \left( \frac{D}{D_b} \right)^{3.8} + \sum \left( \frac{\Delta_i}{D_b} \right)^{3.8} \quad (10-11) \]

where \( D \) is the surface deflection, \( \frac{D}{D_b} \) is the ratio of pavement surface deflections caused by a single axle load to those calculated under the standard 18-kip axle \( (D_b) \). Furthermore, \( \Delta_i \) represents the difference in magnitude between the maximum deflection calculated under each succeeding axle and the intermediate deflection between axles (Kawa et al., 1998).

Ultimately, the highest EALF value among three criteria was selected as the modified axle load equivalency factor as shown in Equation 10-12:

\[ Modified \ EALF = \text{Max} \ (EALF_{fatigue}, EALF_{rutting}, EALF_{deflection}) \quad (10-12) \]
9.5.3. **Field Measurements**

The authors conducted a series of field experiments to realistically simulate the vehicle characteristics and traffic distributions in OW corridors of the Eagle Ford Shale region. Parameters such as axle configuration, tire spacing, axle spacing, tire-pavement footprint, and tire pressure were of primary interest prior to the P-WIM calibration process.

9.5.3.1. **Axle Configuration**

As mentioned, P-WIM units are able to record different axle spacing corresponding to the vehicles passing over the installed piezoelectric sensors. Table 9.1 provides the average values of successive axle spacing for different axle types based on the field traffic information. It should be noted that the previous studies showed that when the center to center distance between each adjacent axle are more than 60 in., the pavement responses under one of the axles won’t be affected by the adjacent axle load (Oh et al., 2007). Such criteria for axle spacing was used to define the various axle groups of SHL vehicles in the FE analysis. Therefore, the axles with more than 60 in. spacing were simulated and analyzed as separate axles.

<table>
<thead>
<tr>
<th>Axle Type</th>
<th>Axle Spacing (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tandem Axle</td>
<td>51.6</td>
</tr>
<tr>
<td>Tridem Axle</td>
<td>50</td>
</tr>
<tr>
<td>Quad Axle</td>
<td>50.4</td>
</tr>
</tbody>
</table>

9.5.3.2. **Calculation of Tire Pressure and Tire Footprint**

In recent years, tire pressure of the commercial vehicles operating in the transportation systems has increased significantly. In the road test of the AASHTO, tire inflation pressure varied from 75
to 80 psi. Nowadays, trucks usually use tires with pressure ranging from 85 to 115 psi and, in some cases; tire pressures can reach 130 to 145 psi (Filho et al., 2016). According to a previous study by the Florida Department of Transportation (FDOT), tire pressures ranged between 80 and 125 psi (Greene et al., 2010).

Considering the fact that the high tire pressure could greatly impact the pavement responses such as stresses and strains, the research team was motivated to conduct some experimental investigations in the field to calculate the actual tire pressure, instead of assuming a typical value. Moreover, it should be noted that the tire pressure value is linked with the tire footprint area and axle weight and it can be calculated from Equation 10-13 as:

\[ P = \frac{L}{A}, \]  

where \( P \) is the tire pressure, \( A \) is the tire footprint area and \( L \) is the specific axle weight divided by the number of tires in the axle.

Collaborating with TxDOT’s Yoakum District Maintenance Division, the research team measured the tire footprint area for single and tandem axles for Class 6 and 9 trucks using the print of painted tires on the papers, as shown in Figure 9.2 and 9.3. In addition, axle weights were measured using the static scales, as shown in Figure 9.4.
Figure 9.2: Painting Tires for Tire Footprint Measurement.

Figure 9.3: Different Tire Footprints for (a) Class 6, Front Axle, (b) Class 6, Rear Axle, (c) Class 9, Front Axle, and (d) Class 9, Rear Axle.

Figure 9.4: Axle Weight Measurement for (a) Class 6, Front Axle, (b) Class 6, Rear Axle, (c) Class 9, Front Axle, (d) Class 9, Rear Axle.
Table 9.2 summarizes the contact area, axle weight and the tire pressure values for different axle types. Based on the calculated values, 120 psi was selected as the most critical value for the average tire pressure in the finite element analysis.

<table>
<thead>
<tr>
<th>Vehicle Classification</th>
<th>Axle Type</th>
<th>Contact Area (in²)</th>
<th>Weight per Axle Side (lb)</th>
<th>Tire Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 6</td>
<td>Single Axle</td>
<td>51.8</td>
<td>6220</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>Tandem Axle</td>
<td>42.2</td>
<td>9780</td>
<td>116</td>
</tr>
<tr>
<td>Class 9</td>
<td>Single Axle</td>
<td>44.4</td>
<td>5120</td>
<td>115</td>
</tr>
<tr>
<td></td>
<td>Tandem Axle</td>
<td>42.8</td>
<td>9440</td>
<td>110</td>
</tr>
</tbody>
</table>

### 9.5.4. Determination of Pavement Responses

The research team analyzed the pavement structures associated with distinct traffic characteristics and material properties for specific sites to obtain the dominant pavement responses. Figure 9.5 schematically illustrates the locations where the critical responses, described below, are evaluated on a typical pavement structure, using the FE model and analysis:

- Surface deflection at Point A to determine deflection-based EALF,
- Tensile strain at bottom of the AC layer (Point B) to determine fatigue-based EALF,
- Compressive strain at top of subgrade (Point C) to determine rutting-based EALF.

Figure 9.6 provides an example of the pavement response contours obtained from the FE numerical analysis to showcase the response calculation results. The plots pertain to the representative axle arrangement of the SHL case 28A-8T simulated over the pavement structure of SH 123 with the summer-based layers material properties. The demonstrated response contours were essentially instrumental in the quantification of the damages imparted on the pavement facilities by SHL vehicle operations.
9.5.5. Sensitivity Analysis with Respect to the Critical Response Location

Three type of pavement responses were calculated at the critical locations within the pavement structure. Since the critical locations will be a function of the axle configuration and the pavement structure, the research team analyzed different locations of the pavement considering different axle
types, and site-specific pavement layer properties, as shown in Figure 9.7. For instance, four locations were evaluated at the bottom of the HMA layer for the estimation of AC bottom-up fatigue cracking. Pavement responses ($\varepsilon_t$) were obtained for locations 1, 2, 3, and 4 for each axle type as shown in Figure 9.7. The highest value of the pavement response was incorporated in the algorithm for further calculations of the EALF values in this study. Similarly, the research team calculated the other pavement responses at all indicated points and obtained the maximum values corresponds to the critical locations. Generally, critical location is defined as the location where the maximum damage is most likely to occur under that specified point. Table 9.3 summarizes the results associated with the mentioned sensitivity analysis.

Figure 9.7: Sensitivity Analysis with respect to the Critical Response Location.

Table 9.3: Critical Locations of Pavement Responses

<table>
<thead>
<tr>
<th>Pavement Responses</th>
<th>Critical Point for Different Axle Types</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single Axle</td>
</tr>
<tr>
<td>Surface Deflection</td>
<td>2</td>
</tr>
<tr>
<td>AC Tensile Strain</td>
<td>2</td>
</tr>
<tr>
<td>Subgrade Compressive Strain</td>
<td>1</td>
</tr>
</tbody>
</table>
9.6. **Analysis of Results and Discussion**

The mechanistic-based EALF values tailored towards the specific loading configurations attributed to three different SHL categories were calculated. The results were then post-processed and clustered for similar SHL categories due to the unique nature of axle assembly and tire configurations associated with each SHL group to better represent the analysis results. Ten representative pavement sections in San Antonio, Corpus Christi, Yoakum, and Laredo Districts were also incorporated in the numerical simulations to account for different types of roadway facilities in this study. The modified EALF values were also contrasted with traditional Asphalt Institute EALFs for comparison purposes in this research effort. Furthermore, the modified EALF values based on different criteria such as axle load, axle type, GVW, wheel load, roadway type, and season of the year were calculated and the results were provided in this section.

9.6.1. **Pavement Damage Factors for SHL Vehicles under Category (I)**

Pavement damage factors for SHL vehicles that fall under category (I) are provided in this subsection. Based on the P-WIM recorded data, such SHL vehicles comprised of super heavy axles and tires; however, the axle and tire arrangements follow the same configurations as indicated by FHWA truck classification for conventional Class 4-13 trucks.

9.6.1.1. **EALF Values Based on Three Proposed Criteria**

Figure 9.8 presents the results for the calculation of three different measures of axle load equivalency factors for single-axle group for different FM, SH, and US highways. The ascending nature of the EALF values in Figure 9.8 shows the influence of the axle load weights on the calculated damage equivalency factors. As evidenced in this plot, increasing axle load levels results in exponential increase in the axle load equivalency factors, which in turn translates into higher damages imparted on pavement facilities. Figure 9.8 also provides relative comparisons of
different measures of axle load equivalency factors proposed in this study. Evidently, the deflection based and rutting based EALF criteria showed the highest sensitivity to the increasing axle load weights. As evidenced in Figures 9.8(a) to 9.8(c), the deflection based and rutting based EALFs were substantially higher compared to the fatigue criteria. This observation was expected as the damages imparted by SHL vehicles are more relevant to distresses associated with load magnitude such as surface deformations rather than load repetitions as in fatigue-related distresses.

Similar EALF trends were observed for multiple axle load equivalency factors in this study. However, the analysis of tandem, tridem and quad axles resulted in much higher axle load factors for all three classes of transportation facilities. The corresponding results associated with SH and US highways showed that the surface deflection criteria resulted in the highest EALF values among the three proposed criteria. As stated earlier in the methodology section, the highest value of the EALF among the three criteria is reported as the site-specific and loading group-specific axle load equivalency value for further post-processing and analysis of the remaining life of pavement structures.
Figure 9.8: Mechanistic EALF values for single-axle loads based on three proposed criteria for: (a) FM roadway, (b) SH, and (c) US highway.

Table 3 provides a qualitative summary of the significance levels for three measures of damage factors proposed in this study. The qualitative significance levels were primarily attributed to the ratios of $\frac{EALF_{ij}}{EALF_{Max}}$, where $EALF_{ij}$ is the average axle load equivalency factor for the $i^{th}$ axle group and $j^{th}$ damage factor criteria, and $EALF_{Max}$ is the highest value of the damage equivalency...
factor calculated for corresponding loading group and roadway type. The load-specific and site-specific damage factors $EALF_{ij}$ exceeding 90% of the maximum EALF value was qualitatively ranked as “significant”, while variants with $\frac{EALF_{ij}}{EALF_{Max}}$ ratios below 0.7 were considered as “not significant” in ranking order presented in Table 9.4.

Table 9.4: Synergistic Influence of Roadway and Axle Types on Damage Equivalency Factors

<table>
<thead>
<tr>
<th>Roadway Type</th>
<th>Single Axle</th>
<th>Axle Type</th>
<th>Multiple Axles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Surface Deflection</td>
<td>Rutting</td>
<td>Fatigue</td>
</tr>
<tr>
<td>FM</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SH</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>US</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Legend*
- **Significant**: $\frac{EALF_{ij}}{EALF_{Max}} > 0.9$
- **Moderately Significant**: $0.7 < \frac{EALF_{ij}}{EALF_{Max}} < 0.9$
- **Not Significant**: $\frac{EALF_{ij}}{EALF_{Max}} < 0.7$

9.6.1.2. **Modified EALF Values Based on Various Roadway Types**

The modified EALF values attributed to the various roadway types (FM, SH, and US highways) in the network were contrasted with each other for comparison purposes in Figure 9.9. The results showed that in all axle types, FM roadways with less robust pavement profile had the highest damage factors among the three roadway types. Conversely, SH and especially US roadways with more robust layer configurations, with average asphalt thicknesses exceeding 5.5 inches in the surveyed network, had the lowest EALFs. This underscores the significance of the pavement profile on the mechanistic-based damage equivalency factors. Therefore, it is imperative to cluster and differentiate between different types of roadways, such as FM, SH, and US roadways, to realistically represent the damages imparted by SHL truck operations.
As illustrated in Figure 9.9, to further clarify the significance of the site-specific damage factors, modified EALF values were also compared with the traditional Asphalt Institute EALFs, commonly used by the pavement design engineers. As evidenced in Figure 9.9, the traditional EALF values were substantially lower than the mechanistic damage factors developed in this study. Such underestimation of the axle load factors can potentially jeopardize the pavement design and rehabilitation plans in OW corridors.

Table 4 presents a summary of the differences between the site-specific and load-group specific EALF values. As evidenced in Table 9.5, the mechanistic EALF values substantially deviate from the traditional industry-standard axle load factors currently employed by the pavement design industry.

Table 9.5: TABLE 4 Percent Difference between Traditional and Modified EALF Values

<table>
<thead>
<tr>
<th>Roadway Type</th>
<th>Axle Type</th>
<th>Single</th>
<th>Tandem</th>
<th>Tridem</th>
<th>Quad</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM</td>
<td></td>
<td>35</td>
<td>47</td>
<td>75</td>
<td>39</td>
</tr>
<tr>
<td>SH</td>
<td></td>
<td>15</td>
<td>18</td>
<td>28</td>
<td>20</td>
</tr>
<tr>
<td>US</td>
<td></td>
<td>6</td>
<td>4</td>
<td>14</td>
<td>8</td>
</tr>
</tbody>
</table>
Figure 9.9: EALF values for different roadway types in the network and different axle types: (a) single axle, (b) dual axle, (c) tridem axle, and (d) quad axle.
9.6.1.3. Influence of Climate on the Modified EALF Values

The research team conducted two series of field data collection in the summer and winter months to study the influence of environmental factors on the axle load equivalency factors. The two sets entail installation of the P-WIM devices to account for the variations in traffic patterns in the network. Additionally, the authors conducted FWD testing of ten representative sites to account for the seasonal variations of the back-calculated modulus values of pavement sections. Evidently, due to the temperature dependency of the material properties in asphalt layers, and the stiffness softening of the base and subgrade soils due to moisture ingress in wet seasons, the structural capacity of multi-layer pavement structures is not monolithic throughout the year. Therefore, the responses under taxing loading conditions, and consequently, the damage factors are affected by the time of the year and seasonal variations of the layers material properties. This was the motivation to deploy P-WIM devices and conduct NDT field testing in summer of 2018 and winter of 2019, to investigate the influence of environmental factors on the damage equivalency factors.

Figure 9.10 provides the axle load equivalency factors for the summer and winter seasons in FM roads for different axle groups in the surveyed network. The results are also contrasted with the traditional industry-standard EALF values for comparison in Figures 9.10 (a) to 9.10 (d). As evidenced in these figures, the damage factors derived from the numerical simulations with June and July back-calculated material properties were substantially higher than the other counterparts. This is primarily attributed to the viscoelastic nature of the asphalt layer, and softening of the surface layers due to elevated temperatures in summer seasons. The variations in the material properties of the layers in summer and winter seasons essentially translate into various degrees of damages imparted by SHL vehicles. Consequently, the damage factors should also manifest such
seasonal sensitivity for accurate assessment of distresses in the highway network. Another noteworthy observation was that the modified EALF values for both summer and winter seasons were substantially higher than the industry-standard damage factors. Such underestimation of the damage factors can potentially incur systematic errors for the design and life-cycle cost analysis of pavement sections (Morovatdar et al., 2020c; 2019).
Figure 9.10: Modified EALF values in FM roadway for different seasons and different axle types: a) single axle, b) dual axle, c) tridem axle, and d) quad axle.
Similar analyses were conducted on representative State Highway and US Highway sections in this study. Table 5 provides summary of comparisons between the summer-based and winter-based EALF values, categorized based on the axle group and roadway type in this study. The differences between the two field trials, reported in percentages, underscores the significance of the variations in material properties and its contribution to the damage calculations in energy corridors. As indicated in Table 9.6, the difference between the summer-based and winter-based EALFs range from 5 percent to 25 percent. Such deviation is more pronounced for less robust pavement structures such as FM roads. However, the seasonal sensitivity of the damage factors is less significant for well-designed and well-maintained US Highways.

Table 9.6: Percent Differences between Summer and Winter-Based EALFs

<table>
<thead>
<tr>
<th>Roadway Type</th>
<th>Percent Difference (%) for Various Axle Types</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single</td>
</tr>
<tr>
<td>FM</td>
<td>18</td>
</tr>
<tr>
<td>SH</td>
<td>8</td>
</tr>
<tr>
<td>US</td>
<td>8</td>
</tr>
</tbody>
</table>

9.6.1.4. **Load Equivalency Factors Based on GVW**

Load Equivalency Factors (LEFs) based on GVW were also calculated, which is defined as the damage per pass to a pavement by a SHL vehicle relative to the imparted damage per pass by a Class 9 reference vehicle with GVW of 80 kips. To further represent the calculated LEF values, damage factors attributed to the SH 123, as a heavily trafficked highway in Corpus Christi District are used as an example in this section. As expected, the most critical criteria for calculation of the load equivalency factors for SHLs was the surface deflection criteria. This is because the SHL vehicles typically consisted of several multiple axles, and as mentioned surface deflection-based equivalency factors are significantly higher than the two other criteria under such conditions.
Figure 9.11 shows the LEFs associated with the SHL vehicles based on different GVW values recorded by the P-WIM units. Based on the analyzed results provided in the plots, SHL trucks can potentially impose significant damages on the pavement sections, due to more taxing stress paths. For instance, based on the deployed mechanistic approach, one passage of a SHL with GVW of 364 kips on SH123 in Corpus Christi can potentially induce 125 times the damage imparted by an 80-kips reference vehicle. Another noteworthy observation was that the traditional Asphalt Institute method presented considerably lower values for LEFs. According to the traditional industry-standard values, one passage of a 364-kips SHL vehicle equals to 35 passes of a reference vehicle. In other words, the use of traditional axle equivalency factors for SHL trucks passing the SH123 can potentially underestimate the damages by approximately 257%. Consequently, the detrimental impacts of SHLs on the pavement structures are more pronounced, and traditional damage quantification approach is not capable of assessing the imparted damages.

Figure 9.11: Load Equivalency Factors (LEFs) for Category (I) SHL Vehicles.

9.6.2. Pavement Damage Factors for SHL Vehicles under Category (II) and (III)

Currently, there is a lack of mechanistic means for quantification of the pavement damages imparted by non-conventional SHL vehicles consisting of multi-axle trailer units that fall under
categories (II) and (III). This was the motivation for the authors to conduct another series of numerical simulations to analyze such SHL vehicles operating in the network, using the developed damage quantification algorithm. It is worth noting that in addition to the GVW, axle configurations and vehicle wheel loads, are the key components that greatly contribute to the level of damages induced by multi-axle SHL trailers. Review of the vehicle plans revealed that SHL vehicles cannot be categorized into specific groups with identical axle loading configurations. Therefore, it deems necessary to account for the non-generic nature of axle configurations when assessing the induced damages by SHL vehicle movements. To incorporate the most demanding loading configurations in this analysis, the authors evaluated different SHL cases. Based on the results provided in Chapter eight, Section 8.5.3, the SHL vehicle 28A-8T was selected as the most critical SHL case with demanding axle assembly and tire arrangement in this study.

The researchers then performed a comprehensive sensitivity analysis to investigate the influence of the heaviest wheel load on the damage imposed by multi-axle SHL units. To accomplish this objective, initially, the research team incrementally increased the load magnitude on each individual tire from 6 kips to 12 kips. Subsequently, the pavement responses and the associated damages under the representative elements of SHL case 28A-8T were calculated. Thus, the damages accumulated under the entire SHL vehicle can be further predicted by superimposing the damages calculated under the representative axle assembly.

Similar analyses were also performed by changing the type of roadway facility. Therefore, different roadway types, i.e., FM, SH, and US highways, were considered in the analysis representing thin, intermediate, and thick AC pavements, respectively. Additionally, the FWD back-calculated material properties of the layers associated with the summer and winter months were incorporated into the analysis to assess the influence of environmental factors on the
cumulative pavement damages. The obtained results, as well the pertinent discussions, are provided in the following sections.

**9.6.2.1. Influence of Wheel Load on Pavement Damage Factors**

Figure 9.12 provides the variations of the three calculated measures of pavement damage equivalency factors in state highways for multi-axle SHL vehicles with changing the wheel load magnitudes. As evidenced in this plot, the deflection based damage equivalency factors showed the highest sensitivity to the increasing wheel load weights, followed by rutting based damage factors; while fatigued based damage factors showed minor variations with increasing the wheel load magnitude. As stated earlier in the methodology section, the highest value of the EALF among the three criteria is reported as the damage equivalency factors for further post-processing and analysis of the pavement structures. It should be also noted that similar trends were observed for representative FM and US highways evaluated in this study.

Figure 9.12: Influence of Wheel Load on Pavement Damage Equivalency Factors for Multi-Axle SHL Vehicles.

Another noteworthy observation from Figure 9.12 pertains to the relevance of the wheel load magnitude to the quantified damage. Evidently, increasing the load magnitude on each
individual tire results in substantial increase in the damage equivalency factors, which in turn translates into higher level of damages imparted on pavement facilities. Based on the results provided in Figure 9.12, one passage of the studied SHL nucleus with 6 kips wheel load on state highways can potentially induce around 8 times the damage imparted by 18-kips reference axle. However, if wheel load of 12 kips is incorporated into the analysis algorithm, the SHL element can impose significantly higher level of damages, i.e., 98 times the damage imparted by the reference axle. Such alarming increase in the imparted damage, by \( \frac{98}{8} = 12.3 \) times, underscored the significance of the heaviest wheel load magnitude and its role on pavement damage quantification during SHL permitting issuance procedure.

**9.6.2.2. Influence of Climate on Pavement Damage Factors**

The pavement damage factors attributed to the various seasons of the year, i.e., summer and winter, in state highways for multi-axle SHL vehicles are contrasted with each other for comparison purposes in Figure 9.13. As evidenced in the plot, the damage factors derived from the numerical simulations with summer-based layer modulus values were substantially higher than the corresponding values with winter-based material properties. Hence, the variations in the material properties of the layers in summer and winter seasons essentially translate into various degrees of damages imparted by SHL vehicles. Such sensitivity was even more pronounced for SHL vehicles with heavier load magnitudes on their tires. Therefore, the damage factors should also manifest such seasonal sensitivity, with consideration of the wheel load magnitude, for proper characterization of the damages imparted in the network.
Figure 9.13: Influence of Seasonal Variations of Material Properties on Pavement Damage Equivalency Factors for Multi-Axle SHL Vehicles with Different Wheel Loads.

9.6.2.3. Influence of Roadway Type on Pavement Damage Factors

Figure 9.14 provides the post-processed results obtained from the numerical simulations for calculation of the damage equivalency factors attributed to the various roadway types, i.e., FM, SH, and US highways. The results indicated that operation of the studied SHL vehicle in FM roadways resulted in higher level of pavement damages compared to the same vehicle operating in State and US highways. This is as expected, since FM roadways with less robust pavement structures are more sensitive to the SHL vehicle operations, compared to the pavement sections in SH and especially US highways with higher structural capacity. More specifically, the detrimental impact of SHL vehicles on FM roadways was more evident for the case scenarios that carry heavier loads on their wheels. Accordingly, the results highlighted the importance of the pavement profile and wheel load magnitude on the analysis of the SHL vehicles with multi-axle trailer units for realistic assessment of the imparted damages.
9.6.2.4. Increase in Pavement Damage Factors for Heavy Wheel Loads

To clarify the influence of wheel load magnitude on the level of damages imposed by SHL vehicles, the researchers analyzed the pavement damage factors for heavy wheel loads exceeding 6 kips and contrasted the corresponding values with 6 kips based damage factors, which serves as the benchmark in this study. The analysis also considers the SHL vehicle operations under different conditions such as moving at different roadway types, and under different seasons of the year.

Table 1 provides a quantitative summary of the proportional increases in damage factors for multi-axle SHL trailers when wheel loads heavier than 6 kips limit are incorporated into the damage assessment protocol. The results are reported as “ratios of damage factors”, i.e.,

\[
\frac{EALF_{WL > 6-kips}}{EALF_{WL = 6-kips}}
\]

where \(EALF_{WL > 6-kips}\) is the damage equivalency factor based on the wheel load that exceeds the 6 kips limit, and \(EALF_{WL = 6-kips}\) represents the damage factor for the same SHL unit with wheel load that equals to 6 kips. The results demonstrate great relevance of the load magnitudes on the vehicle tires to the level of damage imparted on pavements due to SHL vehicle operations. As indicated in Table 9.7, increasing the wheel load from 6 kips to 12 kips resulted in
drastic increase in the imparted pavement damages by 7-12 times, depending on the roadway structural characteristics, and climatic conditions. The results indicated in Table 9.7 provide a mechanistic means for pavement damage assessment in overload corridors subjected to SHL vehicle movements. Furthermore, the presented color contour table facilitates the SHL evaluation process, and can be further instrumental by state highway agencies for approval (or rejection) of the SHL permits, considering the loading conditions, characteristics of the pavement facility, and the maximum allowable level of damage set forth by regulatory agencies.

Table 9.7: Increase in Pavement Damage Equivalency Factors for Multi-Axle SHL Vehicles with Wheel Load Exceeding 6,000 lb. Limit.

<table>
<thead>
<tr>
<th>Wheel Load (lb.)</th>
<th>Summer</th>
<th>Winter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FM</td>
<td>SH</td>
</tr>
<tr>
<td>7,000</td>
<td>1.74</td>
<td>1.71</td>
</tr>
<tr>
<td>8,000</td>
<td>2.82</td>
<td>2.72</td>
</tr>
<tr>
<td>9,000</td>
<td>4.32</td>
<td>4.11</td>
</tr>
<tr>
<td>10,000</td>
<td>6.35</td>
<td>5.93</td>
</tr>
<tr>
<td>11,000</td>
<td>8.97</td>
<td>8.25</td>
</tr>
<tr>
<td>12,000</td>
<td>12.53</td>
<td>11.16</td>
</tr>
</tbody>
</table>

The parametric analysis presented in this section indicated that the heaviest wheel load is the major component that substantially contributes to the damages imparted over the pavement design life. The numerical simulation results also highlighted the importance of the structural properties of roadway pavements and the climatic factors for realistic assessment of the pavement performance under the SHL vehicle movements. Consequently, damage quantification procedures
for the pavement structures servicing the SHL vehicles should properly account for the synergistic influence of the major components such as wheel load magnitude, seasonal variation of material properties, and type of roadway facility, for accurate assessment of the distresses in the highway network.

9.7. **SUMMARY OF THE MAJOR POINTS**

A novel framework to quantify the pavement damages associated with the SHL vehicle operations in overload corridors was developed in this chapter. The proposed methodology provides a mechanistic approach based on three different measures of axle load equivalency factors for the determination of the damage equivalency factors tailored towards the specific SHL categories, vehicle wheel loads, axle load groups, roadway type, and seasons. The adopted mechanistic approach accounts for the environmental impacts and unique features of the transportation systems for accurate assessment of the pavement damage factors.

The SHL representative elements, as well as other prominent features in terms of traffic loading conditions and pavement structural properties, were in turn incorporated in a series of advanced numerical simulations for the determination of the pavement responses subjected to taxing traffic conditions. Subsequently, the site-specific damage equivalency factors tailored towards the specific loading configurations of the SHL vehicles operating in the studied network were calculated. The results were then post-processed and clustered for similar SHL categories due to the unique nature of axle assembly and tire configurations under each SHL group to more realistically represent the analysis results. The major findings of this research effort are summarized as follows:

- Analysis of the mechanistic damage equivalency factors confirmed that the modified EALF values were substantially higher than traditional industry-standard axle load factors
currently employed by the pavement design industry. Adherence to the traditional damage factors can potentially result in underestimation of the pavement damages imparted by OW vehicle in energy zones.

- The EALF values based on surface deflection and rutting criteria were highly sensitive to the increasing axle load magnitude in numerical simulations. This behavior was anticipated, as the distresses associated with heavy truck operations are mainly attributed to the load magnitude rather than the load repetitions. Therefore, rutting and surface deflection criteria led to higher damage factors in comparison with fatigue criterion.

- Analysis of various SHL categories indicated that SHL vehicles with multi-axle trailer units that fall under categories (II) and (III) had lower damage factors per tonnage compared to FHWA SHL trucks under category (I). This is mainly associated with the distribution of axle loads over multiple wheel arrangements, and therefore lower load magnitude on each individual tire and lower pressure exerted at the tire-pavement interface.

- A sensitivity analysis of various influential parameters indicated that the heaviest wheel load is the major component that substantially contributes to the damages imparted over the pavement facilities due to multi-axle SHL operations. Analysis of 10 representative pavement sections in energy sectors indicated that increasing the load magnitude on tires from 6 kips to 12 kips resulted in drastic increase in the imparted pavement damages by 7-12 times, depending on the roadway structural characteristics, and climatic conditions.

- Overlooking the influence of the type of roadway facility in damage quantification can potentially incur systematic errors for accurate assessment of the service life of the pavements in energy corridors. Therefore, it deems necessary to cluster similar highways,
in terms of functionality and structural layer profile, to realistically represent the damages imparted by SHL vehicle operations.

- The detrimental effect of SHL vehicles was higher for pavements with lower structural capacity such as FM roads serving the oil wells. This is in line with expectations due to the fact that super heavy loads are more destructive to less robust pavement profiles such as FM roads than in US roadways. Based on the analysis provided in this research, FM roadways had the highest damage equivalency factors, followed by SH and ultimately US highways.

- Accurate assessment of the damage factors should include seasonal variation of the material properties of pavement layers. This is primarily attributed to the viscoelastic behavior of the asphalt layers and the variations of stiffness properties of granular layers due freeze-thaw effects or changes in the saturation state of the unbound granular layers due moisture infiltration or evapotranspiration during the service life of pavements. Therefore, incremental assessment of the damages, rather than single value average damage factor, better represent the detrimental influence of SHL vehicles in highway networks.

- Damage quantification procedures for pavement facilities servicing the overload corridors with significant frequencies of SHL movements should properly account for the simultaneous effects of the key components such as wheel load magnitude, seasonal variation of material properties, and type of roadway facility, for accurate assessment of the distresses in the highway network.
Chapter 10: Remaining Service Life Analysis

10.1. INTRODUCTION

Recent developments in the energy production activities on one hand, and the increase in freight transportation due to improvements in economic activities of the states on the other, have resulted in unprecedented operations of SHL vehicles in several states across the nation. In Texas, the SHL vehicle operations pertaining to the energy development activities have substantially impacted Southern and South East Districts such as Corpus Christi, Laredo, Yoakum, and San Antonio Districts. Figure 10.1 illustrates comparisons of projected traffic for ten representative pavement sections in this study. This plot provides comparative cumulative ESALs for pre-energy development in 2008, and the corresponding ESALs in 2019 for ten studied roadways.

![Figure 10.1: Cumulative 18-kip ESAL values over 20-year Design Life for Representative Roadways in the Eagle Ford Shale Region.](image)

Based on the analysis of the P-WIM obtained data, FM468 roadway in Laredo District, and US281 and SH123 highways in Corpus Christi District have undergone a considerable increase in truck traffic (2400%, 228%, and 790% increase, respectively) in the past decade. This alarming
increase in the traffic volume can potentially be the culprit for the distresses observed in FM roadways and US highways in the network.

The service life of the highway pavements are affected not only by the traffic volume (ESAL repetitions), but also by axle weights. Based on the post processing of the P-WIM traffic data, on average 32% of the truck traffic in FM468 exceed the Texas permissible axle weight load limits. SH 123 and US281 highways also had alarming OW percentages of 36% and 45%, respectively. The taxing magnitude and frequency of such loading scenarios resulted in substantial damages to the transportation facilities.

In addition to damaging highways designed to carry legal loads of up to 80,000 lb. GVW, heavy trucks used by the energy companies are also traveling over Load-Zoned (LZ) roads, which are designed to accommodate vehicles that weight less than 58,420 lb. These roads are not designed to withstand such heavy loads; therefore, even a few passages of SHL vehicles will cause permanent damage and consume the life of the pavement.

The loss of service life of the pavements is more evident in FM roads in the energy developing areas, since these rural roadways were never designed to carry such high truck traffic volumes and heavy loads. Based on the research team’s distress surveys of the network, many of such roadways have suffered severe distresses. For instance, the pavement condition survey of FM468 in Laredo and FM99 in San Antonio Districts indicated that these FM roadways were in a severely distressed state exhibiting deep rutting, severe flushing, pothole formation, and with several patched areas, as shown in Figure 10.2.
Figure 10.2: Deteriorated FM Roadways in the Eagle Ford Shale Region.

The maintenance and repair of FM roadways are a major concern for state highway agencies. In Texas, routine maintenance costs on FM roadways have increased from $500 to $1,500 per centerline mile to $35,000 to $45,000 per centerline mile due to the energy development activities (Epps and Newcomb, 2016). Repair costs for state and local government roadways have been estimated at 2 billion dollars per year. Due to the sudden explosion of drilling activities, the local government agencies and TxDOT were not able to ramp up their pavement preservation and maintenance efforts to restore the ride quality of the damaged pavements. Lack of funding resources coupled with unclear guidelines are among the many elements that contribute to the delay of the pavement Maintenance and Rehabilitation (M&R) in several counties across the state.

The quantification of the heavy vehicle impacts on the pavement service life is the prelude to develop uniform compensation strategies to offset the costs incurred to the pavement facilities. Consequently, there is a pressing need to properly estimate the service life of the pavements subjected to heavy truck traffic operations. The analysis of the remaining service life of pavements can also provide insights for state highway agencies and other jurisdictions to adopt commensurate
rehabilitation strategies to preserve the existing assets in the affected networks with demanding traffic makeup.

10.2. BACKGROUND

An accurate account of the traffic distribution patterns is the precursor for reliable prediction of the service life of the pavements subjected to heavy traffic loads. Traditionally, the analysis of service life is based on the ESAL concept, in which the influence of various truck axle configurations and loading magnitudes with their temporal variations on the pavement damages is overlooked. In this approach, the traffic mix is converted to 18-kips standard single axles. The pavement service life is then estimated by contrasting the projected ESALs over the design period with the allowable number of 18-kips axle load repetitions. Commonly, the ESALs are calculated using the empirical AASHTO formulations and the traditional industry-standard damage equivalency factors with several simplifying assumptions for generalization across the nation. The major concern for pavement engineers in using this approach is the fact that the damage equivalency factors were developed based on field tests in one location with distinct climate, subgrade soil properties, and limited traffic from field tests in late 1950s and early 1960s, with further revisions later in 1993.

The traditional ESAL approach also overlooks the influence of variations in the material properties throughout the year and its relevance to the incremental damages imparted on pavement facilities in different seasons. The assumptions made in this approach can potentially induce systematic error for the calculation of the pavement damages and distresses during the design life of the pavements. To mitigate this anomaly, new ME design guides incorporated incremental damage concept in combination ALS for better representation of the progression of distresses with
time. This will allow for the determination of incremental damages imparted by a specific vehicle class at a specific timeframe on a pavement section.

The site-specific ALS needed for ME pavement analysis is mainly collected from WIM units in the field. The P-WIM systems are often preferred over WIM stations due to their convenience, cost-efficiency, and flexibility for continuous collection of the traffic data without interrupting the traffic flow in heavily trafficked highways. Additionally, the P-WIM units are capable of collecting reliable and accurate traffic data provided that a verified calibration procedure is implemented upon installation of the piezoelectric sensors in the field. Consequently, several researchers favored P-WIM systems over the traditional stationary WIM devices in order to collect the traffic information required for pavement service life analysis (Ashtiani et al., 2019, and Morovatdar et al., 2020b & c).

10.3. Literature Review

Several researchers studied different methodologies to evaluate the service life of pavements subjected to heavy truck traffic operations. Wang et al. (2015), using WIM traffic data, investigated the impact of heavy vehicles on the service life of the pavements in New Jersey. Based on numerical simulations, the authors reported that by increasing OW truck frequencies the service life of the pavements can be substantially reduced in their studied network. Rys et al. (2016) assessed the influence of overloaded vehicles on fatigue life of pavements based on the WIM data. The authors converted the traffic mix to the equivalent number of standard axles to characterize the traffic data in their analysis of OW zones. Based on analyzed sections in their study, 20% increase in OW truck operations resulted in 50% reduction of the service life of the pavements. Banerjee and Prozzi (2015) developed a methodology for the determination of the consumption of pavement service life due to the OW traffic operations in Texas. The researchers used the ESAL
concept and the pavement damage equivalency factors to characterize the traffic mix in their analysis. The authors in turn used this information to propose a permit fee structure for OW trucks. In a relevant study, Batioja-Alvarez et al. (2018), using the ESAL concept, evaluated the loss of pavement service life imparted by the OW truck operations in Nevada. Evidently, they reported that the reduction of the pavement life was influenced by the environmental conditions and asphalt layer thicknesses. The researchers used records of issued permits as the primary source to characterize the traffic distributions in the network.

Several researchers used PCS and roughness data in lieu of mechanistic-based approaches to assess the performance deterioration and service life reduction of the pavements in overload corridors (Robinson et al., 1996, Al-Suleiman and Shiyab, 2003, Chou et al., 2008, Wu et al., 2017, Al-Qadi et al., 2017, and Romero et al., 2019). The PCS takes into account both the Distress Score (DS) and the International Roughness Index (IRI) to provide a quantitative means to describe the ride quality and pavement conditions. Though not intended as a mechanistic indicator of the distress progressions in pavements, some researchers have preferred its use as a means to predict the service life of pavements in the absence of traffic distributions and pavement structural data. In some states such as Texas, due to the lack of a uniform approach for mechanistic determination of the loss of pavement service life, prioritization of pavement sections to conduct M&R efforts is traditionally based on the PCS and relevant information gathered from visual inspection surveys (Morovatdar et al., 2020e & f).

Recently, several researchers used advanced statistical clustering techniques to categorize pavement sections with similar traffic patterns for further remaining service life analysis (Smith and Diefenderfer, 2010, Haider et al., 2011, Mai et al., 2013, Abbas et al., 2014, Oh et al., 2015, Li et al., 2016, Jasim et al., 2019, and Walubita et al., 2019). The comparative analysis between
different levels for traffic input indicated that the cluster-based ALS can serve as a means for the analysis of the pavement service life in the absence of field ALS database.

The majority of the proposed approaches for predicting the pavement service life either rely on traditional ESAL concepts, or are solely based on the information gathered from the pavement condition surveys. The inaccuracies pertaining to non-mechanistic analyses can potentially jeopardize the accuracy of the damage quantification and remaining life analyses of transportation facilities. Another anomaly persistent in the literature pertains to the incorporation of the unrealistic traffic data, instead of the site-specific ALS, into the ME pavement analysis. Essentially, the majority of the preceding approaches either assume traffic distributions based on the statewide average data, use default values in pavement design software, or tend to predict it based on a secondary means such as issued permits or citation records. Based on authors previous experience with field traffic monitoring in different locations and seasons, the distribution patterns are greatly impacted by socio-economic fabric of regions and may be vastly different from the neighboring network. Therefore, resorting to average values, default statewide ALS database, and permit records, etc. can be detrimental to the accuracy of the analysis of structural impacts of heavy vehicles in the network.

The highlighted issues were the motivations for our research team to devise an all-encompassing protocol to mechanistically characterize the pavement service life based on the ALS directly derived from field deployment of P-WIM devices in overload corridors.

10.4. Objective

The primary objective of this chapter is to develop a mechanistic framework for the quantification of the remaining service life (RSL) of pavements subjected to the heavy truck traffic operations in overload corridors. The proposed approach accounts for specific characteristics of vehicles
operating in the network with consideration of environmental conditions and unique features of transportation systems. The traffic loading characteristics, pavement layer properties, and climatic data are incorporated into a mechanistic pavement design system to predict the distresses imparted during the design life of pavements. Furthermore, using the relevant statistical techniques, the predicted rutting values from the proposed mechanistic approach are contrasted with those obtained from field distress surveys of the representative sections to cross-validate the accuracy of the proposed algorithm for RSL analysis.

The second section of this chapter discusses the influence of traffic characterization methodology on prediction of the service life of pavements. Similar processes but with different traffic input scenarios, including the site-specific ALS (Level 1), traditional ESAL values (Level 2), and mechanistic software average values (Level 3), are performed for comparison purposes.

Ultimately, the last section of this chapter revolves around developing a framework to estimate the reduction of pavement service life due to drastic changes in the traffic characteristics in the past decade in the studied network. The analysis of service life reduction is the primary step for decision-makers at the network level to adopt proper M&R strategies to meet the future traffic demand in the overload corridors.

10.5. PROPOSED METHODOLOGY

10.5.1. Methodology to Assess Remaining Service Life

Figure 10.3 shows the flowchart of the proposed procedure for the prediction of the RSL, based on the site-specific parameters and characteristics directly derived from extensive field data collection efforts. As illustrated in the figure, the mechanistic-empirical pavement design software employed in this study, namely TxME, requires four main categories of input, i.e., structure, traffic, climate, and performance criteria limits. The pavement structural properties, as well as the
relevant traffic information, were obtained from field testing, as elaborated in Chapters 4, and 5, respectively. The characterized traffic database was in turn converted to traffic mix corresponding to the year of roadway reconstruction to account for traffic load applications immediately after the reconstruction and rehabilitation of roadways in this study. This was accomplished by using the historical growth rates of traffic extracted from TxDOT’s Traffic Count Database System (TCDS). Axle load distributions, truck class distributions (TCDs), average annual daily traffic (AADT), two-way average annual daily truck traffic (AADTT), and operational vehicle speed were the most relevant traffic information that were extracted from the ALS databases.

Figure 10.3: Flowchart for the proposed mechanistic approach for the prediction of the remaining service life of pavements.

The authors further developed a procedure for routine measurements of tire pressure and tire footprint in the calibration process of P-WIM devices to achieve an accurate account of the
loading conditions. The site-specific pavement layers properties and traffic loading information were then incorporated in the TxME pavement design software to obtain pavement responses. Climatic data from an adjacent weather station, as well as performance criteria limits, were also assigned using the agency defined thresholds. Subsequently, the critical pavement responses for multiple axle weights and axle/tire configurations were calculated in this study. Based on the internal algorithms and incorporated performance models for the quantification of pavement damages, the incremental increase in pavement damages were calculated. Ultimately, the service life of pavements were determined based on the number of months until the cumulative pavement distresses meet the agency defined threshold. The pavement performance criteria considered in this study include cumulative rutting at the pavement surface, AC layer fatigue cracking, stabilized base layer fatigue cracking, and thermal cracking. The distress thresholds set forth by TxME are shown in Table 10.1.

Table 10.1: Agency-Defined Distress Limits

<table>
<thead>
<tr>
<th>Performance Criteria</th>
<th>Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Surface Rutting (inch)</td>
<td>0.5</td>
</tr>
<tr>
<td>Thermal Cracking (ft/mile)</td>
<td>2112</td>
</tr>
<tr>
<td>AC Fatigue Cracking Area (%)</td>
<td>50</td>
</tr>
<tr>
<td>Stabilized Base Fatigue Cracking (%)</td>
<td>50</td>
</tr>
</tbody>
</table>

10.5.2. Approach to Assess Pavement Life Reduction

Determination of an accurate account of traffic characteristics is an integral component of analyses of Pavement Life Reduction (PLR) due to recent changes in truck traffic operations. Therefore, the authors incorporated the projected ESAL values over a 20-year design life for both current and pre-energy development traffic loading conditions, i.e., $ESAL_{\text{Current}}$ and $ESAL_{\text{Pre-Energy Development}}$. 
respectively, to characterize the traffic in numerical simulations for further comparison purposes. To accomplish this, initially, the cumulative 18-kip ESAL values that correspond to the current traffic condition were calculated from Equation 1 as:

\[
ESAL_{current} = \sum_{i=1}^{m}(EALF)_i n_i 
\]

where:

\[(EALF)_i = Modified\ Mechanistic-Based\ EALF\ Values,\]
\[n_i = Projected\ number\ of\ passes\ of\ i^{th}-axle\ load\ group\ during\ the\ design\ period.\]

Modified EALF values were determined using the mechanistic procedure for pavement damage quantification described in Chapter ten. The number of axle load repetitions were also derived from the site-specific ALS databases. Then, the EALF values for each axle type were multiplied by the projected number of load repetitions to calculate the projected ESAL values considering the design life of the pavements.

Subsequently, the research team developed an algorithm to retrace and backtrack the traffic to the onset of change in the economic activities such as energy production for each roadway. Based on the analysis of traffic growth in the region, the majority of representative pavement sections experienced drastic changes in truck operations between 2008 and 2012. Subsequently, the authors conducted an extensive search within the available databases, such as PMIS, TCDS, and LTPP, to extract traffic information attributed to the pre-energy development traffic conditions. Subsequently, considering the traffic growth rate during the entire period, both ESALs, i.e., ESAL\text{Current} and ESAL\text{Pre-Energy Development}, were converted to the equivalent 18-kip axles corresponding to the reconstruction/rehabilitation year, which serves as the benchmark in this study, as shown in flowchart in Figure 10.4.
Figure 10.4: Flowchart Describing the Procedure for Backtracking the Current Traffic to Pre-Energy Development Traffic.

Ultimately, the research team incorporated the ESAL values into the mechanistic-empirical pavement design software for comparative analysis of service of life of pavements for the aforementioned two sets of traffic data. The difference between the two service life analyses represents the reduction of the service life associated with the changes in traffic patterns for each pavement section in this study. Ultimately, the loss of pavement life based on the distress plots and agency thresholds were determined, as schematically shown in Figure 10.5. Additionally, “pavement life reduction” index was calculated to provide a quantitative measure of the severity of the distresses imparted by energy development activities in the network as:

\[
Pavement\ \text{Life Reduction (PLR}_{\text{Energy-Development})\ \text{Index}} = \frac{L_{PE} - L_C}{L_{PE}} \times 100 \tag{11-2}
\]

where,

\(L_{PE}\): expected pavement life considering the pre-energy development traffic characteristics,

\(L_C\): expected pavement life considering the current traffic characteristics.
10.5.3. Relevant Traffic Data Collected by Portable WIM Devices

The site-specific traffic features derived from the P-WIM units were:

- Axle load spectra, axle load distributions,
- Truck class distributions,
- General traffic information such as AADT, two-way AADTT, and vehicle operational speed, etc.
- Axle configurations.

10.5.3.1. Axle Load Spectra (ALS)

ALS provides hourly, daily, weekly, monthly, seasonal, and annual distribution of the classes of vehicles for mechanistic analysis and design of pavement structures. In this subtask of the study, the traffic data collected by P-WIM units were synthesized to develop site-specific ALS databases. Subsequently, the ALS were analyzed and classified by different axle types with specific load intervals to develop axle load distributions within each vehicle class and seasons of the year.

Figure 10.6 illustrates the axle load distributions for different axle types in one of the heavily trafficked sections, Farm-to-Market 468 in Laredo District, based on the traffic data
collected by P-WIM devices during the summer of 2018. The analysis of the load distributions in
FM 468 indicated high number of OW truck operations in this roadway. The plots show that nearly
16% and 34% of the single and tandem axles, respectively, were attributed to the passages of OW
axles in FM 468 during the studied timeframe. Additionally, OW tridem and quad axles with
alarming percentages as high as 33% and 39%, respectively, were recorded in this site. Such high
percentages of OW truck traffic in the network can potentially jeopardize the longevity of
transportation facilities and result in premature failure of pavement structures.
Figure 10.6: Axle load distributions for all trucks in Farm-to-Market 468 for: (a) Single axle, (b) Tandem axle, (c) Tridem axle, and (d) Quad axle.
10.5.3.2. **Truck Class Distribution (TCD)**

TCDs are essential component of traffic information that significantly influence the analysis of the RSL of pavement structures. Based on the traffic data from the P-WIM systems in this study, the most prevalent vehicles were Class 5 and Class 9 trucks. Figure 10.7 provides relevant information on TCDs associated with different roadway types, i.e., FM, SH, and US. As evidenced in the plot, in all different types of roadway facilities, Class 9 trucks with range of 50% to 75% of total truck traffic were the most predominant truck class, followed by Class 5 trucks, ranging from 8% to 30% of the total truck traffic in the network. Another noteworthy observation from Figure 10.7 was the fact that FM roadways generally tend to accommodate higher Class 5 trucks compared to the SH and US highways. In contrast, US highways had the highest Class 9 and the fewest Class 5 trucks.

![Figure 10.7: Truck class distributions for different roadway types in Texas overload corridors.](image)

10.5.3.3. **General Traffic Information**

The information on the two-way AADTT, ADT, and vehicle operational speed attributed to each studied site was obtained from post-processing of the P-WIM data. Additionally, directional and lane-distribution factors were assumed as 50%, and 95%, respectively, to complement the general
traffic information on Level 1 traffic inputs. This information was a direct input to the service life analysis protocols.

Table 7.1 provides a summary of the traffic information attributed to the current and pre-energy development era for the representative roadways studied in the network. This information, as well as the roadway reconstruction year, are the key elements for further service life analyses performed in this study.

Table 0.1. Traffic Information attributed to the Current and Pre-Energy Development Conditions for Ten Representative Sites in the Network

<table>
<thead>
<tr>
<th>District</th>
<th>Roadway</th>
<th>Traffic (2019)</th>
<th>Pre-Energy Development Traffic (e.g. 2008)</th>
<th>Reconstruction/Rehabilitation (Year)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>20-Year ESAL (Million)</td>
<td>AADTT</td>
<td>ADT</td>
</tr>
<tr>
<td>Laredo</td>
<td>US 83</td>
<td>8.7</td>
<td>1636</td>
<td>6300</td>
</tr>
<tr>
<td></td>
<td>FM 468</td>
<td>17.5</td>
<td>839</td>
<td>2150</td>
</tr>
<tr>
<td>San Antonio</td>
<td>FM 624</td>
<td>3.8</td>
<td>289</td>
<td>875</td>
</tr>
<tr>
<td></td>
<td>FM 99</td>
<td>4.8</td>
<td>267</td>
<td>702</td>
</tr>
<tr>
<td></td>
<td>SH 16</td>
<td>8</td>
<td>1116</td>
<td>5072</td>
</tr>
<tr>
<td>Corpus Christi</td>
<td>US 281</td>
<td>20</td>
<td>1815</td>
<td>7000</td>
</tr>
<tr>
<td></td>
<td>SH 72</td>
<td>14</td>
<td>1763</td>
<td>5700</td>
</tr>
<tr>
<td></td>
<td>SH 123</td>
<td>28.5</td>
<td>1976</td>
<td>10400</td>
</tr>
<tr>
<td>Yoakum</td>
<td>US 183</td>
<td>6</td>
<td>1432</td>
<td>8420</td>
</tr>
<tr>
<td></td>
<td>SH 119</td>
<td>2</td>
<td>659</td>
<td>2530</td>
</tr>
</tbody>
</table>
10.6. **ANALYSIS OF RESULTS AND DISCUSSION**

10.6.1. **Remaining Service Life**

The results pertaining to one of the heavily trafficked sections, US281 in Corpus Christi, is provided as an example to showcase the RSL protocol developed in this study. Figure 1.8 shows the progression of distresses over a 20-year design period for US281. The evaluated section was reconstructed in 2014, therefore, the onset of the analysis (base year) was set for the date of reconstruction of this roadway. As shown in Figure 10.8, the service life of US 281 based on rutting performance is 133 months after reconstruction. The simulations and field rutting measurements were performed in the spring of 2019, therefore the analysis period was (2019-2014=5 years) or 60 months. Based on the distress calculations and agency thresholds, the remaining life for the US281 in the spring of 2019 was calculated as 73 months (i.e., 133-60=73).
Figure 10.8: Predicted service life at US 281 highway, based on different pavement distresses: (a) Rutting, and (b) Fatigue cracking.

Similar analyses were conducted for all representative pavement sections in this study. Figure 10.9 shows the post-processed remaining service life analyses for the roadways in this study. The results indicated that the three evaluated FM roadways, i.e., FM468, FM624, and FM99, have reached or surpassed their intended service lives due to applications of unaccounted for OW traffic in the network. The distresses were primarily associated with cumulative surface rutting due to OW truck traffic in structurally deficient FM roadways. It was also found that, among state highways, SH16 in San Antonio District, originally constructed in more than 30 years ago with several maintenance measures, has already surpassed the agency-defined distress limits. This could be the plausible reason behind recent reconstruction efforts in 2019 and 2020 to improve the ride quality of SH16. Based on the analysis of remaining service life, State Highway 123 should be able to accommodate another three years of current traffic mix with a modest historical growth rate before it reaches the distress limits set forth by the design agency.
The analysis of the service life in this study indicated that the rutting was the primary distress responsible for premature failure of the pavement sections. It worth noting that only two sites (SH16, and SH119) with average asphalt thickness of approximately 3.5 in. (8.9 cm), experienced relatively high fatigue cracking exceeding the failure criterion, among the studied sites.

10.6.2. Comparisons of Predicted and Field-Measured Rut Depth

The predicted rutting values from the proposed mechanistic approach were contrasted with those obtained from field distress surveys of the representative pavement sections to assess the accuracy of the analysis results. Rutting performances of US 281 and FM468 roadways are discussed in this section as case studies. As shown in Figure 10.10, our methodology predicted rut depth of 0.380 in. (9.65 mm) on US 281 by spring 2019 after five years of service. This is in line with the rut depth measurements conducted along US281, as shown in Figure 10.10 a. The pictures provided in Figure 10.10 a show rut depth measurement under the straight edge in the wheel path to be approximately 0.375 in. (9.53 mm) on average, in the spring of 2019. Additionally, based on the

Figure 10.9: Remaining service life of the representative pavement sections in Texas overload corridors.
mechanistic distress calculations in this study, it takes merely five months for FM468 after the spring of 2019 to surpass the 0.5 in. (12.7 mm) rutting threshold. Our field observations and distress monitoring of FM468 during the spring of 2019 also indicated that several segments of this roadway exhibited over 0.5 in. (12.7 mm) of surface rutting, as shown in Figure 10.10 b.

![0.37 inches Rutting](image1)

![0.38 inches Rutting](image2)

![0.50 inches Rutting](image3)

![0.52 inches Rutting](image4)

(a)

(b)

Figure 10.10: Rutting measurements in (a) US 281, Corpus Christi district, and (b) FM 468, Laredo district.

Figure 10.11 illustrates the measured versus predicted rut depths for all sites in this study. The Root Mean Square of Error (RMSE) between the predicted and field-measured rut depths were calculated for cross-validation of the accuracy of the proposed algorithm for RSL analysis. The results show that incorporating the site-specific ALS into the developed algorithm for the studied sections yielded a rut depth RMSE of 65 mils (1.65 mm). Such low measures for error in pavement distress predictions, coupled with a relatively high \( R \)-squared value of 0.90 of the fitted line, provides confidence in the soundness of the algorithm developed for the assessment of the service life of pavements subjected to OW trucks in this study.
Additionally, a student $t$-test was performed to examine whether the differences between predicted and measured rut values were statistically significant. The null hypothesis was defined if the differences in predicted and measured rut values were equal to zero. A significance level ($\alpha$) of 0.05 for 95% confidence was assumed, implying that a $p$-value of 0.05 or less rejects the null hypothesis. Based on the student’s one-sided $t$-test, the $p$-value was found to be 0.66 (>0.05). Therefore, the null hypothesis was accepted, indicating that there is no significant arithmetic difference between the predicted and measured rut depth values.

Comparisons between the predicted and field-measured rutting indicated that the proposed methodology for prediction of the RSL is capable of assessing the incremental progression of the distresses imparted during the service life of the pavements.

![Figure 10.11: Predicted vs. field-measured surface rutting.](image)

10.6.3. **Influence of Traffic Characterization Methodology on Pavement Life Predictions**

The authors conducted a series of numerical analysis in order to highlight the influence of the method of traffic characterization on the analysis of the service life of pavements. Similar
processes but with different traffic input scenarios, including the site-specific ALS (Level 1),
traditional ESAL values (Level 2), and mechanistic software average values (Level 3), were
performed for comparison purposes.

Figure 10.12 shows the comparative results for US281, SH123, and FM468 roadways
based on different traffic characterization methodologies. The results are reported as “number of
months” remaining to surpass the pre-defined distress limits. As evidenced in this plot, simulations
with site-specific ALS resulted in significantly lower predictions of service life when compared to
traditional ESAL concept in Level 2 traffic input. This is in line with our expectations, as the
damage quantification based on the traditional ESAL approach overlooks the load groups and
time-dependency of the load applications (Morovatdar et al., 2021d). For this reason, incorporation
of traditional ESALs into the RSL analysis can underestimate the imparted pavement damages by
heavy truck traffic operations, leading to overestimation of the pavement remaining life in the
overload corridors.

Figure 10.12: Comparative results for RSL analysis based on different traffic inputs.
Another method to assign traffic characteristics is by using the mechanistic software average values, as commonly used for pavement management purposes in practice. The post-processed results illustrated in Figure 10.12 show that using the statewide average traffic distributions as Level 3 traffic inputs resulted in substantial deviations from the ESAL concept and site-specific ALS databases. This underscores the significance of using site-specific traffic data in lieu of software default values for decision-making purposes.

10.6.4. Reduction of the Pavement Service Life due to Changes in Traffic Patterns

Figure 10.13 shows the post-processed results pertaining to the PLR analysis for US281 highway. The plot shows the rutting performance for US281, considering the pre-energy development and current traffic information. Based on the internal distress calculation models in mechanistic analysis, and pre-energy development traffic characteristics with modest historical growth, it takes 254 months for US281 to develop 0.5 in. (12.7 mm) of rut depth after major rehabilitation in 2014. However, if the 2019 traffic characteristics, determined by the P-WIM devices in the field, were incorporated in the mechanistic analysis, it takes 158 months to develop 0.5 in. (12.7 mm) of cumulative rut depth. In other words, changes in the traffic patterns attributed to the socio-economic activities have consumed 96 months (i.e., 254-158=96) or 8 years of the service life of US281 highway.
Figure 10.13: Reduction of pavement service life for US281 highway due to changes in traffic patterns.

Similar analyses were conducted for all ten representative roadways in the network. Figure 10.14 illustrates the post-processed results associated with the service life reduction of the studied overload corridors. As evidenced in the plot, FM roadways with less robust pavement profile were found to be more sensitive to the increasing traffic frequencies, compared to SH and US highways (Morovatdar et al., 2021e). Specifically, FM99, followed by FM468, and ultimately FM624 roadways have been subjected to the largest PLR as 66%, 51%, and 48%, respectively. The plot also showed that the pavement service life of SH123 and US281 highways were greatly impacted by the changes in traffic patterns in the past decade. The results highlighted the influence of load magnitudes and traffic frequencies on pavement life consumption for the heavily trafficked corridors with demanding loading conditions in the surveyed network.
In this chapter, a mechanistic framework was developed for characterization of the service life of pavements subjected to taxing stress paths imparted by heavy truck traffic operations. The proposed methodology accounts for the site-specific traffic information, unique features of the transportation systems, and the environmental factors for accurate assessment of the RSL of pavements. The traffic parameters, pavement structural properties, and climate information for ten representative sections were incorporated into a series of numerical simulations in TxME for the determination of the RSL of the pavements subjected to taxing traffic loading conditions. The authors also proposed a methodology to assess the reduction of the service life of pavements due to changes in traffic patterns, with considerations of demanding loading scenarios in the field, type of transportation facilities, and environmental impacts. The major findings of this research effort summarized as:

- Accurate account of traffic characteristics is the precursor for reliable assessment of the cumulative damages imparted on pavements in overload corridors. Site-specific ALS data...
provides the primary mechanistic traffic data input for accurate and optimal pavement design and analysis.

- A comparative analysis of different traffic characterization methodologies, i.e., ALS, and traditional ESAL approach, underscored the significance of using the ALS database in lieu of traditional conversion of traffic mix to standard axle for the prediction of the service life of pavements.

- Incorporation of traditional ESALs instead of ALS, in RSL analysis, vastly overestimated the remaining service life of the pavements. This is primarily attributed to the fact that the damage quantification based on the traditional ESAL approach overlooks the load groups and time-dependency of the traffic distributions, leading to an underestimation of the imparted pavement damages by heavy truck operations.

- Using the average statewide traffic distributions to characterize the traffic mix resulted in drastic overestimation of the predicted service life compared to traditional ESAL concept and site-specific ALS databases. This highlights the importance of using site-specific traffic data rather than using the software/agency default values for decision-making purposes.

- The results underscored the significance of the type of transportation facilities in the damage quantification protocol as the FM roads, with less robust pavement structural capacity, experienced substantially higher “reduction of service life” due to change in traffic patterns compared to SH and US highways. This is primarily attributed to the fact that FM roadways with extremely thin asphalt treated surface layers were not designed to withstand such demanding loading conditions with large volumes of heavy truck operations.
• Comparisons between the numerical simulations and field distress records in this study showed reasonable agreement between the RSL analysis results using the proposed framework and field distress measurements.

• The primary source of distresses in the evaluated pavement sections in this study was associated with cumulative surface rutting due to heavy truck traffic. This in line with our expectations, as the primary culprit for premature failure of the overload corridors pertains to the passages of OW and SHL vehicles in the network.
Chapter 11: Characterization of the Structural Impacts of SHL Vehicles

11.1. INTRODUCTION

There are several concerns associated with the structural impacts of the SHL vehicles operating in the transportation networks across the nation. Movement of the SHL vehicles can adversely impact the longevity of the transportation infrastructure, leading to major loss of the service life of the pavement facilities. In addition to the pavement life reduction, stability of the sloped pavement shoulders under the movement of the wide SHL units is another major concern that needs to be properly assessed. Moreover, these heavy vehicles pose a potential failure risk to the existing buried utilities along the SHL vehicles’ route. Evidently, these are ongoing nationwide challenges for design agencies with limited precedence in the literature. Consequently, the analysis of the SHL structural impacts should properly account for these challenges.

The highlighted concerns are even more pronounced considering the slow-moving nature of the SHLs, acceleration/deceleration forces, turning movements at the bends, flooding conditions, and elevated temperatures in summer seasons. Accordingly, the analysis protocols presented in this chapter aim to address the following key issues associated with the SHL operations in OW corridors:

- Non-generic and non-conventional arrangement of the SHL axles,
- Detrimental impacts of the heaviest wheel load of SHL vehicles,
- Slow-moving nature of SHLs, considering the viscoelastic properties of asphalt layer,
- Acceleration/deceleration forces induced by SHL moving tires,
- Role of highway geometric features,
- Demanding environmental scenarios,
- Influence of SHLs on pavement life reduction,
Impact of wide SHL vehicles on the stability of the sloped pavement shoulders, and
Potential risk against the failure of the buried utilities.

The relevant databases on SHLs, pavement layer configurations, and back-calculated layer moduli, coupled with the devised analysis algorithms and the FE code developed for advanced modeling of the SHLs, as elaborated in the preceding chapters, make adequate provisions for analysis of the structural impacts on the pavements subjected to the SHL operations. This enables the authors to accurately assess the detrimental effects of the most demanding loading conditions on the structure, longevity, and stability of the pavement facilities in different US Highways, State Highways, and FM roadways.

11.2. Objective

The primary objective of this chapter is to provide the rationale behind the various analysis protocols developed to mechanistically characterize the structural impacts of SHL vehicles on pavement facilities. The developed mechanistic frameworks take into consideration the most demanding loading conditions, various vehicle loading scenarios, site-specific traffic makeup, unique features of the transportation facilities, and environmental impacts for realistic assessment of the SHL structural impacts.

11.3. Analysis Plan for Evaluation of the SHL Structural Impacts

Table 12.1 provides a summary of the major analysis categories and items considered for determination of the structural impacts of SHL vehicles in this study. The all-compelling algorithm developed in this subtask of the study consisted of various analysis categories, including the loss of pavement service life, analysis of the slow-moving nature of the SHL vehicles, analysis of the acceleration/deceleration forces, analysis of the roadway geometric features, stability
analysis of sloped pavement shoulders, and buried utility risk analysis. The primary outcome parameter, namely pavement life reduction \((PLR)\), probability of failure \((P_f)\), and Factor of Safety \((FoS)\), derived from each analysis category is also presented in Table 11.1.

Table 11.1: Major SHL Analysis Categories and Parameters Incorporated for Characterization of the SHL Structural Impacts

<table>
<thead>
<tr>
<th>SHL Analysis Category</th>
<th>Major Parameters Incorporated</th>
<th>Analysis Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis of the Loss of Pavement Service Life</td>
<td>wheel load; roadway type, seasonal variations of material properties; moisture management capability; precipitation rate; ground water table; traffic demand variations; traffic growth rate; and tire pressure</td>
<td>Pavement Life Reduction ((PLR))</td>
</tr>
<tr>
<td>Analysis of the Slow-Moving Nature of SHLs</td>
<td>wheel load; roadway type, seasonal variations of material properties; slow-moving nature of SHLs; and vehicle operational speed</td>
<td>Pavement Life Reduction ((PLR))</td>
</tr>
<tr>
<td>Analysis of the Acceleration/Deceleration Forces</td>
<td>wheel load; roadway type, seasonal variations of material properties; vehicle loading scenarios; and acceleration/deceleration patterns</td>
<td>Pavement Life Reduction ((PLR))</td>
</tr>
<tr>
<td>Analysis of the Roadway Geometric Features</td>
<td>wheel load; roadway type, seasonal variations of material properties; and roadway geometric features (super-elevation, and curve radius)</td>
<td>Pavement Life Reduction ((PLR))</td>
</tr>
<tr>
<td>Stability Analysis of the Sloped Pavement Shoulders</td>
<td>wheel load; shoulder slope; shoulder width; shoulder type; and moisture management capability;</td>
<td>Probability of Failure ((P_f))</td>
</tr>
<tr>
<td>Buried Utility Risk Analysis</td>
<td>wheel load; depth of cover for buried utilities; roadway type; and pipe characteristics</td>
<td>Factor of Safety ((FoS))</td>
</tr>
</tbody>
</table>

Furthermore, a series of parametric analyses are conducted that account for the various analysis parameters associated with the traffic loading conditions, climatic factors, pavement structures, roadway shoulders, and utilities buried underneath the pavement. Table 11.2 represents the numerical simulation matrix developed in this study to properly investigate the influence of major analysis parameters on the structural impacts of the SHL vehicles. It is worth noting that the range of values for different parameters is selected and incorporated into the parametric analysis, based on the review of the pertinent information extracted from the available databases, design
plans and specifications, as well as our research team experience from field data collection efforts in the overload corridors.

Ultimately, the results are post-processed and the summary of the major points are presented in this chapter to provide further insights on the major influential factors and their role on the longevity and structural integrity of the transportation facilities servicing the SHL vehicles.
Table 11.2: Devised Numerical Simulation Matrix for Determination of the Structural Impacts of SHL Vehicles

<table>
<thead>
<tr>
<th>SHL Analysis Category</th>
<th>Wheel Load (kips)</th>
<th>Roadway Type</th>
<th>Season</th>
<th>Moisture Management</th>
<th>Traffic Demand Variations (%)</th>
<th>Traffic Growth Rate (%)</th>
<th>Tire Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis of the Loss of Pavement Service Life</td>
<td>6, 7, 8, 9, 10, 11, 12</td>
<td>FM, SH, US</td>
<td>Summer, Winter</td>
<td>Excellent, Poor</td>
<td>10, 20, 30, 40, 50</td>
<td>2, 3, 4, 5, 6</td>
<td>100, 110, 120, 130, 140</td>
</tr>
<tr>
<td>Analysis of the Slow-Moving Nature of SHLs</td>
<td>6, 7, 8, 9, 10, 11, 12</td>
<td>FM, SH, US</td>
<td>Summer, Winter</td>
<td>10, 20, 30, 60</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Analysis of the Acceleration/Deceleration Forces</td>
<td>6, 7, 8, 9, 10, 11, 12</td>
<td>FM, SH, US</td>
<td>Summer, Winter, Steady Rolling, Acceleration, Deceleration</td>
<td>25, 50, 100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Analysis of the Roadway Geometric Features</td>
<td>6, 7, 8, 9, 10, 11, 12</td>
<td>FM, SH, US</td>
<td>Summer, Winter</td>
<td></td>
<td>800, 1200, 1600, 2000, 2400, 2800, 3200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stability Analysis of the Sloped Pavement Shoulders</td>
<td>6, 7, 8, 9, 10, 11, 12</td>
<td>FM, SH, US</td>
<td>1.0, 1.5, 2.0, 3.0, 4.0, 5.0, 6.0</td>
<td></td>
<td>1, 2, 3, 4, 5, 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Buried Utility Risk Analysis</td>
<td>6, 7, 8, 9, 10, 11, 12</td>
<td>FM, SH, US</td>
<td>0, 3, 6, 9, 12, 15, 18</td>
<td></td>
<td>10, 15, 20, 25, 30, 35</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Paved Unpaved Excellent Poor

Pipe Type Flexible Rigid
11.4. **ANALYSIS OF THE LOSS OF PAVEMENT SERVICE LIFE**

11.4.1. **Introduction**

Despite facilitating the movement of heavy, large, and non-divisible loads, SHL vehicles typically consist of multi-axle trailer units that weigh several folds of the permissible weight limits set forth by regulatory agencies. Operation of these non-conventional vehicles with heavy tires and complex axle arrangements has been a major contributor to the damages for pavement structures in the overload network. Analysis of the traffic distribution patterns in Texas overload corridors underscored the significance of the heavy truck operations and its role to jeopardize the longevity of transportation infrastructures. Accurate quantification of the structural impacts of OW trucks is the prelude to adopting proper rehabilitation strategy by stakeholders. This can effectively protect the taxpayers’ resources allocated to restore the current transportation facilities and expand the network to meet future traffic demands. Consequently, there is a pressing need to accurately quantify the pavement loss imparted by SHL vehicle operations in the affected networks.

Several researchers utilized different methodologies to evaluate the service life of pavements subjected to SHL vehicle operations. Chen et al. (2013), using numerical simulations, investigated the consumption of pavement service life due to the movement of a SHL vehicle in Louisiana. The researchers converted the SHL axles to equivalent number of standard axles to characterize the loading conditions in their analysis. The authors in turn used this information to estimate the costs associated with the pavement damages. Dong and Huang (2013) evaluated the detrimental impacts of SHLs on pavements in Tennessee through field measurements of the pavement responses. Based on field-derived data, the authors concluded that the evaluated SHLs did not cause considerable pavement deformation. Hajj et al. (2018) developed a mechanistic framework to investigate the impacts of the SHLs on pavements on a case-by-case basis. The
researchers used records of issued permits as the primary source to characterize the traffic distributions, and loading conditions attributed to SHLs. In the numerical simulation phase, the researchers considered uniform distribution of contact stresses over a circular loaded area.

The majority of the previous studies for predicting the performance and service life of the pavements either rely on the measurements made in the field, or are based on limited data points. The simplifying assumptions such as the use of permit records in lieu of field data collection, limitation of the type of pavement facilities in the study, and overlooking the influence of seasonal variation of material properties can potentially jeopardize the accuracy and reliability of the damage quantification and remaining life analyses of pavement facilities. Another anomaly persistent in the literature pertains to unrealistic simulation of the tire-pavement contact stresses using uniformly distributed load, rather than considering the non-uniform distribution of the contact stresses. Relying on such simplifying assumptions can be detrimental to the accuracy of the analysis of the structural impacts of SHL vehicles with demanding loading conditions.

This was the motivation for the research team to develop a protocol to quantify the influence of SHL vehicles on the consumption of the service life of the representative pavements in Texas. The developed analysis procedure, which is based on the site-specific ALS, accounts for the slow-moving nature of the SHL vehicles, acceleration/deceleration effects, roadway geometric characteristics, environmental conditions, realistic tire-pavement interactions, and unique features of different roadway types in the surveyed network.

Mechanistic analyses are performed and the post-processed results associated with pavement life reduction in FM, SH, and US highway network affected by the SHL applications are provided. Moreover, the authors conducted a parametric analysis that takes into consideration several influential factors attributed to the pavement structural properties, specific characteristics
of the SHL vehicles, traffic loading scenarios, and environmental factors. The results can potentially provide insights into the PLR studies, and provide means to improve pavement design protocols.

The following section provides relevant information on the specifics of the pavement life reduction methodology adopted in this study, which is followed by the analyzed results and discussions.


Figure 11.1 shows a flowchart of the proposed procedure for the mechanistic characterization of the loss of pavement service life due to SHL vehicle operations. As illustrated in the figure, the proposed analysis framework consisted of three main segments, including: (1) field testing, (2) damage quantification using 3D FE modeling, and (3) prediction of the service life of pavements. As shown in Figure 11.1, the field-derived databases on pavement materials properties and traffic loading conditions, presented in chapter 4 and 5, were direct inputs into the analysis of the loss of pavement service life in this section. The permit records and SHL vehicle plans were also instrumental to complement the information on the axle and tire characteristics of the SHLs.

The site-specific pavement layers properties and the traffic loading information were in turn incorporated into the developed FE code for determination of the critical pavement responses under the SHL vehicle movements. The calculated pavement responses were further used to determine the corresponding EALFs tailored towards the specific SHL axle, roadway type, and season of the year, as described in Chapter Ten.
The last stage of the analysis pertains to characterization of the influence of SHL movements on the pavement service life. To assess the pavement service life reduction in this subtask, initially, the SHL axles were identified and removed from the ALS database. Then, the projected ESAL values over a 20-year design life for two case scenarios of traffic inputs, i.e., ESAL\textsubscript{(with SHLs)} and ESAL\textsubscript{(without SHLs)}, were incorporated into the pavement service life prediction analysis routine to characterize the traffic in numerical simulations for further comparison purposes. The cumulative 18-kip ESAL values were calculated from the following relationship, i.e., $ESAL = \sum_{i=1}^{m} (EALF)_i n_i$, where, $(EALF)_i$ represents the calculated EALF value for $i^{th}$-axle; and $n_i$ is the projected number of passes of $i^{th}$-axle load group during the design period, derived from the site-specific ALS. Evidently, pavement damage equivalency factors, presented in chapter ten, are integral components for accurate characterization of the loss of pavement service life.
Figure 11.1: Flowchart for the proposed mechanistic approach for the quantification of the Pavement Life Reduction (PLR) due to SHL vehicle movements.

Ultimately, the ESAL values were incorporated into the ME pavement design software for comparative analysis of service of life of pavements for the aforementioned two sets of traffic data. The difference between these two results represents the reduction of the service life associated with the SHL vehicle operations for each pavement section. The loss of pavement life based on the distress plots and preset distress limits was then determined, as schematically shown in Figure 11.2. Additionally, “pavement life reduction” index was calculated to provide a quantitative measure of the severity of the distresses imparted by SHL operations in the network as:

\[
\text{Pavement Life Reduction (PLR}_{\text{SHL}}) = \frac{PL_{\text{without SHL}} - PL_{\text{with SHL}}}{PL_{\text{without SHL}}} \times 100
\]
\[
\text{PLR}_{\text{SHL}} = \frac{\text{PL}_{\text{without SHLs}} - \text{PL}_{\text{with SHLs}}}{\text{PL}_{\text{without SHLs}}} \times 100
\]  

where,

- \( \text{PLR}_{\text{SHL}} \): imparted pavement life reduction due to the SHL vehicle operations.
- \( \text{PL}_{\text{with SHL}} \): expected pavement life considering the entire ALS database, including SHLs.
- \( \text{PL}_{\text{without SHL}} \): expected pavement life excluding SHLs from the ALS database.

**Figure 11.2:** Schematic Diagram of the Loss of Pavement Service Life Analysis.

### 11.4.3. Analysis of Results and Discussions

The results pertaining to one of the heavily trafficked sections, SH 123 in Corpus Christi, is provided as an example to showcase the pavement service life reduction protocol developed in this study. Figure 11.3 illustrates the axle load distributions for different axle types based on the traffic information collected by P-WIM devices in the winter of 2019. The analysis of the load distributions in SH 123 indicated high number of SHL operations in this roadway. The plots show that nearly 3% and 17% of the single and tandem axles, respectively, were attributed to the passages of SHL axles in SH 123 during the studied timeframe. Additionally, based on the traffic
data collected using P-WIM devices, SHL tridem and quad axles with alarming percentages as high as 26% and 39%, respectively, were recorded.

Figure 11.3: Axle Load Distributions and the SHL Axles Captured in State Highway 123 for: (a) Single Axle, (b) Tandem Axle, (c) Tridem Axle, and (d) Quad Axle.

Figure 11.4 shows the analysis of the results associated with the mechanistic quantification of the loss of pavement service life in SH 123 in Corpus Christi District. The plot shows the progression of the distresses, i.e., rutting and fatigue cracking, over a 25-year design period for SH 123, based on different traffic input scenarios. Based on the internal distress calculation models in TxME, and the field-collected traffic data, it takes 168 months for SH 123 to develop 0.5 in. of rut depth after major rehabilitation in 2012. However, if the SHLs were removed from the axle load spectra in the traffic inputs, it takes 253 months to develop 0.5 in. of rut depth. In other terms, the
operation of the SHL vehicles in SH 123 has consumed \((253-168 = 85)\) months of the service life of this pavement section. Additionally, the OW trucks operation in SH 123 resulted in 105 months reduction in its pavement service life. Consequently, the analysis of the loss of pavement service life based on the rutting criteria indicated that SH 123 was subjected to a drastic PLR as high as 33%, and 38% due to applications of SHLs, and OW trucks, respectively. It is worth noting that movement of SHL and OW trucks in SH123 can potentially consume 7% and 10%, respectively, of its pavement service life, based on fatigue criteria.
Figure 11.4: Loss of Pavement Service Life for State Highway 123 due to SHL and OW Truck Operations, based on: (a) Rutting, and (b) Fatigue Cracking.

Similar analyses were conducted for all representative pavement sections. Figure 11.5 shows the post-processed PLR analyses for the roadways evaluated in this study. Based on the results provided in the plot, the pavement service life of the representative FM roadways were more substantially impacted by the heavy vehicle operations, compared to the studied pavement sections in State and US Highways. This is mainly attributed to the structural deficiency of FM roadways, coupled with the alarming movements of super heavy vehicles in these rural roadways that were never designed to withstand such demanding loading conditions.

According to the post-processed results provided in Figure 11.5, SHL and OW truck operations in FM roadways can impart substantial PLR as high as 55%, and 62%, respectively; while the greatest loss of pavement service life associated with the other types of roadway facilities ranged between 33% and 38% for State Highways, and between 25% and 32% for US highways, due to heavy truck traffic operations. The results underscore the significance of type of roadway
facility and its role on proper characterization of the loss of pavement service life in overload corridors.

Another noteworthy observation from Figure 11.5 was the fact that SHL vehicles can more significantly contribute to the loss of pavement service life, compared to the OW truck movements. In other terms, one passage of the SHL vehicles can potentially induce higher level of damages and therefore higher pavement life consumption, compared to one movement of OW trucks. For instance, operations of the SHL vehicles in SH 123 with 20% of total truck traffic have resulted in 33% PLR; while passages of OW trucks in this site with 35% frequency have consumed 38% of its pavement service life. Consequently, substantial portion of the imparted loss of pavement service life is attributed to the SHL vehicle operations in the studied network.

This observation was expected, since SHL vehicles carry extremely heavy axle and wheel loads that are primarily characterized in the tail end of the load spectrum. Therefore, operation of such SHL vehicles, despite having lower frequencies relative to the OW trucks, make an enormous

![Figure 11.5: Loss of Pavement Service Life for all 10 Representative Sites due to SHL and OW Truck Operations.](image-url)
contribution to the damages accumulated over the service life of pavement facilities and can potentially jeopardize the longevity of transportation infrastructures.

The analysis of the loss of pavement service life in this subtask also indicated that the rutting-based pavement life consumption was greatly higher compared to the PLR level based on the fatigue performance (Morovatdar et al., 2021c). This observation was anticipated as the distresses associated with the heavy truck operations are mainly attributed to the load magnitude rather than the load repetitions. Such behavior was also manifested by the numerical simulations, as the mechanistic prediction of pavement service life for the studied sites with large volume of heavy truck movements revealed that it takes longer for fatigue performance to reach the failure criterion, i.e., 50% AC fatigue cracking area. According to the sensitivity analysis performed in this study, the predicted fatigue cracking for the majority of the roadways evaluated in this study rarely reached or surpassed the distress limits over the 20-year design period. Consequently, rutting was the primary distress responsible for the loss of pavement service life imparted by SHL and OW vehicle movements.

11.4.3.2. Relationship between SHL Vehicle Operations and PLR
The post-processed traffic data indicated a significant number of SHL vehicles operating in the surveyed network. For instance, based on the traffic data collected during 2018 and 2019, alarming frequencies of SHL vehicles as high as 25% and 20% were recorded in FM468 roadway in Laredo District, and SH123 in Corpus Christi District, respectively. Such SHL vehicle passages can be the primary culprit for the premature failure of pavement structures in overload corridors. For this reason, the authors investigated the potential relationship between the SHL vehicle movements and the imparted loss of pavement service life.
Figure 11.6 shows the PLR, as predicted using the developed framework, versus SHL vehicle percentages, as recorded by P-WIM units in the field. An $F$-test was performed to statistically examine whether a linear relationship exists between the two aforementioned parameters. The significance level ($\alpha$) of 0.05 for 95% confidence was assumed. The null hypothesis was then defined if the slope of the fitted line was equal to zero, i.e., no linear relationship exists between PLR and SHL vehicle percentage.

![Figure 11.6: PLR vs. SHL Vehicle Percentage.](image)

To construct a statistical decision rule on the null hypothesis, it is necessary to calculate the $F$-value, which is the ratio between the mean square due to regression (MSR) and the mean square error (MSE). Since $F$-value follows the $F$ distribution, the $F$-value higher than $F_{(1-\alpha; 1, n-2)}$ indicates rejection of the null hypothesis, where $n$ is the number of samples. The analysis of variance (ANOVA) results are provided in Table 11.3. The one-way ANOVA results indicated that $F$-value was equal to 28.08, while $F_{(0.95; 1, 8)}$ was calculated as 5.32. Therefore, the null hypothesis was rejected, indicating that there exists a linear relationship between the PLR and SHL vehicle percentage.
vehicle percentage. $F$-test statistic results, coupled with the $R$-square value (0.77) of the fitted line, highlighted the SHL vehicle movements and their detrimental impacts on service life of pavement facilities in overload corridors.

Table 11.3: Analysis of Variance (ANOVA) Table for Testing the Linear Relationship between PLR and SHL Vehicle Percentage

<table>
<thead>
<tr>
<th>Source of Variation</th>
<th>df</th>
<th>Sum of Squares</th>
<th>Mean Square</th>
<th>$F$-value</th>
<th>Significance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>1</td>
<td>1,999.65</td>
<td>1,999.65</td>
<td>28.08</td>
<td>0.00</td>
</tr>
<tr>
<td>Error</td>
<td>8</td>
<td>569.75</td>
<td>71.22</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Total</td>
<td>9</td>
<td>2,569.40</td>
<td>2,070.87</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: df = degrees of freedom.

The loss of pavement service life is attributed not only to the frequency of the SHL vehicles, but also, more substantially to the load magnitudes on the vehicle axles and tires. This was the motivation for the authors to conduct similar analysis in order to examine the possible relationship between the ATHWL, as an index linked to the traffic load magnitude, and the imposed pavement life reduction.

Figure 11.7 shows the relationship between these two parameters, namely ATHWL and PLR. The one-way ANOVA results are also provided in Table 11.4. The results confirmed that the $F$-value (31.37) was higher than $F_{(0.95; 1, 8)} = 5.32$. Therefore, the null hypothesis was rejected, showing the linear relationship between the PLR and ATHWL. Accordingly, the results obtained from $F$-test statistic on one hand, and the $R$-square value (0.80) of the fitted line on the other, underlined the heaviest wheel load magnitude of the passing vehicles and its relevance to the damages accumulated during the design life of pavement facilities.
11.4. Parametric Analysis of the Loss of Pavement Service Life

A series of parametric analyses was conducted to investigate the influence of major analysis parameters on the loss of pavement service life imparted by SHL vehicle operations. To perform sensitivity analysis in this subtask, different inputs for traffic, pavement structure, and environmental factors, were incorporated into the ME analysis routine to determine the SHL-

---

Table 11.4: Analysis of Variance (ANOVA) Table for Testing the Linear Relationship between PLR and ATHWL

<table>
<thead>
<tr>
<th>Source of Variation</th>
<th>df</th>
<th>Sum of Squares</th>
<th>Mean Square</th>
<th>F-value</th>
<th>Significance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>1</td>
<td>2164.28</td>
<td>2164.28</td>
<td>31.37</td>
<td>0.00</td>
</tr>
<tr>
<td>Error</td>
<td>8</td>
<td>551.92</td>
<td>68.99</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Total</td>
<td>9</td>
<td>2,716.2</td>
<td>2,233.27</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: df = degrees of freedom.
induced PLR under various scenarios. Three representative roadways, i.e., FM 468, SH 123, and US 281 with the most demanding loading conditions, experiencing the greatest PLR levels were selected as case studies in this analysis. The following sections describe the rationale behind the sensitivity analysis and the items incorporated in each analysis category, which is complemented by the post-processed results and relevant discussions.

11.4.4.1. Influence of Traffic Demand Variations on PLR

Traffic demands throughout the transportation networks are essentially dynamic in nature. Hence, the accommodated traffic demands are not monolithic throughout the years. Specifically, traffic patterns in the overload corridors are primarily subjected to large traffic fluctuations during their design service life, due to the changes in variety of economic activities such as energy production, freight, and transportation of heavy machinery and equipment. A prime example of such unforeseen traffic fluctuations is found in highly active oil fields in the Permian Basin and the Eagle Ford Shale regions in Texas. The energy production activities on one hand, and the increase in freight transportation due to improvements in economic activities of the state on the other, have resulted in unprecedented increase in truck traffic operations during the past decade.

Similarly, transportation networks throughout the overload corridors might also experience significant decreases in the accommodated traffic demands. For instance, a dramatic reduction in traffic demand has been observed during 2020 and 2021 due to the COVID-19 pandemic that produced noticeable declines in truck traffic movements.

To investigate the influence of traffic demand variations on the PLR in this subtask, the authors considered different percentage changes, both increase and decrease, in traffic volume ranging all the way from 10% to 50% that represent “small”, “moderate”, and “large” fluctuations in the passing traffic. Then, corresponding values for ADT and the projected ESAL, as major
parameters representing the traffic volume, were incorporated into the sensitivity analysis for further comparisons. Ultimately, the variations in the imparted PLR were monitored accordingly.

Figure 11.8(a) and 11.8(b) provide the variations of the PLR level with changes in the volume of traffic demand associated with different roadway types. The results are reported as “percent increase” and “percent decrease” in the PLR, when new adjusted traffic volume, instead of 2019 traffic characteristics determined by the P-WIM devices in the field, were incorporated in the developed analysis framework.

As evidenced in Figure 11.8, traffic demand fluctuations directly affected the PLR imparted on the pavement sections. Furthermore, for all traffic input permutations, FM 468 roadway had the highest sensitivity to the changes in the volume of traffic, followed by SH 123, and ultimately US 281 roadway. This is mainly attributed to the demanding traffic makeup in FM 468 roadway, coupled with its pavement profile with extremely thin asphalt treated surface layer that was not designed to accommodate significant changes in the passing traffic. Hence, any changes in the traffic volume can greatly impact the predicted service life for FM 468 roadway.

According to the numerical simulation results provided in Figure 11.8(a), increasing the cumulative traffic volume over the 20-year design life by 50% in FM 468 results in 30% increase in the PLR imparted in this roadway; while the same increase in traffic volume of SH 123 and US 281 roadways increases the corresponding PLR levels by 20% and 15%, respectively. This highlights the type of roadway facility and its relevance to the incremental damages imparted on pavement facilities under different traffic demand scenarios. Another noteworthy observation from Figure 11.8(a) pertains to the fact that unexpectedly large traffic fluctuations might overwhelm the transportation network’s capability for accommodating future traffic demands in overload corridors.
Based on the sensitivity analysis results provided in Figure 11.8(b), decreasing the projected traffic volume by 50% in FM 468, SH 123, and US 281 roadways can potentially reduce the imparted loss of pavement service life by 43%, 33%, and 23%, respectively. Such significant decreases in the PLR further clarify how adopting proper mitigation strategies by stakeholders and...
controlling the traffic volume can contribute to the prolonged service life of pavement infrastructures in overload corridors.

The results of this sensitivity analysis revealed that overlooking the dynamic nature of traffic demands in transportation networks can potentially jeopardize damage quantification and PLR studies. This is because of the fact that the unaccounted for traffic fluctuations is one of the major sources of overestimation/underestimation of the damages accumulated over the pavement design life. Consequently, pavement design and analysis protocols should properly account for the change in the truck traffic volumes and its role on the accumulated damages; otherwise, it will be a major challenge to accommodate traffic demand fluctuations in transportation networks in overload corridors.

11.4.4.2. Influence of Traffic Growth Rate on PLR

Traffic growth rate is an essential component of traffic information that significantly influences the PLR analysis. To evaluate the influence of traffic growth rate on the loss of pavement service life in this subtask, the authors considered different traffic growth rates, i.e., 2%, 3%, 4%, 5%, and 6%, compounded annually, to project the traffic mix over the design life of the pavements. The corresponding 20-year ESAL values were then incorporated into the ME pavement analysis routine for further comparison purposes.

Figure 11.9 illustrates the variation of the imparted PLR with incremental increases in the traffic growth rate. The ascending nature of the PLR values in Figure 11.9 shows the influence of traffic growth rates on the predicted loss of pavement service life. As evidenced in this plot, increasing the traffic growth rates results in exponential increase in the PLR levels. The results also indicated that FM 468 roadway with less robust pavement structure was more sensitive to the
changes in traffic growth rate, compared to the studied pavement sections in SH 123 and US 281 highways with higher structural capacity.

![Figure 11.9: Influence of Traffic Growth Rate on the Loss of Pavement Service Life.](image)

Based on the sensitivity analyses performed in this subtask, incorporating 2% growth rate into the analysis framework results in reduction of 48%, 28%, and 22% of the service life of pavement sections in FM 468, SH 123, and US 281 roadways, respectively. The loss of pavement service life was more substantial for permutations with higher growth rates than 2%. Based on the results provided in Figure 11.9, projection of the traffic mix based on 6% growth rate resulted in significant PLRs as high as 68%, 38%, and 28% for FM 468, SH 123, and US 281 roadways, respectively. Accordingly, proper estimation and incorporation of traffic growth rate is the key step for accurate assessment of the loss of service life imparted on pavements.

It is worth noting that review of the traffic growth attributed to all studied highway sections in this study showed that using 4% of growth rate value for traffic projection purposes could properly represent the traffic growth patterns in the studied region.
11.4.4.3. **Influence of Tire Pressure on PLR**

Tire pressure is another essential piece of traffic information that greatly influences the PLR analysis results. To investigate the influence of tire pressure on the imparted PLR, various tire pressure magnitudes were considered in the numerical simulations that ranged from 100 psi to 140 psi, while the corresponding PLR values were contrasted with each other for comparison purposes.

Figure 11.10 shows the sensitivity of the PLR to the changes in the tire pressure values. Based on the numerical simulation results provided in Figure 11.10, the loss of pavement service life is highly dependent on the tire pressure value. Evidently, increasing the tire pressure magnitude led to a substantial increase in the imparted loss of pavement service life. This is in line with our expectations, since the vehicles with higher tire-pavement contact stresses can impart higher level of damages on the pavement structures. This in turn translates into expedited deterioration of the pavement facilities and can impose significant loss of pavement service life under excessive contact stresses.

![Figure 11.10: Influence of Tire Pressure on the Loss of Pavement Service Life.](image)

According to the sensitivity analyses provided in Figure 11.10, inclusion of tire pressure as 100 psi in the analysis framework leads to reduction of 24%, 15%, and 11% of the service life
of pavement sections in FM 468, SH 123, and US 281 roadways, respectively. The imparted PLR was more significant for permutations with higher tire pressure magnitudes. As evidence in plot, incorporation of 140 psi for tire pressure in the analysis, resulted in substantial PLRs as high as 71%, 42%, and 29% for FM 468, SH 123, and US 281 roadways, respectively. Consequently, it is imperative to monitor the tire pressure magnitudes of the heavy vehicles operating in transportation networks to select an appropriate value that realistically represents the demanding loading conditions imposed on the pavement structures. It should be also noted that, in this study, based on the field testing data maximum measured tire pressure of 120 psi was selected for further numerical simulations.

Another noteworthy observation from Figure 11.10 pertains to the type of roadway facility and its relevance to the imparted loss of pavement service life under different tire pressures. As evidenced in Figure 11.10, FM 468 roadway showed the highest sensitivity to the tire pressure magnitude, among the three studied roadways. This highlights the inherent vulnerability of structurally deficient roadways, such as FM roadways, to traffic input scenarios, as opposed to the well-designed and well-maintained US Highways with less significant sensitivity.

11.4.4.4. Influence of Wheel Load on PLR for Different Roadway Types

The performance of pavement sections are highly affected by the wheel load of passing vehicles in the transportation networks. For this reason, the distribution and magnitude of vehicle wheel loads substantially contribute to the accumulated damages and loss of service life imparted on pavement sections. This motivated the authors to quantify the PLR induced by SHL vehicle movements under various wheel load scenarios. To accomplish this objective, the research team, considered different permutations of traffic inputs with the heaviest wheel load magnitudes ranging all the way from 6 kips to 12 kips.
Thus, in each series of numerical analysis, wheel loads heavier than the considered load magnitude were identified and removed from the axle load spectra. For instance, for the case scenario with wheel load magnitude as 6 kips, loads exceeding the 6 kips limit were removed from the traffic data and a new axle load spectra was developed. Subsequently, the axle load distribution factors were re-calculated, and the traffic mix was then regenerated that correspond to the highest wheel load magnitude included in the sensitivity analysis. Ultimately, the corresponding 20-year ESAL values were projected and incorporated into the proposed PLR algorithm for further comparison purposes.

Figure 11.11 illustrates the variation of the PLR with incremental increases in the wheel load magnitudes that ranged between 6 kips and 12 kips. The results are reported as PLR imparted by SHL vehicle operations during the 20-year design life of the pavements corresponding to the heaviest wheel load incorporated into the loss of service life analysis protocol. The upward trend of the PLR values in Figure 11.11 clarifies the influence of wheel load magnitude and its contribution to the imparted loss of pavements service life. Based on the sensitivity analysis performed in this subtask, inclusion of the heavier wheel loads in the analysis results in substantial increase in the PLR levels. This observation was expected, since operations of the vehicles with heavier wheel loads can impart higher damages on pavements, and therefore, more substantially consume the service life of pavement facilities. The results also revealed that in all evaluated tire weights, FM 468 roadway with less robust pavement structure had the highest PLR, followed by SH 123, and eventually US 281 highway with higher structural capacity.
Figure 11.11: Influence of Wheel Load on PLR for Different Roadway Types.

Based on the sensitivity analyses provided in Figure 11.11, operation of the SHL vehicles with wheel loads equal to or below 6 kips, leads to reduction of 15%, 8%, and 6% of the service life of pavement sections in FM 468, SH 123, and US 281 roadways, respectively. The loss of pavement service life was more substantial for permutations with heavier wheel loads than 6 kips. As evidenced in Figure 11.11, inclusion of the heavy wheel loads up to 12 kips into the analysis framework, resulted in significant PLRs as high as 55%, 33%, and 25% due to SHL vehicle movements in FM 468, SH 123, and US 281 roadways, respectively.

Accordingly, based on the case scenarios evaluated in this subtask, increasing the maximum wheel load of passing vehicles from 6 kips to 12 kips leads to drastic increase, by 3-4 times, in the imparted loss of pavement service life during the 20-year design life. Such alarming increase in the loss of pavement life can result in premature failure of pavement facilities subjected to frequent applications of heavy wheel loads. Consequently, damage assessment and service life analysis protocols should properly account for the heaviest wheel load of SHL-vehicles and its contribution to distress accumulation in overload corridors.
11.4.4.5. Influence of Environmental Factors on PLR

Temperature and moisture are primary climatic factors that play an important role in the structural integrity and longevity of the pavement systems. Evidently, the structural capacity of multi-layer pavement systems is not monolithic under different temperatures. This is primarily attributed to the temperature-dependent nature of the viscoelastic AC layers. Additionally, the infiltration of water in pavement layers due to heavy precipitation, surface water flow, or Ground Water table (GWT) fluctuation can lead to a substantial change in structural integrity of pavements under traffic loading. For instance, in recent natural disasters, such as hurricane Harvey in Texas in 2017, segments of major highway pavements were under the water for nearly eight successive days. Exposing to such moisture conditions can potentially reduce the orthogonal stiffness of unbound aggregate systems. This reduction of stiffness properties in unbound systems is synonymous with the softening behavior of granular layers due to prolonged inundation conditions such as flooding events, and hence accelerates the rutting potential in granular layers.

Accordingly, the responses under taxing loading conditions, and consequently, the cumulative damage and pavement service life are affected by the temperature and moisture regimes that each pavement section experiences during its design service life. Hence, the detrimental impact of OW and SHL vehicles on flexible pavement structures is more pronounced when demanding environmental scenarios such as flooding conditions, poor moisture management situations under heavy rain events, and extremely hot climatic conditions, are incorporated into the damage quantification algorithms. This was the motivation to devise a comprehensive sensitivity analysis to account for different environmental scenarios and their contributions to the imparted loss of pavement service life in this study.
To perform sensitivity analysis in this subtask of the study, prominent climatic factors including annual temperature, precipitation rate, moisture management, and depth of Ground Water table (GWT), were incorporated in the analysis, considering the specific climatic conditions in different Districts across Texas overload corridors. Table 11.5 provides a summary of different environmental scenarios considered in the designed sensitive analysis. Major incorporated parameters, as well as relevant ranges/descriptions associated with each parameter, are also provided in Table 11.5. As indicated in Table 11.5, the following four environmental scenarios were investigated using the numerical simulation models in ME pavement design software:

- **Scenario #1**: Low Annual Temperature + Excellent Moisture Management,
- **Scenario #2**: Low Annual Temperature + Poor Moisture Management,
- **Scenario #3**: High Annual Temperature + Excellent Moisture Management,
- **Scenario #4**: High Annual Temperature + Poor Moisture Management.

### Table 11.5: Different Environmental Scenarios Considered in the Analysis

<table>
<thead>
<tr>
<th>Environmental Scenario</th>
<th>Major Climatic Factors Incorporated in the Sensitivity Analysis</th>
<th>Mean Annual Temperature (°F)</th>
<th>Moisture Management</th>
<th>Mean Annual Precipitation (in.)</th>
<th>Wet-day Frequency</th>
<th>Depth of GWT&lt;sup&gt;1&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario #1 (Low Temperature + Excellent M.M.&lt;sup&gt;2&lt;/sup&gt;)</td>
<td></td>
<td>&lt; 70</td>
<td>Excellent</td>
<td>&lt; 30</td>
<td>Occasional</td>
<td>Equilibrium GWT</td>
</tr>
<tr>
<td>Scenario #2 (Low Temperature + Poor M.M.)</td>
<td></td>
<td>&lt; 70</td>
<td>Poor</td>
<td>&gt; 30</td>
<td>Frequent</td>
<td>GWT Close to Surface</td>
</tr>
<tr>
<td>Scenario #3 (High Temperature + Excellent M.M.)</td>
<td></td>
<td>&gt; 70</td>
<td>Excellent</td>
<td>&lt; 30</td>
<td>Occasional</td>
<td>Equilibrium GWT</td>
</tr>
<tr>
<td>Scenario #4 (High Temperature + Poor M.M.)</td>
<td></td>
<td>&gt; 70</td>
<td>Poor</td>
<td>&gt; 30</td>
<td>Frequent</td>
<td>GWT Close to Surface</td>
</tr>
</tbody>
</table>

<sup>1</sup> GWT = Ground Water Table  
<sup>2</sup> M.M. = Moisture Management
Summer-based material properties were incorporated into the analysis as representative pavement layers properties of roadways under climatic conditions with mean annual temperature over 70°F and subjected to extremely hot summer months to capture the hot climate effects. Additionally, the winter-based back-calculated layer modulus were used as representative pavement materials properties under relatively more moderate climate conditions with mean annual temperature below 70°F.

Furthermore, poor and excellent moisture management conditions were considered in the sensitivity analysis to investigate the effect of moisture management capability of roadway infrastructures on the longevity of pavements under heavy traffic loading conditions. Numerical permutations with mean annual precipitation over 30 in., frequent occurrence of wet days, and GWT close to surface, intend to mimic the demanding environmental conditions such as flooding scenarios during heavy rain events in roadways with poor moisture management capability. In contrast, the numerical simulations with less challenging moisture conditions include case scenarios with mean annual precipitation below 30 in., occasional occurrence of wet days, and GWT at the hydraulic equilibrium state, which represent low moisture ingress rates under excellent moisture management situations.

Figure 11.12 provides the numerical simulation results associated with the variations of the PLR under different environmental conditions for three representative roadways. Based on the sensitivity analysis performed in this subtask, in all studied roadway types, the loss of pavement service life based on hot climate conditions were substantially higher than the other counterparts with low annual temperature scenarios. This is primarily attributed to the viscoelastic nature of the asphalt layer, and softening of the surface layers due to excessive temperatures under hot climate
conditions, that can accelerate the rutting failure of flexible pavements accommodating super heavy traffic loads.

Figure 11.12: Influence of Environmental Conditions on the Loss of Pavement Service Life.

The post-processed results illustrated in Figure 11.12 properly captured the influence of moisture management capability of roadway infrastructures on the imparted PLR. As evidenced in the plot, operation of the heavy vehicles under heavy rain events with poor moisture management performance imparted substantially higher PLR levels, compared to the climatic scenarios with efficient drainages systems, equilibrium GWT, and occasional occurrence of wet days in the year. This observation was anticipated, because high precipitation rates, coupled with poor drainage systems under prolonged inundation conditions such as flooding or elevated GWT, can dramatically accelerate the infiltration of water in pavement layers. Hence, structural properties of unbound layers significantly reduce due to their softening behavior under such extreme moisture conditions. This softening behavior of unbound systems in turn translates into
early development of distresses and can potentially jeopardize the longevity of pavement structures under heavy traffic loads. The analysis of results also indicated that FM roadways with less robust pavement structure experienced considerably higher PLR under different environmental conditions, compared to the studied pavement sections in SH and US highways.

According to the sensitivity analyses provided in this subtask, the simulated pavement sections under moderate and dry climates with excellent moisture management capability performed better under the SHL vehicle operations during the 20-year design life, in comparison with the case scenarios simulated under hot and humid climates with poor moisture management performance. Figure 11.13 provides a qualitative summary of the relative severity of the loss of pavement service life imparted due to SHL movements under four environmental scenarios evaluated in this study.

Figure 11.13: Relative Severity of Loss of Pavement Service Life Imparted due to SHL Vehicle Operations under Various Environmental Scenarios.

The results provided in this section underscored the significance of environmental factors for proper characterization of the loss of pavement service life due to SHL vehicle operations.
Consequently, accurate assessment of potential environmental scenarios should be an integral component in PLR studies of pavement facilities servicing the overload corridors.

**11.4.5. Synergistic Influence of Wheel Load and Environmental Conditions on PLR**

The results of the sensitivity analysis performed in section 12.4.4. underlined the significance of wheel load magnitudes and environmental factors and their role for accurate assessment of distress accumulation in pavement facilities under SHL vehicle applications. Accordingly, wheel load and environmental conditions are the key components in analysis of the loss of pavement service life in overload corridors. Therefore, damage assessment and service life analysis protocols should properly account for the simultaneous effects of these parameters.

This was the motivation for the authors to synthesize the relevant information obtain from the series of numerical simulations for all ten representative sites in this study to provide better insights on the loss of pavement service life under simultaneous effects of complex traffic loads and demanding environmental scenarios. The results were then post-processed and clustered for roadways with similar pavement structural properties to more realistically represent the results achieved by PLR studies associated with different roadway types.

Figure 1 illustrates the 3D color coded plots developed in this study to represent the loss of pavement service life imparted in various roadway types due to SHL vehicles, considering the synergistic influence of various wheel loads and environmental scenarios. The areas highlighted in warm colors (red and yellow) indicate substantially higher level of PLR, in contrast to the
regions highlighted in cold colors (blue and light blue) with lower PLRs. The developed contour plots shown in Figure 1.14 properly captured the influence of wheel load magnitude and environmental conditions on the imparted PLR. As evidenced in the plots, operation of SHL vehicles with heavy wheel loads is extremely detrimental to the longevity of pavement facilities, particularly under severe climate conditions with hot temperature and heavy precipitation regimes, coupled with poor moisture management performance of roadway infrastructure.

The results provided in Figure 1.14 also underscored the significance of the type of roadway facilities in PLR studies. Evidently, in all numerical permutations, FM roadways experienced substantially higher “reduction of service life”, compared to the SH and US highways. This is due to the fact that structurally deficient FM roadways are less efficient in dissipating the stresses induced in the pavement layers under extreme loading/climatic conditions. Such intrinsic vulnerability of FM roadways to demanding loading and environmental scenarios can potentially accelerate the incremental progression of distresses over the design service life.

Accordingly, FM roadways with less robust pavement structures and less efficient moisture management systems, compared to SH and US highways, are more susceptible to heavy loading conditions imposed by SHLs, when coincided with demanding environmental scenarios such as elevated temperature regimes and extreme moisture ingress conditions under flooding events.
Figure 11.14: Contour Plot Showing the Synergistic Influence of Wheel Load and Environmental Conditions on PLR for: (a) Farm-to-Market Roadway, (b) State Highway, and (c) US Highway.
The 3D contour plots presented in this chapter, can be instrumental for pavement design practitioners in having a mechanistic means for characterization of the loss of pavement service life imparted by SHL vehicles operating in overload corridors. Furthermore, the synthesized plots can be potentially used for load restriction decision making which results in provision of safe traffic passage as well as preservation of pavements’ structure under demanding loading conditions and/or extreme climatic events.

11.4.6. Summary of the Major Points
This subtask of the study revolved around development of a framework for the mechanistic characterization of the loss of pavement service life due to SHL vehicle operations. The devised framework accounts for the demanding loading conditions induced by SHLs, unique features of transportation systems, and the environmental factors for accurate assessment of the pavement service life in overload corridors. Mechanistic analyses were performed and the post-processed results associated with pavement life reduction in representative FM, SH, and US highways affected by the SHL applications were provided. Moreover, the authors conducted a series of parametric analysis to investigate the influence of several influential factors attributed to the traffic, pavement structure, and environmental factors on the loss of pavement service life imparted by SHL vehicles. The results were then synthesized and clustered for roadways with similar pavement structural capacities to more realistically represent the results obtained from PLR studies. The major findings of this subchapter summarized as:

- The analysis of loss of pavement service life in this study underscored the significance of the SHL vehicle operations and its role to jeopardize the longevity of pavement facilities.

The numerical simulation of ten representative pavements sections in Texas energy sectors
indicated that operation of SHL and OW vehicles in FM roadways can impart substantial PLRs as high as 55%, and 62%, respectively, over the 20-year design life. The greatest loss of pavement service life due to heavy vehicle movements ranged between 33% and 38% for the State Highways, and between 25% and 32% in US Highways.

- One passage of SHL vehicles induces higher level of damage and therefore higher pavement life consumption, compared to one movement of OW trucks. This is because SHL vehicles carry extremely heavy axles and tires that are primarily characterized in the tail end of the load spectrum, and hence can potentially impose more taxing stress paths on pavement structures.

- The primary source of pavement life consumption in the evaluated pavement sections in this study was associated with the cumulative surface rutting due to heavy truck traffic. This observation was anticipated as the distresses associated with the heavy truck operations are mainly attributed to the load magnitude rather than the load repetitions. Consequently, rutting was the primary distress responsible for the loss of pavement service life in OW corridors.

- A comparative analysis of various traffic input scenarios underscored the significance of prominent features, such as traffic demand fluctuations, traffic growth rate, and tire pressure magnitude, for proper characterization of the loss of pavement service life imparted in this study. The numerical simulation results showed that increasing the projected traffic demand over design period, anticipated annual growth rate, and applied tire pressure, results in substantial increase in the PLR. Accordingly, service life analysis protocols should include realistic measures of traffic parameters for proper prediction of distress accumulation in overload corridors.
• The detrimental impact of passing SHL vehicles was substantially higher under heavier wheel load magnitudes. Based on the case scenarios evaluated in this research, increasing the maximum wheel load of passing vehicles from 6 kips to 12 kips leads to drastic increase, by 3-4 times, in the imparted PLR during the 20-year design life. Consequently, design and analysis of pavement facilities servicing overload corridors, should properly account for the heaviest wheel load of SHL vehicles and its role to jeopardize the longevity of pavement structures.

• The results highlighted the synergistic damaging influence of taxing loading conditions when combined with demanding environmental scenarios such as elevated temperature regimes and/or extreme moisture conditions during flooding events. Based on the sensitivity analyses provided in this research, operation of the heavy vehicles under hot and humid climates with frequent heavy rain events and poor moisture management performance imparted substantially higher PLR levels, compared to moderate and dry climatic scenarios with low annual temperature regimes, and occasional occurrence of wet days in the year. This is primarily attributed to the softening of the viscoelastic asphalt layers due to elevated temperatures in hot climates and reduction in stiffness properties of granular layers due to moisture infiltration under prolonged inundation conditions such as flooding or elevated GWT during the service life of pavements with poor drainage systems. Consequently, accurate assessment of potential environmental scenarios should be an integral component in PLR studies of pavement facilities servicing the overload corridors.

• The numerical simulations underscored the significance of the type of roadway facilities in PLR studies, as the FM roadways experienced substantially higher “reduction of service life”, compared to SH and US highways under SHL vehicle movements. This is due to the
fact that FM roadways with less robust pavement structures and less efficient moisture management systems are not capable of effectively dissipating the stresses induced under demanding loading/climatic conditions.

- The 3D color coded plots developed in this study clarified how adopting proper mitigation strategies and controlling the traffic loading conditions can contribute to the prolonged service life of pavement infrastructures in overload corridors. Furthermore, the developed contour plots can be potentially used by stakeholders for load restriction decision making which results in provision of safe traffic passage as well as preservation of pavements’ structure under demanding loading conditions and/or extreme climatic events.

- Quantification of the loss of pavement service life imparted by heavy truck operations is the primary step for the selection of proper M&R strategy in transportation networks in overload corridors. This can potentially protect state assets by reduction or elimination of reconstruction costs associated with premature failure of the transportation facilities. Accurate assessment of such detrimental impacts is also beneficial in upgrading the current pavement design protocols to account for the demanding loading conditions induced by SHL vehicles.
11.5. Analysis of the Slow-Moving Nature of SHL Vehicles

11.5.1. Introduction

SHL vehicles commonly operate at relatively low speed due to the weight- and size-related attributes of heavy loads, as well as the safety concerns. Low vehicle operational speeds can essentially translate into a higher level of damages imparted on the pavements due to the viscoelastic behavior of AC layers. Hence, detrimental impact of SHLs on flexible pavement structures is more pronounced when the slow-moving nature of these vehicles is incorporated in the damage quantification algorithms (Morovatdar et al., 2021b). Consequently, it is necessary to properly assess the distresses imparted by SHL vehicles at multiple operational speeds, such as conventional speed and slow-moving conditions for further comparison, with consideration of seasonal effects on material properties, and type of pavement facility in the networks. The results can provide insights on the PLR studies, and provide means to improve current pavement design protocols.

Several researchers investigated the influence of the operational speed of conventional vehicles on the pavement responses (Ulloa et al., 2013; Yoo et al., 2006). In a recent study, Wu et al. (2020) developed a program to evaluate the effect of loading patterns and vehicle speed on the critical pavement responses. Based on the numerical modeling performed on four representative pavement structures, the authors concluded that the pavement responses decreased as the vehicle speed (i.e., loading frequency) increased. The researchers considered different shapes (i.e., rectangle, sinusoid, ellipse, and circle) of loads to simulate the contact stresses at the tire-pavement interface. Bazi et al. (2020), using numerical simulations, quantified the pavement surface deflection under a moving wheel load at multiple speeds. The numerical simulation results indicated that decreasing the speed from 62 mph to 25 mph led to an increase in the maximum...
surface deflection of the pavements by up to 17%. In the aforementioned study, the authors considered the uniform distribution of tire-pavement contact stresses.

Despite the fact that previous studies improved the existing knowledge of pavement performance prediction under various vehicle speed scenarios, the slow-moving nature of the non-conventional SHLs, as well as their impacts on the pavement service life, is often overlooked in the literature. Hence, the authors devised a protocol to assess the influence of the slow-moving nature of SHL vehicles on the imparted damages and loss of service life of pavements in ten overload corridors. The proposed approach accounts for the demanding loading conditions attributed to SHLs, site-specific ALS databases, various vehicle operational speeds, realistic tire-pavement interactions, and unique features of pavement facilities in the network.

11.5.2. Proposed Methodology

This section presents the developed procedure for the mechanistic characterization of the loss of pavement service life due to the slow-moving nature of the SHL vehicles. A similar procedure as elaborated in Section 12.4., but with different permutations for vehicle operational speed, was followed to investigate the influence of slow-moving nature of SHLs on the service life of pavement structures. To assess the loss of pavement service life in this subtask, initially, a series of numerical simulations were performed for determination of the critical pavement responses under the SHLs at various speeds, i.e., 60 mph for conventional highway speed condition, and 10 mph, 20 mph, and 30 mph to represent the slow-moving nature of SHLs. Subsequent to determination of the pavement responses, the research team calculated four sets of damage equivalency factors attributed to different speed scenarios.

At the last stage of the analysis, the research team, based on the calculated damage factors, determined the projected ESAL values over a 20-year design life for the evaluated case scenarios,
i.e., ESAL (conventional speed) and ESAL (slow-moving), to characterize the traffic in numerical simulations for further comparison purposes. Ultimately, the ESAL values were incorporated in the ME pavement design software for comparative analysis of service of life of pavements for the aforementioned sets of traffic data. The analysis results were contrasted with the conventional highway speed of 60 mph, which serves as the benchmark in this analysis. Additionally, “pavement life reduction” index was calculated to provide a quantitative measure of the severity of the distresses imparted due to the slow-moving nature of SHL vehicles in the network as:

\[
PLR_{\text{slow-moving}} = \frac{PL_{\text{conventional speed}} - PL_{\text{slow-moving}}}{PL_{\text{conventional speed}}} \times 100
\]

where,

- \(PLR_{\text{slow-moving}}\): imparted pavement life reduction due to slow-moving nature of SHLs.
- \(PL_{\text{conventional speed}}\): expected pavement life considering the conventional speed of SHLs.
- \(PL_{\text{slow-moving}}\): expected pavement life considering the slow-moving state of SHLs.

11.5.3. Analysis of Results and Discussions

11.5.3.1. Influence of SHL Vehicle Speed on Damage Factors

Figure 11.15 provides the axle load equivalency factors for various vehicle operational speeds (i.e., 10 mph, 20 mph, 30 mph, and 60 mph) for SHL tandem-axle load group in State Highways. According to the results provided in Figure 11.15, decreasing the vehicle speed results in substantial increase in the axle load equivalency factors, which in turn translates into higher damages imparted on pavement facilities. For the case scenarios evaluated in this study, in all axle weights, SHLs with operational speed of 60 mph had the lowest damage factors, while the incorporation of 10 mph vehicle speed into the damage algorithms led to the highest damage equivalency factors. This is primarily attributed to the fact that when the SHL-vehicle operates at
relatively low speed, the induced pavement responses increase due to the time-dependent nature of the viscoelastic AC layers subjected to longer loading time. In other terms, the longer loading time in flexible pavements with viscoelastic AC layers can essentially lead to larger creep deformation, and deeper rutting accumulated in the pavement structures. Consequently, damage assessment and service life analysis protocols should properly account for the SHL-vehicle operational speed and its contribution to the distress accumulation in overload corridors.

Figure 11.15: Pavement damage factors for SHL tandem-axles at various vehicle speeds.

Similar EALF trends were observed from the numerical simulations for various axle types and roadway types evaluated in this research project. Tables 11.6 presents summary of the differences between the 10 mph-based EALFs and the 60 mph-based damage factors associated with various roadway types and axle load groups. The results are reported as “percent increase” of EALF values when the slow-moving state, instead of conventional speed condition, of the SHL vehicles are incorporated into the damage assessment protocols. As indicated in Table 11.6, the difference between the slow moving-based and conventional speed-based EALFs ranges from 15 percent to 68 percent, depending on the type of roadway facility and the evaluated axle groups. Such substantial differences between the two evaluated scenarios underscored the significance of
the slow-moving nature of the SHLs and their detrimental impacts on the accumulated damages imparted on the pavement facilities.

Table 11.6: Percent Increase of EALF Values due to Slow-Moving State of SHL Vehicles

<table>
<thead>
<tr>
<th>Roadway Type</th>
<th>Axle Type</th>
<th>Single</th>
<th>Tandem</th>
<th>Tridem</th>
<th>Quad</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM</td>
<td></td>
<td>37%</td>
<td>68%</td>
<td>42%</td>
<td>26%</td>
</tr>
<tr>
<td>SH</td>
<td></td>
<td>30%</td>
<td>65%</td>
<td>36%</td>
<td>18%</td>
</tr>
<tr>
<td>US</td>
<td></td>
<td>27%</td>
<td>57%</td>
<td>30%</td>
<td>15%</td>
</tr>
</tbody>
</table>

11.5.3.2. Influence of SHL Vehicle Speed on Pavement Service Life

Figure 11.16 provides the results pertaining to one of the heavily trafficked sections, SH123 in Corpus Christi, as an example to showcase the analysis protocol developed in this study for mechanistic quantification of the loss of pavement service life. The plot shows the progression of distresses, i.e., rutting and fatigue cracking, over a 20-year design period for SH123. Based on the internal distress calculation models in mechanistic analysis, for steady rolling-based damage factors at vehicle speed of 60 mph, it takes 220 months for SH123 to develop 0.5 in. of rut depth. However, if the slow moving-based damage factors at vehicle speed of 30 mph, 20 mph, and 10 mph were incorporated into the analysis framework, it takes 177, 168, and 152 months to observe 0.5 in. rut depth, respectively. Consequently, consideration of the slow-moving nature of the SHLs operating in SH123 at vehicle speeds of 30 mph, 20 mph, and 10 mph resulted in substantial pavement life reductions as high as 20%, 24%, and 31%, respectively, based on the rutting criteria. This underscores the significance of consideration of vehicle speed for accurate determination of the loss of pavement service life due to SHL operations.
The analysis of the service life for SH123 also indicated that the rutting was the primary distress responsible for the loss of pavement service life in this roadway, as it takes longer for fatigue performance of SH123 to reach the failure criterion, i.e., 50% AC fatigue cracking area. Based on the sensitivity analysis performed in this study, the predicted fatigue cracking for SH123 with different permutations of SHL vehicle speed, rarely reached or surpassed the distress limits over the 20-year design period.
11.5.3.3. Pavement Life Reduction for Various Roadway Types

Figure 11.17 illustrates the results associated with the pavement life reduction for various roadway types, i.e., FM, SH, and US highways, due to the slow movement of SHLs at multiple vehicle speeds, i.e., 10 mph, 20 mph, and 30 mph. Based on the sensitivity analyses performed in this subtask of the study, FM roadways with less robust pavement structures were found to be more sensitive to the slow movement of SHL vehicles, compared to the pavement sections in SH and US highways with higher structural capacity. This highlighted the importance of the pavement profile on the analysis of the slow-moving nature of the SHL vehicles.

The results provided in Figure 11.17 also underscored the significance of the operational speed of SHL vehicles. Based on the sensitivity analyses conducted in this subtask, operation of the SHLs at vehicle speed of 30 mph can potentially reduce the service life of pavements. The numerical simulations indicated reduction of 17%, 20%, and 27% of the service life of pavement sections in US, SH, and FM roadways, for SHLs speed of 30 mph. The loss of pavement service
life was more substantial for permutations with lower operational speed than 30 mph. Based on the results provided in Figure 11.17, the SHLs movement at vehicle speed of 10 mph can result in significant PLRs as high as 25%, 31%, and 36% for US, SH, and FM roadways, respectively. Consequently, the analysis of the loss of pavement service life in this study indicated that pavement structures servicing overload corridors can potentially experience excessive distresses due to the slow-moving nature of the SHL vehicles operating in the network.

Figure 11.17: Loss of pavement service life due to slow-moving nature of SHLs for different roadway types.

11.5.4. Summary of the Major Points

In this subtask of the study, a mechanistic approach was presented for the mechanistic characterization of the loss of pavement service life due to the slow-moving nature of the SHL vehicles. The authors performed a series of sensitivity analyses to investigate the influence of operational speed of the SHL vehicles on the damages accumulated in the pavement structures. Based on the sensitivity analyses performed in this study, slow-moving state of the SHLs can potentially result in higher level of damages imparted on the pavement facilities, compared to the same vehicle traveling at conventional speed conditions. This is primarily attributed to the
viscoelastic nature of the asphalt layer, and the loading rate dependency of the materials properties in flexible pavements.

The sensitivity analysis provided in this study showed that low operational speed of the SHL vehicles can result in substantial pavement life reduction. Notably, permutations pertaining to the FM roadways with less robust pavement profile exhibited significantly higher accumulated rut-depth in presumed 20-year service life, compared to SH and US highways. The post-processed results in this study revealed that inclusion of the slow-moving nature of SHLs into the damage assessment algorithms can result in significant PLRs as high as 25%, 31%, and 36% for the representative US, SH, and FM roadways, respectively. Consequently, analysis of the slow-moving nature of SHL vehicles should be an integral component in risk management studies of pavement facilities servicing the overload corridors.
11.6. **Analysis of the Acceleration/Deceleration Forces**

11.6.1. **Introduction**

The detrimental impact of SHLs on transportation infrastructure is more pronounced when acceleration and deceleration forces were incorporated in the damage quantification algorithms. Evidently, this is an ongoing statewide challenge for design agencies across the nation and worldwide, with limited precedence in the literature (Morovatdar et al., 2021a). Therefore, there is an urgent need to properly assess the distresses imparted by SHL vehicles at multiple loading scenarios, such as sudden acceleration and deceleration, with consideration of seasonal effects on material properties, and type of the pavement facility in the impacted networks.

Several researchers investigated the influence of acceleration and deceleration of the conventional vehicles on the tire-pavement contact stresses (Wang et al., 2012, and Hu et al., 2017). Based on the numerical simulations, the authors reported that decelerating (braking) vehicles can result in higher contact stresses and therefore higher pavement responses under sudden braking conditions.

Despite the fact that previous studies provided insights on pavement performance under different vehicle loading scenarios, the acceleration and deceleration of non-conventional SHLs with taxing loading conditions, as well as their impacts on the pavement service life is often overlooked in the literature. Hence, the authors devised a protocol study to assess the influence of acceleration and deceleration of SHL vehicles on the imparted damages and loss of service life of pavements in ten overload corridors. The developed approach accounts for the demanding loading conditions attributed to SHLs, site-specific ALS databases, various acceleration/deceleration patterns, realistic tire-pavement interactions, and unique features of pavement facilities in the network.
11.6.2. Proposed Methodology

This section explains the procedure developed to mechanistically characterize the loss of pavement service life due to acceleration and deceleration of the SHL vehicles. A similar procedure as elaborated in Section 12.4., but with different permutations for vehicle loading conditions, was followed to investigate the influence of acceleration/deceleration of SHLs on the service life of pavement structures. To assess the loss of pavement service life in this subtask, initially, a series of numerical simulations were performed for determination of the critical pavement responses under the SHL tires at various rolling conditions, i.e., steady rolling, acceleration, and deceleration (braking). It is noted that relatively low vehicle speed (i.e., \( v = 20 \) mph, \( \omega = 11.7 \) rad/s) to account for the slow-moving nature of SHL vehicles, as well as translational and rotational acceleration/deceleration rate (i.e., \( a = 10 \) ft/s\(^2\); \( \alpha = 5.8 \) rad/s\(^2\)) were incorporated into the comprehensive sensitivity analyses conducted in this study (Hu et al., 2017, Maurya et al., 2012). Subsequent to determination of the pavement responses, the research team calculated three sets of damage equivalency factors attributed to the evaluated loading scenarios.

At the last stage of the analysis, the research team, based on the calculated damage factors, determined the projected ESAL values over a 20-year design life for three evaluated loading scenarios, i.e., ESAL (steady rolling), ESAL (acceleration), and ESAL (deceleration), to characterize the temporal variation of vehicle speed in numerical simulations for further comparison purposes. Ultimately, the ESAL values were incorporated in the ME pavement design software for comparative analysis of service of life of pavements for the aforementioned three sets of traffic data. The analysis results were contrasted with the steady rolling conditions, which serve as the benchmark in this analysis. Additionally, “pavement life reduction” index was calculated to
represent the reduction of the service life associated with the acceleration/deceleration of the SHL vehicles as:

\[
PLR_{\text{Acc/Dec}} = \frac{PL_{\text{conventional speed}} - PL_{\text{slow-moving}}}{PL_{\text{conventional speed}}} \times 100
\]  

(2)

where,

- \(PLR_{\text{Acc/Dec}}\) = imparted pavement life reduction due to acceleration/deceleration of SHLs.
- \(PL_{\text{steady rolling}}\) = expected pavement life considering the steady rolling condition of SHLs.
- \(PL_{\text{acceleration}}\) = expected pavement life considering the acceleration condition of SHLs.
- \(PL_{\text{deceleration}}\) = expected pavement life considering the deceleration condition of SHLs.

11.6.3. Analysis of Results and Discussions

11.6.3.1. Influence of SHL Acceleration/Deceleration on Tire-Pavement Contact Stresses

Proper prediction of the tire-pavement contact stresses is the prelude for accurate assessment of the loss of pavement service life imparted by acceleration/deceleration of SHL vehicles. Figure 11.18 schematically shows the torques and forces applied to the SHL tire during acceleration and braking conditions to provide a better understanding of tire rolling impacts. Essentially, when a SHL tire operates under acceleration/deceleration condition, the additional driving/braking torques applied on the tire can significantly influence the resulting tire-pavement contact stresses due to the frictional interactions at the tire-pavement interface. Therefore, the pavement responses and the damages imparted on the pavement sections are impacted by acceleration/deceleration forces. This was the motivation for the authors to investigate the acceleration/deceleration impacts on tire-pavement contact stresses.
Figure 11.18: Torques and Forces Applied to the SHL Tire at (a) Acceleration, and (b) Deceleration (Braking) Conditions.

Figure 11.19 provides comparisons of the tire-pavement contact stresses developed under a SHL tire with 15 kips load magnitude as an example to showcase the influence of different tire rolling scenarios, i.e., steady rolling, acceleration, and deceleration (braking), on contact stress distributions. The demonstrated panel on the left side of the figure pertains to the vertical stresses; while the right-side panel shows the simulated longitudinal (tangential) stresses at the tire-pavement interface. Based on the 3D tire-pavement contact stress distributions provided in Figure 11.19, acceleration/deceleration of the SHL tire had negligible influence on the vertical stresses induced at the tire-pavement interface. As evidenced in the plots, the maximum vertical contact stress was calculated as 180 kips, 183 kips and 186 kips, under steady-rolling, accelerating, and decelerating conditions, respectively, within the middle tire ribs; while, the average value was calculated as 125 psi for all three cases. Such slight difference in the vertical contact stresses is as expected due to the identical load magnitude incorporated in all three case scenarios.
Figure 11.19: Tire-Pavement Contact Stresses under Various Loading Scenarios under a 15-kips SHL Tire: (a) Steady Rolling State, (b) Acceleration, and (c) Deceleration (Braking).

Despite observing negligible changes in the vertical stresses monitored in this study, inclusion of the acceleration/deceleration of the SHL vehicle greatly impacted the longitudinal (tangential) contact stresses in the tire footprint zone. According to the 3D plots illustrated in Figure 11.19, the steady-rolling SHL tire induced relatively low longitudinal stresses along the
contact length, which could be attributed to the minimal rolling resistance of the tire when moves at steady-rolling state. However, the longitudinal contact stress distributions were extremely demanding under acceleration/deceleration scenarios.

As evidenced in Figure 11.19, the maximum calculated longitudinal contact stress was 27 psi for the steady-rolling state; however, the corresponding contact stresses were found to be substantially higher under acceleration, and deceleration scenarios, with 105 psi, and 135 psi, respectively. Such substantial increase in the longitudinal contact stresses is primarily attributed to the pronounced frictional interactions at tire-pavement interface under accelerating/decelerating scenarios of SHLs that carry extremely heavy wheel loads. Therefore, any additional driving/braking torque applied on the SHL tire during acceleration/deceleration conditions leads to significant increase in the longitudinal contact stresses at the pavement surface.

The numerical simulations of various tire rolling conditions in this study also revealed that the influence of SHLs on tire-pavement contact stresses was more pronounced under decelerating scenarios, as the maximum longitudinal contact stress was higher under deceleration condition, compared to the corresponding value predicted under the same vehicle operating at acceleration condition. This is mainly due to the fact that when a tire is under deceleration condition, the generated braking torque acts in the same direction as the rolling resistance forces act on the tire, in contrast to the accelerating tire with opposite directions of the driving torque and rolling resistance forces. Hence, deceleration of SHLs can potentially result in higher frictional forces and therefore higher longitudinal tire-pavement contact stresses, in comparison with the frictional forces applied to the tire-pavement interface under accelerating SHL vehicle.

Another noteworthy observation from Figure 11.19 pertains to the shape of contact stress distribution patterns characterized in the numerical simulation models. As evidenced in Figure
11.19, the longitudinal contact stresses under the steady-rolling tire showed a reversed-shape pattern along the tire contact length, with positive values (forward stresses) in the footprint departing zone, and negative values (backward stresses) in the footprint approaching zone. Nonetheless, tire acceleration/deceleration tends to induce one-directional longitudinal contact stress distribution patterns with positive values (forward stresses) under deceleration, and negative values (backward stresses) under acceleration condition.

Figure 11.20 presents a comparative summary of the influence of tire rolling conditions on the tire-pavement contact stresses, namely, vertical and longitudinal stresses, at the middle tire ribs imposing the most critical contact stresses. As evidenced in Figure 11.20, the longitudinal tire pavement contact stresses are significantly affected by acceleration/deceleration of SHL vehicles.

The results provided in this section underscored the significance of sudden variations in SHL vehicle speed and its impact on the tire-pavement contact stress patterns. Consequently, design and analysis of pavement sections subjected to acceleration/deceleration of SHL vehicles should include realistic simulation of tire-pavement contact stresses associated with various tire rolling scenarios, rather than relying on simplifying assumptions to use uniformly distributed loads that overlook the acceleration/deceleration impacts.
Figure 11.20: Influence of Tire Rolling Condition on the Maximum Contact Stresses at the Center Rib of the 15-kips SHL Tire for: (a) Vertical Stress, and (b) Longitudinal Stress.
11.6.3.2. Influence of SHL Acceleration/Deceleration on Damage Factors

The EALF values attributed to the various tire rolling conditions (i.e., steady rolling, acceleration, and deceleration) were contrasted with each other for comparison purposes in Figure 11.21. The results are specifically tailored towards the SHL tandem-axle load group in State Highways. As evidenced in the plot, for all axle weights, decelerating (braking) vehicles had the highest damage factors, followed by accelerating and ultimately steady rolling vehicles. This is primarily attributed to the fact that when a tire is under deceleration/acceleration conditions, the resulting longitudinal tire-pavement contact stresses significantly increase due to the frictional interactions at the tire-pavement interface. Such excessive contact stresses lead to the development of significant shear stresses in the pavement structure, which in turn translates into higher shear rutting in pavements. Consequently, damage assessment and service life analysis protocols should properly account for the deceleration and acceleration of the SHLs and their detrimental impacts on the longevity of pavement structures.

Figure 11.21: Pavement damage factors for various tire rolling conditions of SHL tandem-axles.

Similar EALF trends were observed from the numerical simulations for various axle types and roadway types evaluated in this research study. Tables 11.7 presents a summary of the
differences between the steady rolling based-EALFs, as the benchmark measures, and the acceleration- and deceleration-based damage factors associated with various roadway types and axle load groups. The results are reported as “percent increase” of EALF values when the acceleration/deceleration scenarios, instead of steady rolling condition, of the SHL vehicles are incorporated into the damage assessment protocols. As indicated in Table 11.7, the difference between the acceleration-based and steady rolling-based EALFs ranges from 10 percent to 45 percent, depending on the type of roadway facility and the evaluated axle groups. The calculated deviation was more pronounced under decelerating scenarios. As evidenced in Table 11.7, inclusion of deceleration (braking) condition of SHLs can substantially increase the pavement damage factors by 17% to 82%, depending on the type of roadway facility and the evaluated axle group. Such substantial differences between the evaluated vehicle loading scenarios underscored the significance of the acceleration/deceleration of the SHLs and their detrimental impacts on the accumulated damages imparted on the pavement facilities.
Table 11.7: Percent Increase of EALF Values due to Acceleration/Deceleration of SHL Vehicles

<table>
<thead>
<tr>
<th>Roadway Type</th>
<th>Single</th>
<th>Tandem</th>
<th>Tridem</th>
<th>Quad</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM</td>
<td>24%</td>
<td>45%</td>
<td>28%</td>
<td>17%</td>
</tr>
<tr>
<td>SH</td>
<td>20%</td>
<td>43%</td>
<td>24%</td>
<td>12%</td>
</tr>
<tr>
<td>US</td>
<td>18%</td>
<td>38%</td>
<td>20%</td>
<td>10%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Roadway Type</th>
<th>Single</th>
<th>Tandem</th>
<th>Tridem</th>
<th>Quad</th>
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<tbody>
<tr>
<td>FM</td>
<td>44%</td>
<td>82%</td>
<td>51%</td>
<td>31%</td>
</tr>
<tr>
<td>SH</td>
<td>36%</td>
<td>78%</td>
<td>45%</td>
<td>23%</td>
</tr>
<tr>
<td>US</td>
<td>33%</td>
<td>68%</td>
<td>36%</td>
<td>17%</td>
</tr>
</tbody>
</table>

11.6.3.3. Influence of SHL Acceleration/Deceleration on Pavement Service Life

Figure 11.22 provides the results for SH123 in Corpus Christi as an example to showcase the analysis protocol for mechanistic quantification of the loss of pavement service life. Based on the internal distress calculation models in mechanistic analysis and steady-rolling based damage factors, it takes 168 months for SH123 to develop 0.5 in. of rut depth. However, if the acceleration-based damage factors were incorporated into the analysis framework, it takes 135 months to develop 0.5 in. of cumulative rut depth. In other terms, realistic consideration of acceleration patterns of SHL vehicles in SH123 resulted in (168-135 = 33) months reduction of the rutting service life of this pavement section. Additionally, deceleration considerations of the SHL vehicles in SH123 resulted in 63 months reduction in its pavement service life. Consequently, the analysis of the loss of pavement service life for SH123, based on rutting criteria, indicated that acceleration and deceleration of the SHL vehicles operating in this roadway can impart significant PLRs, as
high as 20% and 38%, respectively. It is worth noting that acceleration and deceleration of such SHLs in SH123 can potentially consume 8% and 14%, respectively, of its pavement service life, based on fatigue criteria.

Figure 11.22: Pavement life reduction for SH123 due to acceleration and deceleration of SHLs, based on: (a) Rutting, and (b) Fatigue cracking.
11.6.3.4. Influence of Acceleration/Deceleration Patterns on Pavement Life Reduction

The research team conducted a series of sensitivity analyses to investigate the influence of different patterns of SHL acceleration/deceleration on the loss of pavement service life. To determine the pavement service life reduction in this subtask of the project, various fractions of the SHLs involved in acceleration/deceleration scenarios were incorporated in the devised analysis framework for comparison purposes. Different percentages of accelerating/decelerating SHLs, i.e., 25% and 50% for occasional acceleration/deceleration conditions, and 100% to represent the frequent acceleration/deceleration conditions, were incorporated in the sensitivity analysis conducted in this study.

Figure 11.23 presents the results for the imparted loss of pavement service life in SH123 associated with different patterns of acceleration/deceleration of SHL vehicles. The post-processed results provided in Figure 11.23 show that increasing the percentage of accelerating/decelerating SHLs results in substantial increase in pavement life consumption. The imparted PLR is more pronounced in decelerating scenarios of SHL vehicles. For SH123 roadway, the results indicated that increasing the percentage of accelerating SHLs from 25% to 100% increased the PLR from 8% to 20%, while increasing the percentage of decelerating vehicles within the same range resulted in substantial increase in the PLR, from 12% to 38%. This underscored the significance of acceleration/deceleration pattern of SHLs and their impact on the accumulated damages imparted on pavement sections. Consequently, the algorithms for determination of the loss of pavement service life in overload corridors should have the flexibility to adjust when SHL vehicles show different patterns of acceleration/deceleration, with consideration of the type of roadway segment (i.e., straight segment, stop-go section, intersection, etc.) and different traffic control attributes.
Figure 11.23: Influence of acceleration/deceleration pattern on PLR for SH123.

11.6.3.5. **Pavement Life Reduction for Various Roadway Types**

Figure 11.24 shows the results associated with the loss of pavement service life for various roadway types (i.e., FM, SH, and US highways) due to frequent acceleration and deceleration of SHL vehicles. Based on the sensitivity analyses performed in this subtask of the project, FM roadways with less robust pavement structures are more impacted by the acceleration and deceleration of SHL vehicles, compared to the pavement sections in SH and US highways with higher structural capacity. This underscores the significance of the pavement profile on the analysis of the acceleration and deceleration of the SHL vehicles.

The results illustrated in Figure 11.24 also highlighted the importance of sudden variations in SHL vehicle speed. Based on the results provided in Figure 11.24, frequent acceleration of SHL vehicles in US, SH, and FM roadways can potentially consume 16%, 20%, and 26%, respectively, of the service life of their pavement sections; while, deceleration of such vehicles can result in PLRs as high as 25%, 38%, and 42% for US, SH, and FM roadways, respectively. Therefore, pavement structures servicing overload corridors, particularly at intersections or stop-go sections with significant frequencies of braking, can potentially experience excessive distresses due to the
absence of a protocol to account for horizontal and tangential forces exerted during acceleration and deceleration scenarios during the pavement design process.

Figure 11.24: Loss of pavement service life due to acceleration/deceleration of SHLs for different roadway types.

11.6.4. Synergistic Influence of Acceleration/Deceleration and Climate Conditions on PLR

A series of parametric analyses were performed to account for the influence of SHL acceleration/deceleration patterns, wheel load magnitudes, and climatic conditions in the network, on the loss of pavement service life. Hence, the following four case scenarios were incorporated in the sensitivity analysis conducted in this study as:

- **Scenario #1**: Frequent Acceleration/Deceleration + High Temperature Regimes,
- **Scenario #2**: Frequent Acceleration/Deceleration + Low Temperature Regimes,
- **Scenario #3**: Occasional Acceleration/Deceleration + High Temperature Regimes,
- **Scenario #4**: Occasional Acceleration/Deceleration + Low Temperature Regimes.

In the numerical simulation models, “frequent acceleration/deceleration” is defined when the entire (~100%) SHL vehicles are involved in acceleration/deceleration scenarios; while “occasional acceleration/deceleration” pertains to the case scenarios with approximately 25% accelerating/decelerating SHLs. Additionally, the simulated cases under hot climatic conditions
with mean annual temperature over 70°F and extremely hot summer months aim to imitate the “High Temperature Regimes”; however, “Low Temperature Regime” is defined when the simulated pavement section was subjected to more moderate climate conditions with mean annual temperature below 70°F. More specifics associated with different climate conditions, and the relevant items and ranges incorporated in the sensitivity analysis, are provided in Section 12.4.4.4. Ultimately, the numerical simulations results for all ten representative sites in this study were synthesized and then clustered for roadways with similar pavement structural properties to more realistically represent the results obtained from PLR studies associated with different roadway types. Additionally, since there is equal possibility for a roadway segment to experience either acceleration- or deceleration-related forces under the SHL vehicle movements, the results pertaining to deceleration (braking) of SHLs were included in the analysis, to consider more critical loading scenarios.

Figure 11.25 illustrates the 3D color coded plots developed in this study to represent the loss of pavement service life imparted due to acceleration/deceleration of SHL vehicles in various roadway types, considering various environmental scenarios and wheel load magnitudes included in the analysis framework. The areas highlighted in warm colors (red and yellow) indicate substantially higher level of PLR, in contrast to the regions highlighted in cold colors (blue and light blue) with lower PLRs. The developed contour plots shown in Figure 11.25 properly captured the synergistic influence of acceleration/deceleration pattern, wheel load magnitude, and climate conditions on the imparted PLR.
Figure 11.25: Contour Plots Showing the PLR Imparted due to Acceleration/Deceleration of SHLs with Various Wheel Load Magnitudes and under Different Climate Scenarios for: (a) FM Roadway, (b) State Highway, and (c) US Highway.
As evidenced in Figure 11.25, frequent acceleration/deceleration of SHL vehicles carrying heavier wheel loads is extremely detrimental to the longevity of pavement facilities, particularly under hot climates with high annual temperature regimes. This is primarily due to the stress softening of the viscoelastic AC layers under hot climate conditions, which in turn translates into higher rutting potential when combined with demanding loading conditions during acceleration/deceleration scenarios of SHL vehicles. Figure 11.26 provides a qualitative summary of the relative severity of the loss of pavement service life imparted by acceleration/deceleration of SHL vehicles during the 20-year design period under different loading/climatic scenarios evaluated in this study.

![Figure 11.26: Relative Severity of Loss of Pavement Service Life Imparted by Acceleration/Deceleration of SHL Vehicles under Various Loading/Climatic Scenarios.](image)

Another notable finding from Figure 12.25 pertains to the significance of the type of roadway facilities in PLR studies. Evidently, in all numerical permutations, FM roadways experienced substantially higher “reduction of service life”, compared to the SH and US highways. Consequently, FM roadways with less robust pavement structures and less stress dissipation capacities, compared to SH and US highways, are more vulnerable to taxing stress paths imposed under accelerating/decelerating SHLs, when coincided with demanding environmental scenarios.
such as elevated temperature regimes. Such inherent vulnerability of FM roadways to demanding loading/climatic scenarios can result in expedited deterioration of pavement structures throughout the rural roadway networks servicing SHL units.

Based on the results presented in this subtask of the study, the detrimental impact of SHL vehicles on the pavement service life is not monolithic for different segments of a roadway with different acceleration/deceleration patterns. Essentially, critical highway areas such as stop-go sections and speed hump areas with significant frequencies of braking and/or stopping SHLs are more likely to experience excessive distresses and therefore, higher loss of pavement service life, compared to the less impacted roadway segments including straight segments, intersections, junctions, and downhills/uphills, with rare or occasional acceleration/deceleration scenarios. Consequently, analysis of the acceleration and deceleration effects should properly account for the specific characteristics of the roadway segments with different traffic control attributes.

The synthesized contour plots presented in this chapter are tailored towards the specific patterns of accelerating/decelerating SHL vehicles in the surveyed network, with frequent and occasional occurrences of acceleration/braking scenarios. The developed contour plots can be further instrumental for pavement design practitioners in having a mechanistic means for characterization of the loss of pavement service life imparted by acceleration/deceleration of SHL vehicles under demanding loading/climatic events, with consideration of the traffic control characteristics in the SHL route.

11.6.5. Summary of the Major Points

In this subtask, a mechanistic-based approach was proposed for characterization of the loss of pavement service life due to acceleration and deceleration of the SHL vehicles. The developed approach accounts for the taxing stress paths imparted by SHLs, tire-pavement interactions, unique
features of transportation systems, and the environmental factors for accurate assessment of the pavement service life. The numerical simulation results highlighted the importance of sudden variations in SHL vehicle speed and its impact on the tire-pavement contact stress distributions. Based on the post-processed results, inclusion of acceleration/deceleration of SHLs in the analysis resulted in substantial increase in the longitudinal (tangential) tire-pavement contact stresses, due to the pronounced frictional interactions at tire-pavement interface under such loading scenarios. Therefore, pavement performance prediction and damage assessments in overload corridors should include realistic simulation of tire-pavement contact stresses under various tire rolling scenarios, rather than relying on simplifying assumptions that overlook the acceleration/deceleration impacts.

The sensitivity analysis provided in this study showed that accelerating and decelerating heavy vehicles can impart substantially higher damages on the pavement structures, compared to the same vehicle traveling at constant speed. This is primarily attributed to the dynamic nature of the analysis, and the added horizontal and tangential forces associated with speeding and braking actions. Additionally, mechanistic analysis of the pavement service life in this study showed that deceleration of the SHL vehicles results in higher levels of PLR, compared to the acceleration of these vehicles under similar conditions. Loss of pavement service life was more substantial in FM roadways with less robust pavement structures, compared to SH and US highways. The post-processed results for the representative pavement sections in this study revealed that frequent acceleration and deceleration of the SHL vehicles operating in FM roadways can impart PLR as high as 26% and 42%, respectively. Further investigation of the climatic factors in this study indicated that the detrimental impact of SHLs acceleration/deceleration on the pavement service life was more pronounced under hot climates with high annual temperature regimes, due to the stress softening of the viscoelastic AC layers under hot climate conditions.
The numerical simulations results also indicated that the fraction of SHLs involved in acceleration/deceleration scenarios is an essential component that contributes to the pavement life consumption in overload transportation networks. Hence, operation of SHLs at different segments of a roadway that experience different acceleration/deceleration frequencies can impart different levels of PLR. Accordingly, mechanistic approaches for characterization of the loss of pavement service life in overload corridors should have the capability for assessing the SHL vehicle impacts under various acceleration/deceleration patterns with frequent, occasional, and rare occurrences of accelerating/braking events.

The authors synthesized the numerical simulations results for ten representative sites in this study and then clustered for roadways with similar pavement structural capacity to more realistically represent the results obtained from PLR studies. The 3D contour plots developed in this study facilitates the implementation of the research findings to have an accurate account of the loss of pavement service life imparted by acceleration/deceleration of SHL vehicles, with consideration of the type of roadway facility, environmental factors, wheel load magnitude, acceleration/deceleration patterns, and route-specific traffic control attributes.

In summary, this subtask of study underlined the significance of acceleration/deceleration of SHL vehicles and their impact on the accumulated damages imparted on the pavement sections. Therefore, pavement structures, particularly at intersections, junctions, stop–go sections, or speed hump areas, can potentially experience excessive distresses due to absence of a protocol to account for the detrimental impacts of acceleration and deceleration scenarios during the pavement design process. Consequently, analysis of acceleration and deceleration of SHL vehicles should be an integral component in risk management studies of pavement facilities servicing the overload corridors.
11.7. **ANALYSIS OF THE ROADWAY GEOMETRIC FEATURES**

11.7.1. **Introduction**

Operation of SHL vehicles can impose excessive shear stresses at the pavement surface during turning movements at curved segments of the roadway, as compared with the same vehicle operating at straight sections. This is mainly due to the development of lateral frictional forces at tire-pavement interface that tends to counterbalance the induced centrifugal forces acting on the SHL vehicle when moves along the horizontal curves. This issue is more pronounced when the SHL vehicle consists of specialized trailers with non-steerable axles, as the frictional interactions substantially increase under such conditions. Therefore, it is imperative to differentiate between different types of roadway segments, i.e., curved and straight segments, with distinct geometric designs, for proper quantification of pavement damages imposed by super heavy vehicles.

The current procedure for SHL analysis used by TxDOT Maintenance Division also emphasizes the need to consider road geometry during the SHL route evaluation, as elaborated in Section 7 of Chapter 13th of TxDOT’s pavement design manual. Review of the current procedures commonly used by pavement design industry underlined the existence of a gap for proper evaluation of SHLs under turning/cornering movements at horizontal curve sections. Furthermore, geometric design and characteristics of roadway curve segments, as well as their impacts, on the pavement distresses accumulated under the SHL vehicle movements are often overlooked in damage mechanisms proposed in the literature. The lack of a mechanistic procedure to account for the geometric design features and their role on distress accumulation during the design and analysis procedures could be the plausible reason behind significant shearing of the pavement surfaces observed at horizontal curve segments in several overload corridors across the nation.
Consequently, this study was designed to assess the influence of roadway geometric features on the cumulative damages and imparted loss of service life for pavements subjected to SHL vehicle movements. The proposed mechanistic approach accounts for the site-specific geometric design characteristics, realistic tire-pavement interactions, demanding loading conditions attributed to SHLs, slow-moving nature of SHLs, acceleration/deceleration patterns, and unique features of pavement facilities in the network. The research team also performed a series of numerical simulations to assess the synergistic influence of several influential factors such as geometric design parameters, i.e., super-elevation and curve radius, wheel load magnitude, and roadway type, on the PLR imparted at curved segments. The results can provide insights on the imparted damage and loss of pavement service life when the SHLs operate at curved segments.

11.7.2. Background

Figure 11.27 provides a schematic view of various forces acting on SHL vehicle during turning movements at horizontal curve segments to provide a better understanding of the influence of roadway curvature on tire-pavement contact stress distributions. The schematic plot shows different forces, including centrifugal force acting on SHL vehicle ($F_c$), tire-pavement frictional force ($F_f$), SHL vehicle weight ($W$), and normal reaction force from the road ($N$). Essentially, passage of a vehicle through circular curves and/or highway interchanges results in development of centrifugal forces that act on the traveling vehicle, and can be calculated as:

$$ F_c = m \frac{V^2}{R} $$

where, $m$ represents the vehicle weight, $V$ is the vehicle operational speed, and $R$ is the curve radius. Based on the point-mass model described in AASHTO for horizontal curve design, the generated centrifugal force acts outwards the center of gravity of the vehicle moving around the center of road curvature. Lateral frictional forces are then developed at the tire-pavement interface.
as a consequence of the centrifugal effects to prevent skidding of the vehicle. Super-elevation is another essential component that tends to maintain the stability of the turning vehicle at horizontal curves. Super-elevation aims to improve vehicle safety and comfort during turning movements, by changing the road slope in the transverse direction. Super-elevation \((e)\) is defined as the rise (i.e., change in elevation) in feet per 100 ft across the road (i.e., in the transverse direction). Hence, 
\[
e/100 = \tan \alpha,
\]
where \(\alpha\) represents the angle of cross section shown in Figure 11.27.

![Figure 11.27: Lateral Forces Acting on SHL Vehicle at Turning Movements.](image)

As elaborated, the outward centrifugal forces acting on the turning vehicle are counterbalanced by the stimulated frictional forces at the tire-pavement interface, and/or the component of vehicle’s weight acting parallel to the road due to super-elevation. Consequently, tire-pavement contact stresses are significantly affected when transition of the SHL vehicle from straight segment to a curve segment occurs. This variation in contact stress distributions essentially translates into changes in the pavement performance, and therefore, level of damages imparted by SHL vehicles when move along different segments of a roadway, i.e., straight and curved segments. This motivated our research team to quantify the damages imparted by SHLs at various roadway segments with different geometric characteristics for further comparison purposes.

### 11.7.3. Proposed Methodology

This section describes the mechanistic procedure adopted to characterize the loss of pavement service life imparted at curved segments of a roadway due to SHL applications. A similar procedure as elaborated in Section 12.4., but with different permutations for roadway geometric
design, was followed to investigate the influence of horizontal roadway curvature on the loss of pavements service life imparted by turning SHL vehicles at curved segments. The predicted pavement life consumptions for curved segments were further contrasted with those estimated for straight segments to highlight the underestimation of the pavement distresses when roadway geometric features are overlooked in the service life analysis protocols.

To assess the loss of pavement service life in this subtask, initially, a series of numerical simulations were performed for determination of the critical pavement responses at roadway curved segments under SHL vehicle movements. Various roadway geometric characteristics, i.e., super-elevation and curve radius, were incorporated in the analysis framework to consider the influence of geometric features on the pavement responses. It is noted that relatively low vehicle speed (i.e., $v = 20$ mph, $\omega = 11.7$ rad/s) to account for the slow-moving nature of SHL vehicles, as well as occasional deceleration patterns, were included in the numerical simulation models to consider more taxing stress paths. Subsequent to determination of the pavement responses, the research team calculated the corresponding damage equivalency factors tailored towards the specific geometric characteristics of the studied roadway.

At the last stage of the analysis, the research team, based on the calculated damage factors, determined and incorporated 20-year ESAL values for two case scenarios of traffic inputs, i.e., $\text{ESAL}_{\text{(with SHLs)}}$ and $\text{ESAL}_{\text{(without SHLs)}}$, into the pavement life prediction analysis routine to characterize the SHL impacts in numerical simulations for further comparison purposes. The loss of pavement life associated with the SHL vehicle operations was then determined based on the distress plots and preset distress limits for each pavement section. Additionally, “pavement life reduction” index was calculated to provide a quantitative measure of the severity of the distresses imparted by SHL vehicles during turning movements at roadway curved segments.
11.7.4. Analysis of Results and Discussions

11.7.4.1. Tire-Pavement Contact Stress under SHL Movement at Roadway Curve Segments

Figure 11.28 provides comparisons of the 3D tire-pavement contact stresses developed under a SHL tire with 15 kips load magnitude as an example to showcase the influence of geometric characteristics of roadways on contact stress distributions. The numerical simulation results presented in Figure 11.28 pertain to different sections of SH 123 in Corpus Christi with different geometric designs analyzed in the FE program, including straight segment, curve segment without super-elevation, and curve segment with super-elevation. The demonstrated panel on the left side of the figure pertains to the vertical stresses; while the right-side panel shows the simulated transverse (shear) stresses at the tire-pavement interface. It is also noted that curve radius of 1,200 ft and super-elevation equal to 8% was considered in the numerical simulation models, based on the review of the roadway design plans for SH 123.

The 3D tire-pavement contact stress distributions shown in Figure 11.28 substantiated the fact that turning SHL tires at horizontal curve segments induce significantly higher peaks of vertical contact stresses within the contact zone, as compared to stresses characterized under the same tire normally traveling at straight paths. The primary culprit for the observed increase in the vertical contact stresses pertains to the tire inclination/cornering impacts during turning movements at curved segments that result in concentration of contact stresses shifting towards the right-hand side of the tire footprint zone in a right-turn curve. The localized stress concentration in turn translates into higher peak stresses with asymmetrical stress distribution patterns with respect to the centroid, under turning SHL tires.

Furthermore, the tendency of the vertical stresses to asymmetrical distributions with higher peaks increased with inclusion of the super-elevation to the simulated pavement profile. As
evidenced in the plots, the maximum vertical contact stress was calculated as 180 kips, 212 kips and 214 kips, at straight segment, curve segment without super-elevation, and curve segment with super-elevation, respectively. It is also worth noting that the peaks of vertical contact stresses at straight segment are located at the middle of tire ribs, while the peaks tend to shift towards the edge of tire ribs when turning movements occur. The numerical simulation results highlighted the importance of roadway geometric features for accurate assessment of the demanding loading conditions imposed by SHL vehicles.

Similar trends were also observed for transverse (shear) stresses monitored under the SHL tire moving at roadway segments with horizontal curvature. Based on the sensitivity results provided in Figure 11.28, the simulated curve segment with consideration of super-elevation experienced the highest transverse stresses as high as 82 psi, under SHL tire, compared with curve segment without super-elevation, and straight segment, with lower peaks as 68 psi, and 56 psi, respectively. The plausible reason behind higher transverse stress peaks at curved segments is the additional frictional forces developed at the tire-pavement interface that tend to counterbalance the induced centrifugal forces acting on SHL vehicle during turning movements.

Figure 11.29 presents a comparative summary of the influence of roadway geometric features on the tire-pavement contact stresses, namely, vertical and longitudinal stresses, at the critical tire ribs that impose the most demanding contact stresses. Based on the results, both vertical and transverse (shear) tire pavement contact stresses induced under SHL tire were significantly higher at curved segment as compared to the corresponding values calculated for straight segments. The results also showed that super-elevation was an essential parameter of roadway geometry that greatly contribute to the contact stress distributions at tire-pavement interface, as
the permutations with super-elevation experienced noticeably higher levels of contact stresses under SHL tire movements, among different case scenarios evaluated in this study.

The results presented in this subtask underscored the significance of roadway geometric features and their impacts on tire-pavement contact stress distribution patterns for pavements subjected to SHL vehicle operations. Consequently, realistic simulation of tire-pavement contact stresses should be an integral component in damage quantification algorithms for accurate assessment of the pavement performance and damages imparted by SHLs during turning movements at roadway curved segments. Otherwise, pavement structures, particularly at curved segments, intersections, junctions, or highway interchanges with frequent vehicle maneuvering occurrences can potentially experience excessive distresses, when the detrimental impacts of super heavy vehicles during turning/cornering scenarios are overlooked in the analysis procedure.
Figure 11.28: Tire-Pavement Contact Stresses under 15-kips SHL Tire at Different Roadway Segments: (a) Straight Segment, (b) Curve Segment without Super-elevation, and (c) Curve Segment with Super-elevation.
Figure 11.29: Influence of Roadway Geometric Design on the Maximum Contact Stresses under 15-kips SHL Tire for: (a) Vertical Stress, and (b) Transverse (Shear) Stress.
11.7.4.2.  Influence of Geometric Design Parameters on Pavement Damage Factors

Figure 11.30 demonstrates the pavement damage equivalency factors associated with different design parameters for super-elevation and curve radius to underline the influence of roadway geometric features on the damages imparted at curve segments under SHL vehicles. The results pertain to the SHL tandem-axle with 120 kips axle weight and 15 kips load magnitude on each individual tire simulated over the pavement profile of SH 123 in Corpus Christi. The authors considered different super-elevation rates, ranging all the way from 2% to 12%, and various radius of horizontal curve that varied between 1,200 ft and 3,200 ft, to cover the most prevalent range of geometric design parameters in the surveyed network, with consideration of the specifics provided in TxDOT roadway design manual, and AASHTO geometric design guide (*the Green Book*). It is worth noting that super-elevation was considered in all roadway curved segments simulated in this study to comply with the recommendations provided in Section 4: *Horizontal Alignment* of TxDOT roadway design manual for designing horizontal curvature in non-urban highways.

As evidenced in Figure 11.30(a), increasing the super-elevation rates results in considerable increase in the axle load equivalency factors, which in turn translates into higher damages imparted on the pavement facilities. This increasing trend in the pavement damage factors is mainly connected with the variations in tire-pavement contact stresses with changes in the super-elevation rate. The tendency to asymmetrical distribution of vertical contact stresses due to localized stress concentration on one hand, and the increase in transverse (shear) tire-pavement contact stresses on the other, are the primary reasons behind more substantial pavement damages characterized at higher super-elevation rates. It is noted that the curve radius was considered constant as 1,200 ft in the numerical simulations analyzed in this subtask.
Figure 11.30: Pavement Damage Factors for 120-kips SHL Tandem-Axle associated with Different Roadway Geometric Parameters: (a) Super-elevation, and (b) Curve Radius.

Based on the sensitivity results provided in Figure 11.30(b), pavement damage factors showed an ascending trend with decreasing the curve radius. This is mainly attributed to the higher centrifugal forces acting on the SHL vehicle at bend segments with low curve radius, due to the inverse relationship between radius of curvature and the developed centrifugal forces. Higher centrifugal forces at sharp curves can essentially result in higher lateral frictional forces induced at the tire-pavement interface to prevent skidding of the SHL vehicle over the pavement surface. This in turn translate into higher pavement responses, and therefore, higher damage equivalency
factors. It should be noted that the super-elevation was assumed as 8% in the sensitivity analysis provided in Figure 11.30(b).

Figure 11.31 shows the synergistic influence of super-elevation and curve radius on the pavement damage factors associated with the same SHL axle and pavement profile evaluated in this section. Based on the 3D plot illustrated in Figure 11.31, the worst case scenario pertains to the numerical simulations with higher super-elevation rate and lower curve radius, as the calculated damage factors under such geometric characteristics were substantially higher than the other counterparts. According to the sensitivity analysis performed in this subtask, the detrimental impact of turning SHL vehicles is more pronounced at sharp horizontal curves with low to intermediate radius of curvature and high super-elevation rate. Consequently, it is imperative to properly account for the site-specific geometric design parameters and their synergistic influence on the damage quantification mechanisms in OW highway networks.

Figure 11.31 also provides comparisons between the EALFs calculated for curved segments and the EALF value determined at straight segment to clarify the significance of roadway curvature on the imparted damages. As evidenced in Figure 11.31, in all numerical permutations, the curved-based EALFs were relatively higher than the EALF value obtained for straight segment. This underscores the significance of the type of roadway segment with distinct geometric designs for accurate assessment of the damage equivalency factors, rather than using single value average damage factor for the entire sections along the roadway.
Figure 11.31: Synergistic Influence of Super-elevation and Curve Radius on Pavement Damage Equivalency Factors for 120-kips SHL Tandem-Axle.

According to the sensitivity analysis provided in this section, operation of SHL vehicles during turning movements at roadway curved segments can impart substantially higher level of damages on the pavement facilities due to more demanding loading conditions induced at tire-pavement interface, as compared with the same vehicle normally operating at straight segments. The excessive shear and vertical contact stresses under turning SHL vehicles lead to the development of taxing stress paths within the pavement structure, and hence can potentially accelerate the shear/surface rutting failures. Hence, damage quantification algorithms should have the capability for assessing the detrimental impacts of SHL vehicles during turning movements at curved segments. Otherwise, the underestimation of the damages due to the absence of a protocol to account for the roadway geometric features can potentially incur systematic errors for the design and life-cycle cost analysis of pavement sections servicing overload corridors.
11.7.4.3. **Influence of Roadway Geometry on Damage Factors (Straight vs. Curve Segment)**

The EALF values attributed to the various roadway segments, i.e., straight segment and curve segment, were contrasted with each other for comparison purposes in Figure 11.32 to clarify the influence of roadway geometry on the damages imparted under SHL vehicle movements. The results are specifically tailored towards the SHL tridem-axle load group in State Highways. A relatively low curve radius as 1,200 ft, as well as high super-elevation rate as 8%, were incorporated in the FE models as representative geometric design parameters to consider more taxing tire-pavement interactions at curved sections.

As evidenced in Figure 11.32, for all axle weights, curved segments had substantially higher damage factors compared with the corresponding values calculated for straight segments of the roadway under the same SHL vehicles with identical loading conditions. This is primarily attributed to the fact that when a SHL vehicle travels along a horizontal curve, the resulting vertical and shear tire-pavement contact stresses significantly increase due to the localized stress concertation under turning tires, coupled with the lateral frictional interactions at the tire-pavement interface that tend to prevent skidding of the SHL vehicle. Such excessive contact stresses lead to the development of significant shear and compressive stresses in the pavement structure, which in turn translates into higher shear/surface rutting in pavement facilities. Consequently, damage quantification and service life analysis protocols should properly account for the type of roadway segments and the relevant geometric design features for accurate assessment of the SHL detrimental impacts on the longevity of pavement structures.

SHL vehicles that move along the roadway curved segments can potentially induce more demanding loading conditions on the pavement structures under accelerating/decelerating scenarios. To quantify the influence of decelerating SHL vehicles in this subtask, the authors
conducted another series of numerical simulations to calculate the damage equivalency factors tailored towards the SHL vehicles under sudden braking conditions at curved segments. Figure 11.32 provides the post-processed results associated with the decelerating SHL axles during turning movements at roadway curved segments. As evidenced in Figure 11.32, inclusion of the vehicle braking conditions into the damage analysis protocol resulted in substantially higher pavement damage equivalency factors at roadway curved segments, compared to the same SHL vehicle operating at steady-rolling condition. The results highlighted the simultaneous influence of roadway curvature and vehicle decelerating scenarios on the damages imparted on the pavement facilities. Consequently, curved segments of roadway can potentially experience excessive distresses under the SHL vehicle movements with frequent vehicle braking actions.

![Graph showing pavement damage factors for various axle weights and roadway segments](image)

**Figure 11.32: Pavement Damage Factors for Various Types of Roadway Segments under SHL Tridem-Axles.**

Similar EALF trends were observed from the numerical simulations for various axle types and roadway types evaluated in this research study. Tables 11.8 presents a summary of the differences between the straight segment based-EALFs, as the benchmark measures, and the curve segment-based damage factors associated with various roadway types, axle load groups, and under different vehicle loading conditions. The results are reported as “percent increase” of EALF values.
when the roadway curvature, instead of straight path, are incorporated into the damage assessment protocols. As indicated in Table 11.8, the difference between the straight-based and curved-based EALFs under steady-rolling SHLs ranges from 16 percent to 68 percent, depending on the type of roadway facility and the evaluated axle groups. Such substantial differences between the evaluated case scenarios indicated that overlooking the influence of geometric design of roadways in curved sections results in underestimation of damages imparted by super heavy loads.

Table 11.8: Percent Increase of EALF Values due to Roadway Curvature

<table>
<thead>
<tr>
<th>Roadway Type</th>
<th>Percent Increase under Steady Rolling SHL Axles</th>
<th>Percent Increase under Decelerating SHL Axles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Percent</td>
<td>Single</td>
</tr>
<tr>
<td>FM</td>
<td>32%</td>
<td>68%</td>
</tr>
<tr>
<td>SH</td>
<td>28%</td>
<td>62%</td>
</tr>
<tr>
<td>US</td>
<td>22%</td>
<td>53%</td>
</tr>
</tbody>
</table>

The observed deviation was more pronounced under decelerating scenarios. As evidenced in Table 11.8, inclusion of deceleration condition of SHLs during turning movements at roadway curved segments can substantially increase the pavement damage factors by 26% to 118%, as compared with the EALF values for the same SHL operating at steady rolling condition over straight segment. Consequently, it deems necessary to account for the synergistic effects of
roadway geometric characteristics and vehicle loading conditions for realistic assessment of damages accumulated in pavement structures in overload corridors.

11.7.4.4. **Pavement Life Reduction Imparted by SHLs at Roadway Curve Segment**

Figure 11.33 shows the analysis of the results associated with the mechanistic quantification of the loss of pavement service life at roadway curve segment of SH 123. The plot shows the progression of the distresses, i.e., rutting and fatigue cracking, over a 20-year design period for SH 123, associated with different traffic input scenarios. Based on the internal distress calculation models in mechanistic analysis and steady-rolling based damage factors, it takes 115 months for SH123 to develop 0.5 in. of rut depth. However, if the SHLs were removed from the axle load spectra in the traffic inputs, it takes 215 months to develop 0.5 in. of rut depth. In other terms, operation of the SHL vehicles under steady-rolling state in SH 123 has consumed (215-115 = 100) months of the rutting service life of this pavement section at the studied curved segment.

Such detrimental impact was more pronounced with consideration of the acceleration/deceleration scenarios. Based on the results, operation of SHLs with occasional acceleration/deceleration patterns in SH123 resulted in 118 months reduction in its pavement service life. Consequently, the analysis of the loss of pavement service life for SH123, based on rutting criteria, indicated that operation of SHL vehicles at curve segments can impart significant PLRs as high as 47%, and 55%, under stead-rolling, and occasional accelerating/decelerating (braking) scenarios, respectively. It is worth noting that regular movement of SHLs under steady-rolling state, and occasional acceleration/deceleration of such SHLs in SH123 can potentially consume 10%, and 15%, respectively, of its pavement service life at curve segments, based on fatigue criteria.
11.7.4.5. **PLR Imparted at Different Roadway Segments and Under Different Vehicle Loading Scenarios**

The research team investigated different case scenarios associated with the type of roadway segment and vehicle loading scenarios to highlight the influence of roadway geometric design and vehicle speed variations on the pavement life consumptions due to the SHL vehicle movements.

Figure 11.33: Pavement Life Reduction at Curved Segment of SH123 due to SHL Operations, based on: (a) Rutting, and (b) Fatigue cracking.

**Figure 11.33:** Pavement Life Reduction at Curved Segment of SH123 due to SHL Operations, based on: (a) Rutting, and (b) Fatigue cracking.
To characterize the loss of pavement service life in this subtask, the authors considered various roadway segments, i.e., straight and curve segment, and incorporated different vehicle loading scenarios, i.e., steady-rolling state, and occasional accelerating/decelerating condition, into the devised mechanistic framework for further comparisons. The results attributed to the loss of pavement service life for various roadway segments and under different vehicle loading conditions are contrasted with each other in Figure 11.34. Furthermore, three types of roadway facilities, i.e., FM 468, SH 123, and US 281, with large volumes of heavy traffic loads were considered in the parametric study provided in this subtask of the study.

Based on the numerical simulation results provided in Figure 11.34, operation of SHL vehicles at curved segments of the studied roadways can impose substantially higher level of PLR, compared to the same SHLs typically operating along the straight paths without vehicle maneuvering actions. Therefore, damage quantification and pavement life prediction protocols should be capable of accounting for the excessive distress accumulation in pavement structures when transition of the SHL vehicle from straight sections to curve sections with different geometric features occur.

The results also revealed that acceleration/deceleration of passing SHL vehicles significantly contributed to the pavement life consumption predicted for all studied roadways. Particularly, curved segments were found to be highly sensitive to the sudden changes in the SHL vehicle speed, as the numerical permutations with consideration of SHL acceleration/deceleration showed higher PLR at curved segments, compared with the corresponding PLR calculated at straight segments. This observation is primarily attributed to the development of more taxing contact stresses within tire-pavement interface, when turning movements along the horizontal curves coincide with speeding/braking scenarios. In other terms, the increase in the transverse
(shear) contact stresses under turning tires due to the centrifugal effects, coupled with the added horizontal and tangential forces under speeding/braking scenarios due to the frictional interactions, essentially translate into elevated tire-pavement contact stresses under acceleration/deceleration of SHLs when move along the curved segments.

![Figure 11.34: Comparative PLR Results at Different Roadway Segments and under Various Loading Scenarios.](image)

Accordingly, the worst-case scenario among the evaluated cases pertains to the operation of SHL vehicles at roadway curve segments with occasional acceleration/deceleration patterns. As evidenced in Figure 11.34, operation of SHLs at straight segments of FM 468, SH 123, and US 281 under steady-rolling state can potentially consume 55%, 33%, and 25%, respectively, of the service life of their pavement sections. However, occasional acceleration/deceleration of such vehicles at curved segments can result in PLRs as high as 73%, 55%, and 35% for FM 468, SH 123, and US 281, respectively. Therefore, pavement structures servicing overload corridors, particularly at curved segments, intersections, junctions, or highway interchanges with significant vehicle acceleration/deceleration actions, can potentially experience excessive distresses due to
the absence of a protocol to account for the additional forces exerted at roadway curve segments during turning movements, and/or accelerating/decelerating scenarios in the pavement design process.

Another noteworthy observation from Figure 11.34 pertains to the type of roadway facility and its role on PLR studies. Based on the sensitivity analyses performed in this subtask of the study, in all evaluated case scenarios, the simulated pavement section of FM 468 with less robust structure experienced greater loss of service life under SHL vehicle movements, compared to the pavement sections in SH, and particularly US highways, with higher structural capacity. This underscores the significance of the pavement profile on the analysis of loss of service life in OW highway networks.

The results provided in this section highlighted the importance of unique characteristics of the roadway geometry and vehicle loading scenarios in PLR studies. Therefore, relying on simplifying assumptions that tend to overlook the influence of roadway geometric characteristics at curved segments and acceleration/deceleration impacts at highway intersections can potentially result in premature failure of the pavement facilities due to the underestimation of the damages imparted by super heavy vehicles. Consequently, realistic simulation of these components is the key step for accurate assessment of distresses accumulation in pavement facilities in overload corridors.

11.7.4.6. Influence of Geometric Design Parameters on PLR Imparted at Curve Segments

The authors conducted a series of numerical simulations to investigate the influence of roadway geometric features on the loss of pavement service life imparted at curve segments due to SHL vehicle movements. Different super-elevation rates, ranging all the way from 2% to 12%, and various curve radius that varied between 1,200 ft and 3,200 ft, were incorporated into the devised
sensitivity analysis to cover the most prevalent range of design parameters in the surveyed highway network. Figure 11.35 demonstrates the loss of pavement service life imparted by SHL operations at curved segments associated with different design parameters for super-elevation and curve radius. Three representative sites, i.e., FM 468, SH 123, and US 281, with large volumes of heavy traffic loads were also considered in the sensitivity analysis provided in this subtask of the study.

Based on the sensitivity results provided in Figure 11.35(a), increasing the super-elevation rate results in considerable increase in the loss of pavement service life imparted on pavement facilities subjected to SHL vehicles. The plausible reason behind such observation is more taxing vertical tire-pavement contact stresses, and therefore expedited damage accumulation, when the SHL units move along the horizontal curves with higher super-elevation rates. Furthermore, as evidenced in Figure 11.35(b), the imparted loss of pavement service life was more substantial at curve sections with smaller curve radii. This is primarily attributed to the centrifugal effects and the stimulated frictional interactions, which result in higher shear forces exerted at the tire-pavement contact zone during turning movements at sharp curves. This in turn accelerates shearing of the pavement surface, and hence, can result in significant pavement life reduction at sharp-curve segments subjected to SHL units.

The post-processed results underscored the significance of geometric design features of curved segments and their contribution to distress accumulation under SHL vehicle movements. The sensitivity analysis performed in this subtask also showed that the detrimental impact of turning SHL vehicles is more pronounced at sharp horizontal curves with low to intermediate radius of curvature and with high super-elevation rate. Hence, operation of SHL units at such roadway segments can potentially jeopardize the longevity of pavement sections if the influence of roadway geometry is overlooked during the pavement analysis procedure. Consequently,
The analysis of the SHL structural impacts should properly account for the site-specific geometric characteristics of roadways for accurate assessment of the damages and loss of pavement service life imparted by heavy traffic operations. The analysis results can be instrumental in mitigating the detrimental impacts of SHL vehicles on highway infrastructure, and provide means to optimize roadway geometric designs.

Figure 11.35: PLR Imparted at Roadway Curved Segments associated with Different Geometric Design Parameters: (a) Super-elevation, and (b) Curve Radius.
11.7.4.7. Influence of Wheel Load on PLR Imparted at Roadway Curve Segments

Figure 11.36 demonstrates the variation of the PLR imparted at roadway curve segments with incremental increases in the wheel load magnitudes ranging all the way from 6 kips to 12 kips. The results are reported as PLR imparted by SHL vehicle operations during the 20-year design life of the pavements corresponding to the heaviest wheel load incorporated into the loss of service life analysis protocol. The ascending trend of the PLR values in Figure 11.36 highlights the influence of wheel load magnitude and its contribution to the imparted loss of pavements service life at curve segments. Based on the sensitivity analysis performed in this subtask, inclusion of the heavier wheel loads in the analysis resulted in substantial increase in the PLR levels. This is as expected, since operations of the vehicles with heavier wheel loads can impart higher damages on pavements, and therefore, more substantially consume the pavement service life. The results also revealed that in all evaluated tire weights, FM 468 roadway with less robust pavement structure had the highest PLR, followed by SH 123, and eventually US 281 highway with higher structural capacity.

![Figure 11.36: Influence of Wheel Load on PLR at Roadway Curve Segments.](image)
As evidenced in Figure 1.36, operation of SHL vehicles with wheel loads equal to or below 6 kips, leads to reduction of 18%, 12%, and 6% of the pavement service life of curve sections in FM 468, SH 123, and US 281 roadways, respectively. The loss of pavement service life was more substantial for permutations with heavier wheel loads than 6 kips. The numerical simulation results showed that inclusion of the heavy wheel loads up to 12 kips into the analysis framework, resulted in significant PLRs as high as 65%, 47%, and 32% due to SHL vehicle movements in the studied curved segments of FM 468, SH 123, and US 281 roadways, respectively. Such alarming service life reduction can potentially result in premature failure of pavement facilities at roadway curved segments subjected to frequent passages of heavy wheel loads. Consequently, damage assessment and service life analysis protocols should properly account for the heaviest wheel load of SHL-vehicles and its contribution to distress accumulation in overload corridors.

11.7.4.8. Synthesis of Results: PLR Imparted at Curve Segments at Different Roadway Types

The authors synthesized the relevant information obtain from the series of numerical simulations for all ten representative sites in this study to provide better insights on the loss of pavement service life imparted at roadway curved segments due to SHL vehicle operations. The results were then post-processed and clustered for roadways with similar pavement structural properties to more realistically represent the results achieved by PLR studies associated with different roadway types. It should be also noted that a relatively low curve radius equal to 1,200 ft, as well as occasional acceleration/deceleration patterns, i.e., 25% accelerating/decelerating SHLs, were included in the analysis to consider more critical loading scenarios under turning SHL vehicles.

Figure 11.37 demonstrates the 3D color-coded plots developed in this study to represent the loss of pavement service life imparted at curved segments due to SHL vehicle movements,
considering the synergistic influence of integral components such as wheel load, super-elevation, and type of roadway facilities. The areas highlighted in warm colors (red and yellow) indicate substantially higher level of PLR, in contrast to the regions highlighted in cold colors (blue and light blue) with lower PLRs. The developed contour plots shown in Figure 1.37 clarified the significance of wheel load magnitude and super-elevation in the imparted PLR. As evidenced in the plots, operation of SHL vehicles with heavy wheel loads is highly detrimental to the longevity of pavement facilities, particularly at horizontal curve sections with high super-elevation rate. Such detrimental impact was more pronounced in less robust pavement structures of FM roadways, compared to the SH and US highways with higher pavement structural capacity.

The 3D contour plots presented in this chapter can be instrumental for pavement design practitioners in having a mechanistic means for characterization of the loss of pavement service life imparted at curved segments by SHL vehicles, with consideration of the vehicles wheel load, roadway geometric characteristics of horizontal curves, and unique features of roadway facilities. The synthesized plots can also guide state DOTs during the SHL evaluation process when the proposed route includes curve segments.
Figure 11.37: Contour Plots Showing the Synergistic Influence of Wheel Load and Super-elevation Rate on PLR Imparted at Curve Segments due to SHL Vehicle Movements for: (a) FM Roadway, (b) State Highway, and (c) US Highway.
11.7.5. Summary of the Major Points

This subtask of the study revolved around the mechanistic characterization of the loss of pavement service life imparted at roadway curved segments due to SHL vehicle operations. The proposed methodology accounts for the site-specific characteristics of roadway geometric design at horizontal curve sections, demanding loading conditions induced by SHLs, realistic tire-pavement interactions, unique features of transportation systems, and the environmental factors for accurate assessment of the pavement service life in overload corridors. Moreover, a comprehensive parametric analysis was further devised to investigate the synergistic influence of several influential factors, including geometric design parameters such as super-elevation and curve radius, wheel load magnitude, SHL acceleration/deceleration patterns, and type of roadway facilities, on the damage and loss of pavement service life imparted under SHL applications. Mechanistic analyses were performed and the post-processed results associated with representative FM, SH, and US highways affected by SHLs were provided. The results were then synthesized and clustered for roadways with similar pavement structural capacities to more realistically represent the results obtained from PLR studies. The major findings of this study summarized as:

- The results underlined the importance of type of roadway segments with various geometric design characteristics for realistic consideration of tire-pavement contact stress distributions under SHL vehicle movements. The 3D numerical simulation models showed that the maximum shear and vertical stresses imparted at the tire-pavement interface under a 15-kips SHL tire moving at horizontal curve section with curve radius of 1200 ft. and super-elevation of 8%, were 46%, and 34%, respectively, higher compared to the corresponding stresses calculated at straight segments. Consequently, realistic simulation of roadway geometric features and the resulting tire-pavement contact stresses is the
precursor for reliable assessment of the cumulative damages imparted on pavements in overload corridors.

- Movement of SHLs at roadway curve segments can impart substantially higher level of damages on the pavement facilities, as compared with the same vehicles operating at straight paths. This is primarily attributed to the fact that when a SHL vehicle travels along a horizontal curve, the resulting vertical and shear tire-pavement contact stresses significantly increase due to the localized stress concertation under turning/cornering tires, coupled with the additional frictional stresses developed at the tire-pavement interface that tend to counterbalance the centrifugal forces acting on SHL vehicle. Therefore, damage factors should also manifest such sensitivity to roadway curvature for accurate assessment of distresses at horizontal curves. Otherwise, the unaccounted for damages imparted by SHL vehicles during turning movements at curved segments can potentially jeopardize the longevity of pavement facilities in overload corridors.

- Overlooking the influence of geometric design of roadways at curved sections, resulted in underestimation of damage factors by 16% to 68%, depending on the type of roadway facility and the loading conditions imposed by super heavy vehicles. Accordingly, the underestimation of the damages due to the absence of a protocol to account for the roadway geometric features can potentially incur systematic errors for the design and life-cycle cost analysis of pavement sections servicing overload corridors. Therefore, realistic damage factors tailored towards the specific geometric design of roadway segments, rather than single value damage factor for the entire sections along the roadway, can better represent the detrimental influence of SHL vehicles in highway networks.
For the representative roadways evaluated in this study, operation of SHL vehicles at curved segments during the 20-year design life resulted in significantly higher pavement life consumption by 18% to 42%, compared to the PLR imparted by the same SHLs typically operating along the straight paths without vehicle turning/maneuvering actions. Consequently, damage quantification and PLR mechanisms should be capable of accounting for the excessive distress accumulation in pavement structures when transition of the SHL vehicle from straight sections to curve sections with different geometric features occurs.

The detrimental effect of SHLs was more manifested in the bend segments, when synergistic influence of roadway curvature and the changes in the SHL vehicle speed during acceleration/deceleration actions was included in the devised analysis algorithm. This is primarily attributed to the development of more critical contact stress patterns at tire-pavement interface, when turning movements along the horizontal curves coincide with speeding/braking scenarios. Therefore, pavement structures servicing overload corridors, particularly at horizontal curve segments, intersections, junctions, or highway interchanges with significant vehicle acceleration/deceleration frequencies, can potentially experience excessive distresses due to the absence of a procedure to account for the additional forces exerted at pavement surface during turning movements and under accelerating/decelerating scenarios, in the pavement design process.

A parametric analysis of major geometric design parameters underscored the significance of super-elevation and curve radius for accurate assessment of the SHL structural impacts on pavement facilities. The results indicated that the damages and loss of pavement life imparted by turning SHL vehicles is more substantial at sharp horizontal curves with low
to intermediate radius of curvature and with high super-elevation rate. The plausible reasons behind such observation pertain to the centrifugal effects and the stimulated frictional interactions during turning movements at sharp-curved segments, as well as more pronounced vertical stress concentration under SHL tires traversing the horizontal curves with high super-elevation rates. Such taxing tire-pavement contact stresses is synonymous with development of higher shear and vertical forces exerted at the tire-footprint zone. This in turn translates into higher shearing/surface rutting potential, and hence, can result in substantial PLR when the SHL units move along sharp-curved sections with high super-elevation rate.

- Another noteworthy observation from this study pertains to the influence of wheel load on PLR imparted by SHL vehicles at curve sections. Incorporation of different wheel load magnitudes into the devised analysis framework showed that increasing the maximum wheel load of the passing vehicles from 6 kips to 12 kips significantly increased the PLR level from “6%-18%” to “32%-65%” range, depending on the traffic makeup and pavement structural properties attributed to different representative roadways. Such alarming service life reduction can potentially result in premature failure of pavement facilities at curve segments subjected to frequent passages of heavy wheel loads, if the influence of the heaviest wheel load of SHL-vehicles and its contribution to distress accumulation is overlooked during the pavement design and analysis procedures.

- The numerical simulation results substantiated the fact that relying on simplifying assumptions that tend to overlook the influence of roadway geometric characteristics can potentially result in expedited deterioration of pavement facilities at curved segments due to the underestimation of the damages imparted by SHL vehicles. Therefore, analysis of
roadway geometry should be an integral component in risk management studies of pavement facilities servicing the overload corridors.

- The synthesized results provided in this subchapter can be instrumental in mitigating the detrimental impacts of SHL vehicles on highway infrastructure during turning movements at curve segments, and provide means to optimize roadway geometric designs in overload corridors. The presented 3D contour plots can also guide state DOTs during the SHL evaluation process when the proposed route includes curve segments.
11.8. Stability Analysis of the Sloped Pavement Shoulders

11.8.1. Introduction

SHL vehicles typically carry heavy loads that exceed the permissible truck weight/size limits set forth by state highway agencies. In Texas, vehicles with GVW exceeding 254,300 lb., or vehicles that exceed the maximum permissible weight on any axle group, are considered as SHLs. In terms of the vehicle size-related attributes, the majority of SHL vehicles consist of specialized hauling units with multi-axle and multi-wheel trailers that are essentially regarded as oversize vehicles. Operation of these non-conventional vehicles with heavy tires and complex axle arrangements has been a major contributor to the slope failure for roadway shoulders in the overload networks.

To minimize such failure risk, it is recommended that SHL vehicle keep moving with an appropriate offset from the pavement edge and roadway shoulder. However, in routes with narrow traffic lanes, it is not always feasible to maintain wide SHL vehicles far from the pavement edge. Figure 11.38 illustrates an example of a trailer-mounted SHL unit that was allowed to operate in a specified route in Channelview, Texas, in October 2019. The demonstrated SHL unit has GVW exceeding 1.8 million lb., and comprised of 45 axles with “8-tires per axle” arrangement. As indicated in Figure 11.38, the trailer width was equal to 22 ft, which is nearly twice the standard lane width (10 ft ~ 12 ft). Hence, it is entirely feasible for such SHL trailer to move along the edge of travel lanes or even enter the roadway shoulders. This clarifies the need to consider size-related attributes of SHLs, as well as travel-lane/shoulder widths, during SHL evaluation procedure to maintain the stability of sloped shoulders (Morovatdar et al., 2022b).
Review of roadway design plans, coupled with our research team’s visual survey in the relevant projects, revealed that the majority of rural and state highways in Texas lack appropriate lane width to accommodate specialized trailer units with great width. Thus, passages of such wide trailers with multi-axle and multi-wheel configurations can potentially jeopardize the stability of pavement shoulders. This essentially translates into the accelerated damage of the pavement structures, which in turn causes safety concerns for the traveling public. Consequently, it is imperative to properly assess the potential risks against failure of sloped shoulders subjected to SHL vehicles, considering the non-conventional axle assembly of SHLs, and unique characteristics of pavement shoulders in the network. The results can provide insights on the risk management studies of pavement facilities in overload corridors.

Since the 1990s, several researchers studied different methodologies to evaluate the detrimental impacts of heavy vehicles on pavement facilities (Gonzalez et al., 2021, Hajj et al. 2018, Chen et al. 2013, Dong and Huang 2013, Oh et al. 2011, Chatti el al. 2009, and Fernando
However, there are a few studies in the literature associated with the stability analysis of pavement shoulders subjected to SHL vehicles. In a relevant study, Hajj et al. (2018) developed an approach to investigate the stability of pavement shoulders under SHL movements. The researchers utilized the wedge method to calculate the Factor of Safety (FoS) against slope-shoulder failure. In the aforementioned study, the horizontal stresses obtained from 3D-Move software were multiplied by a stress adjustment factor to estimate resultant horizontal forces induced on the sloping shoulder due to SHL vehicle movement. Several researchers developed various approaches for stability analysis of soil slopes and embankments under typical loading scenarios (Su et al. 2021, Ma et al. 2021, Liu et al. 2017, Cho 2010, and Griffiths and Fenton 2004). Based on the numerical simulations, the researchers favored probabilistic approaches over traditional deterministic approaches for slope stability assessments.

The majority of the previous studies for slope stability analysis either rely on simplifying assumptions in the numerical simulation phase, or are not designed to handle the demanding loading conditions imposed by non-conventional SHL vehicles with complex axle arrangements. Additionally, the majority of the preceding analysis approaches deploy the traditional Limit Equilibrium Method (LEM) that overlook the inherent variability of the soil strength parameters. This can potentially jeopardize the accuracy and reliability of the slope stability analyses.

The highlighted issues were the motivations for the authors to develop a probabilistic methodology to assess the stability of sloped pavement shoulders under SHL vehicle operations. Advanced numerical techniques, in combination with Monte Carlo Simulation (MCS) method, were deployed to upgrade the traditional LEM to a probabilistic methodology that accounts for the uncertainties associated with shear strength properties of pavement unbound layers. The devised approach also accounts for the demanding loading conditions attributed to wide SHLs with multi-
axle trailers, dynamic nature of moving vehicles, as well as unique features of pavement shoulders and highway facilities in the network for realistic assessment of stability of sloped shoulders.

11.8.2. Proposed Methodology

Figure 11.39 shows the flowchart of the proposed approach for probabilistic assessment of the stability of pavement shoulders subjected to SHL vehicles. As demonstrated in the figure, the proposed procedure consisted of three main segments, including: (1) field testing, (2) calculation of SHL-induced horizontal forces using 3D FE modeling, and (3) determination of FoS using MCS technique. The site-specific information on pavement layer properties and SHL vehicle characteristics were determined from extensive field data collection efforts using NDT equipment, as well as P-WIM deployments, in ten representative sites. The roadway design plans were further instrumental in obtaining the relevant information on features and characteristics of pavement shoulders such as side slope, shoulder width, shoulder type, and the material properties. Additionally, the authors conducted extensive data mining on the permit records and SHL vehicle plans to complement the information on the axle and tire characteristics of the SHLs operating in Texas overload corridors. GVW, axle weight, axle configuration, wheel load, and tire pressure, were the most relevant traffic load parameters included in the devised analysis framework.
Figure 11.39: Flowchart for the proposed probabilistic approach for stability analysis of the sloped pavement shoulders subjected to SHL vehicles.

The field-derived information on pavement layers properties, shoulder characteristics, and SHL-vehicle loading conditions, were in turn incorporated into a 3D FE system for determination of the horizontal forces applied to the pavement shoulder due to SHL vehicle movement. Figure 11.40 provides a schematic view of a typical pavement structure with sloped shoulder subjected to SHL vehicle. The figure also shows the potential failure surfaces along the shoulder, as well as different forces acting on the failure block, including SHL-induced horizontal force ($F_{SHL}$), lateral earth pressure from adjacent soil ($F_{earth}$), block’s weight (W), and resisting shear force along the potential failure surface ($F_R$). The Mohr-Coulomb shear strength parameters of base and subgrade layers, as primary components in the analysis, were then randomly generated to account for their
inherent uncertainties. Afterwards, lateral earth pressure from the adjacent subgrade soil induced on the failure block using the Rankine theory, as well as block’s weight, were calculated.

![Diagram of typical sloped pavement shoulder](image)

**Figure 11.40:** (a) Typical sloped pavement shoulder subjected to SHL vehicle with potential failure surfaces, and (b) Sketch showing forces acting on the failure block.

The last stage of the analysis pertains to the characterization of FoS against shear failure of the sloped shoulders. To assess FoS in this subtask, initially, the force equilibrium equations in the parallel and perpendicular directions to the slip surface were applied. The resisting shear forces \(F_R\) were then compared with the sliding forces \(F_S\) along the trial slip surfaces to determine the FoS, using Equations 1 through 3. Ultimately, the MCS technique was used to determine the distribution of FoS based on the generated random variables, i.e., shear strength parameters of pavement unbound layers. The same procedure was followed to evaluate various slip surfaces at different points of interest along the sloped shoulder to identify the most critical slip surface, imposing the minimum average FoS.

\[
F_R = (W \cos \alpha - F_{SHL} \sin \alpha - F_{earth} \sin \alpha) \tan \varphi + c_L \tag{1}
\]

\[
F_S = W \sin \alpha + F_{SHL} \cos \alpha + F_{earth} \cos \alpha \tag{2}
\]

\[
FoS = \frac{F_R}{F_S} = \frac{(W \cos \alpha - (F_{SHL} + F_{earth}) \sin \alpha) \tan \varphi + c_L}{(W \sin \alpha + (F_{SHL} + F_{earth}) \cos \alpha)} \tag{3}
\]
where:

- \( F_s \) = sliding force along the potential failure surface
- \( F_r \) = resisting shear force along the potential failure surface
- \( W \) = sliding block’s weight
- \( F_{SHL} \) = horizontal driving force due to the SHL vehicle
- \( F_{earth} \) = lateral earth pressure from the adjacent soil
- \( \alpha \) = angle between the slip surface and horizontal surface
- \( L \) = length of the slip surface
- \( c \) and \( \varphi \) = Mohr-Coulomb cohesion and angle of internal friction, respectively.

11.8.2.2. Site-Specific Shoulder Characteristics

The authors conducted data mining on available databases and roadway design plans to extract the required information on shoulder characteristics such as type, width, and side slope of the pavement shoulders. Reviewing the available resources, coupled with visual inspection surveys in the network, indicated that roadway sections evaluated in this study consisted of both paved and unpaved shoulders with a side slope ranging all the way from 1:2 (V: H) to 1:6 (V: H), while the shoulder width range between 2 ft and 10 ft. Table 11.9 provides a summary of the site-specific shoulder characteristics associated with ten representative sites in Texas overload corridors. This information serves as basis for the comprehensive sensitivity analysis devised in this study.
### Table 11.9: Shoulder Characteristics of Ten Representative Roadways in the Project

<table>
<thead>
<tr>
<th>District</th>
<th>County</th>
<th>Roadway</th>
<th>Side Slope (V:H)</th>
<th>Shoulder Width (ft)</th>
<th>Shoulder Type (Paved/Unpaved)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laredo</td>
<td>Dimmit</td>
<td>US83</td>
<td>1:4</td>
<td>4-8</td>
<td>Paved with thinner AC layer</td>
</tr>
<tr>
<td></td>
<td>La Salle</td>
<td>FM468</td>
<td>1:4</td>
<td>2-3</td>
<td>Unpaved</td>
</tr>
<tr>
<td>San Antonio</td>
<td>McMullen</td>
<td>FM624</td>
<td>1:3 – 1:4</td>
<td>3</td>
<td>Unpaved</td>
</tr>
<tr>
<td></td>
<td>McMullen</td>
<td>FM99</td>
<td>1:4</td>
<td>6</td>
<td>Paved with thinner asphalt layer</td>
</tr>
<tr>
<td></td>
<td>Atascosa</td>
<td>SH16</td>
<td>1:3 – 1:6</td>
<td>2-8</td>
<td>Paved</td>
</tr>
<tr>
<td>Corpus Christi</td>
<td>Live Oak</td>
<td>US281</td>
<td>1:6</td>
<td>4-10</td>
<td>Paved</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>SH72</td>
<td>1:4 – 1:6</td>
<td>2-4</td>
<td>Paved</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>SH123</td>
<td>1:2 - 1:6</td>
<td>8</td>
<td>Paved with thinner AC layer</td>
</tr>
<tr>
<td>Yoakum</td>
<td>Gonzales</td>
<td>US183</td>
<td>1:6</td>
<td>10</td>
<td>Paved</td>
</tr>
<tr>
<td></td>
<td>Dewitt</td>
<td>SH119</td>
<td>1:4 – 1:6</td>
<td>2-6</td>
<td>Paved</td>
</tr>
</tbody>
</table>

#### 11.8.2.3. Numerical Simulation Modeling

Realistic simulation of moving SHL vehicles and their impact on pavement shoulders involves several aspects of complexity in terms of the analysis approach, loading conditions, vehicle speed incorporation, and tire-pavement interactions. In this study, ABAQUS FE program was used for the determination of the horizontal driving forces applied to the pavement shoulders under SHL vehicle movement, with considerations of the complex loading conditions, SHL-vehicle dynamics, and realistic tire-pavement interactions.

Figure 11.41 provides an example of the simulated SHL nucleus, as well as the sloped pavement shoulder, for calculation of the SHL-induced horizontal forces using 3D finite element modeling. The representative set of 4×4 tires shown in Figure 11.41 pertains to the most critical SHL case, i.e., 28A-8T with demanding axle assembly and tire arrangement, which was selected as the SHL case study in this analysis. Other pertinent information on the numerical simulation
phase was modeling of the most outer SHL tire with no offset from the shoulder edge to consider the potential failure risks when the SHL vehicle moves along the shoulder edges. It should be also noted that relatively low vehicle speed ($v = 20$ mph) was assigned to the explicitly modeled tire elements to account for the slow-moving nature of SHLs in this study. Detailed information on FE modeling, tire element characteristics, boundary conditions, tire-pavement contact stresses, and frictional interactions, are provided in Chapter 8.

![Figure 11.41: Simulation of sloped pavement shoulder subjected to SHL, using 3D FE models.](image)

### 11.8.2.4. Monte-Carlo Simulation (MCS)

The MCS is a technique commonly deployed in probabilistic modeling approaches. In a MCS, discrete values of the input random variables are generated according to their probability distribution functions, and the corresponding output parameters are calculated at each simulation step (Cho, 2010). Despite the fact that this technique requires excessive computational efforts, the mathematical formulation of the MCS is relatively simple and the method has the capability of handling complex problems. Therefore, MCS technique in combination with modified LEM, can be instrumental in probabilistic assessment of stability of sloped shoulders in geotechnical practice.

In this study, MCS technique was employed to obtain FoS against pavement shoulders failure as the analysis output, based on the randomly sampled shear strength parameters of the pavement unbound layers, as stochastic inputs. The shear strength parameters were assumed to be independent and characterized statistically by a log-normal distribution defined by a mean ($\mu$), and
a standard deviation ($\sigma$), as shown in Figure 11.42. For the subgrade layer, the mean and standard deviation of the Mohr-Coulomb cohesion ($c$) were defined as $\mu_c = 1.5$ psi, and $\sigma_c = 0.4$ psi, while the layer’s angle of internal friction ($\phi$) was characterized by $\mu_\phi = 30$ degrees, and $\sigma_\phi = 6$ degrees. The described log-normal distribution parameters ($\mu$, and $\sigma$) were selected based on the relevant research studies conducted by Cho (2010), and Griffiths and Fenton (2004). Additionally, the same mean and standard deviation values were defined for the base layer’s friction angle, while the cohesion of the unbound aggregate base layer was assumed to be zero (Zeng et al., 2016).

The authors further developed a MATLAB® program to efficiently deploy the MCS technique in this study. Parallel processing was also used to optimize the simulation runtime. A reasonably large number of simulations scenarios, i.e., 5,000 simulation repetitions, was considered to achieve sufficient output data points. In each MCS repetition, the FoS value was calculated for each generated set of $c$ and $\phi$. Ultimately, the corresponding values of FoS associated with 5,000 permutations were then translated into histograms and distribution diagrams to more realistically represent the probabilistic results on FoS measures. It worth noting that the
*fmincon* optimization function offered in MATLAB® was also deployed to systematically identify the most critical failure surface along the sloped shoulder, imposing the minimum average FoS.

### 11.8.2.5. Calculation of the Probability of Failure \((P_f)\)

The MCS method is not merely limited to calculating the factor of safety. Various statistical parameters can be calculated after the simulation process, such as probability of failure, and coefficient of variance. Essentially, probability of failure can provide insights on the stability studies, and provides means to have a better understanding of the stability of a given slope. For this reason, the authors aimed to calculate the probability of failure, based on the FoS values obtained from the probabilistic analysis performed in this study. As described in Zhang et al 2017, in the case that FoS follows the log-normal distribution, the reliability index \((\beta_L)\), and probability of failure \((P_f)\) can be calculated from Equations 4, and 5, respectively, as:

\[
\beta_L = \frac{\ln\left(\frac{FoS_{ave}}{\sqrt{1+COV^2}}\right)}{\sqrt{\ln(1+COV^2)}} \tag{4}
\]

\[
P_f = 460\exp\left(-4.3\beta_L\right) \tag{5}
\]

where, \(\beta_L\) is reliability index, \(FoS_{ave}\) is the average value of the FoS, \(COV\) is the coefficient of variance of the FoS, and \(P_f\) is the probability of failure.

### 11.8.3. Analysis of Results and Discussions

#### 11.8.3.1. Factor of Safety Distribution

This section provides the synthesized results pertaining to one of the heavily trafficked sections with narrow traffic lane and shoulder width, i.e., State Highway 16 in San Antonio, to showcase the analysis protocol developed in this study for probabilistic assessment of the stability of
shoulders subjected to SHLs with multi-axle trailer units. The studied roadway section in SH16 consisted of a paved shoulder with a shoulder width of 2 ft, and the side slope equal to 1V:3H.

Figure 11.43 shows the distributions of FoS against failure of the studied pavement shoulder in SH 16, associated with different load magnitudes on SHL vehicle tires, i.e., 6 kips, 9 kips, and 12 kips. Based on the probabilistic analysis performed in this subtask of the study, with inclusion of the wheel load as 9 kips in stability analysis algorithm, the FoS ranged from 0.41 to 2.18, while the mean, a representative measure of central tendency, was found to be 1.12, as shown in Figure 11.43(a). The figure also indicates that around 25% of simulated cases had a FoS lower than one, indicting failure condition. The characterized distribution pattern illustrated in Figure 11.43(a) properly captured the inherent variability of the shear strength parameters of the pavement layers and their impact on FoS measures. This underscored the significance of deploying probabilistic approaches, instead of merely relying on one single value in deterministic methods, for realistic assessment of the pavement shoulders stability. Therefore, overlooking the inherent uncertainties in the analysis of pavement shoulders with heavy traffic loads can potentially jeopardize the pavement shoulder design and reinforcement strategies in overload corridors.

Figure 11.43(b) shows the FoS distributions associated with different wheel loads, i.e., 6 kips, 9 kips, and 12 kips. According to the results provided in Figure 11.43(b), FoS distributions for three evaluated loads resemble a bell-shaped curve with positive skewness. However, as the load magnitude on tire increases, the distribution peaks tend to shift towards the left portion of the diagram, representing lower FoS values. As evidenced in Figure 11.43(b), incorporating the wheel load as 12 kips in the analysis resulted in the lowest mean value for FoS, i.e., 0.81, compared to other permutations with wheel load of 9 kips, and 6 kips, with higher FoS means as 1.12, and 1.28, respectively. Similarly, failed scenarios were found to be as high as 59%, for wheel load of 12
kips, while incorporation of 9 kips and 6 kips as wheel load into the stability analysis framework led to considerably lower failed cases as 25%, and 10%, respectively. Such observed trend is as expected, since heavier wheel loads can impose more taxing stress paths on the pavement structures, leading to an increase in the horizontal sliding forces \( F_{SHL} \) applied to sloped shoulders.

Figure 11.43: Factor of safety distribution for (a) Wheel load of 9 kips, and (b) Different wheel loads: 6 kips, 9 kips, and 12 kips.

11.8.3.2. Parametric Analysis of Stability of Sloped Pavement Shoulders

The authors conducted a series of parametric analyses to investigate the influence of major parameters such as wheel load magnitude, wheel offset from shoulder edge, shoulder slope, shoulder width, shoulder type, and moisture management capability of roadway infrastructure, on the stability of sloped shoulders under SHL vehicle movement. Figure 11.44 provides an overview of the analysis parameters, as well as the ranges/permutations, incorporated into the devised parametric study. As illustrated in Figure 11.44, wheel load magnitude ranged all the way from 6 kips to 12 kips, side slope ranged between 1:2 (V: H) and 1:6 (V: H), shoulder width varied between 2 ft and 10 ft, wheel offset from shoulder edge changed from zero (no offset) to 4 ft, while different shoulder types, i.e., paved and unpaved, with different moisture management capabilities,
namely, excellent and poor, were incorporated in the analysis. Review of the available resources, such as TxDOT databases and roadway design plans, coupled with our team’s field data collection, considering the unique features of roadway infrastructures, pavement shoulders, and traffic loading conditions attributed to ten representative sites, provided the rationale behind the ranges selected for each analysis parameter.

The parametric analysis results pertaining to two studied sites, i.e., SH 16 in San Antonio District and FM 468 in Laredo District, representing paved and unpaved shoulders, respectively, were further contrasted with each other for comparison purposes in this study. The FoS distributions obtained from the MCS technique were in turn converted to “probability of failure ($P_f$)” measures to facilitate the quantitative comparison of different case scenarios. The following sections provide the post-processed results and relevant discussions.

![Figure 11.44: Numerical Permutations Incorporated in the Parametric Analysis of the Stability of Sloped Pavement Shoulders.](image)

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11.8.3.3. Influence of SHL-Vehicle Wheel Load on Shoulder Probability of Failure

Figure 11.45 shows the probability of failure for various wheel load magnitudes to provide a broader insight on the sensitivity of sloped shoulders to loading magnitude. Shoulder width of 2 ft and shoulder slope equal to 1V:3H were defined as identical parameters in the sensitivity analysis performed in this subtask. The ascending nature of probability of failure values in Figure 11.45 clarifies the influence of wheel load magnitude on the stability of sloped shoulders. Based on the post-processed results provided in Figure 11.45, increasing the wheel load from 6 kips to 12 kips results in drastic increase in the probability of failure by nearly 5.9 times. Such substantial increase in the failure risk can potentially compromise the stability and structural integrity of pavement shoulders, considering the non-conventional loading conditions of SHLs with numerous axles and tires. Therefore, analysis of the wheel load magnitude should be an integral component in stability analysis of pavements servicing SHLs with multi-axle trailers in overload corridors.

Another noteworthy observation from Figure 11.45 pertains to the type of roadway shoulders and their sensitivity to the demanding loading conditions imposed by SHL vehicles. As evidenced in Figure 11.45, the studied unpaved shoulder was found to be more sensitive to SHL vehicle movement compared to the paved shoulder evaluated in this study, as in all wheel load magnitudes, unpaved shoulder had a higher failure probability than the other counterpart. Such intrinsic sensitivity of unpaved shoulder to SHL vehicles was more pronounced at numerical permutations with higher wheel load magnitudes. The highlighted difference between the two shoulder types is mainly attributed to the less structural and load dissipation capacity of unpaved shoulders, in comparison with paved shoulders with substantially higher structural capacity.

In addition, our visual inspection survey in ten representative sites showed that the majority of unpaved shoulders or shoulders paved with rather thin asphalt layers are constructed in rural
highway networks such as FM roadways, while paved shoulders are mainly observed in SH and US highway networks. Evidently, the studied pavement sections in FM roadways consisted of extremely thin asphalt treated surface layers, as opposed to the pavement profiles in SH and US highways with thicker layers and good quality materials. Pavement profile is an integral component in the analysis of SHL structural impacts, due to its role on stress dissipation behavior. Accordingly, unpaved shoulders with less structural capacity and less robust profile of their adjacent pavements are less efficient in dissipating the imposed stresses under taxing loading conditions, as compared to paved shoulder structures that dissipate the stresses in more efficient manner. This in turn translates into higher level of horizontal driving forces, and therefore, higher shear failure potential at unpaved shoulders under SHL vehicles. The inherent vulnerability of unpaved shoulders to demanding loading conditions can potentially jeopardize the stability of sloped shoulders when subjected to SHL applications with heavy wheel loads.

![Figure 11.45: Influence of wheel load on probability of failure of the pavement shoulder.](image)

Figure 11.45: Influence of wheel load on probability of failure of the pavement shoulder.
11.8.3.4. **Influence of Load Distance from Edge on Shoulder’s Probability of Failure**

Figure 11.46 shows the variations of probability of failure with changes in the lateral distance of outer tires from the shoulder edge. Vehicle wheel load of 10 kips, shoulder slope of 1V:3H, and shoulder width of 2 ft, were the identical parameters incorporated in the sensitivity analysis performed in this subtask. Based on the numerical simulation results provided in Figure 11.46, approaching the SHL vehicle to the edge of roadway shoulders significantly increases the risk against shear failure of sloped shoulders. Such increase in the failure possibility was more apparent for the unpaved shoulder with less structural capacity, as compared to the other counterpart. As evidenced in Figure 11.46, decreasing the relative distance between SHL tires and shoulder edge from 4 ft to zero (right on the edge) results in 12% (34%-22% = 12%) and 15% (41%-26% = 15%) increase in the probability of failure for the studied paved and unpaved shoulder, respectively. Consequently, it deems necessary to keep wide SHL vehicles with an appropriate offset from the shoulder edge to mitigate the detrimental impacts of SHLs on sloped shoulders.

![Figure 11.46: Influence of Location of SHL Vehicle on shoulder’s probability of failure.](image-url)
11.8.3.5. **Influence of Slope Inclination on Shoulder Probability of Failure**

Figure 11.47 shows the variations of probability of failure with changes in the slope inclination for various shoulder types. It is noted that the vehicle wheel load equal to 10 kips and shoulder width of 2 ft were incorporated in the numerical simulations, as constant parameters, in this subtask. Based on the sensitivity results provided in Figure 11.47, probability of failure significantly increases with increasing the slope inclination. As evidenced in the plot, increasing the side slope ratios from 1V:6H to 1V:2H results in 20% increase (41%-21% = 20%) in the probability of failure for the studied paved shoulder under SHL vehicle movement. Furthermore, the unpaved shoulder with less efficient stress dissipation features was found to be more sensitive to the slope inclination variations, as increasing the side slope within the same range and under the same SHL vehicle leads to 27% increase (52%-25% = 27%) in its failure probability. The higher failure risk, as observed for shoulders with higher side slopes, is primarily attributed to the lower length of slip surface ($L$), and therefore, lower shear resisting forces ($F_R$) developed along the slip surface, which is synonymous with their reduced stability levels. Consequently, roadway shoulders, particularly if unpaved, with steep slopes are highly sensitive to the demanding loading conditions imposed by SHL vehicles.
Figure 11.47: Influence of slope inclination on probability of failure of the pavement shoulder.

### 11.8.3.6. Influence of Shoulder Width on Probability of Failure

Figure 11.48 presents the results associated with the sensitivity of probability of failure to the shoulder width for various shoulder types. The results pertain to the vehicle wheel load of 10 kips and shoulder slope of 1V:3H, as identical parameters considered in this sensitivity analysis. Additionally, in the numerical simulation models, the outer SHL tires were located right at the edge of travel lane to account for the shoulder width variations and their impact on shoulder stability. According to the results provided in Figure 11.48, incorporating narrow shoulder widths into the slope stability analysis resulted in higher level of risk of failure under SHL vehicle movement. This is because when the SHL vehicle operates at roadways with narrow shoulders the vehicle is in close proximity to shoulder edges, and hence, it can impart more taxing stress paths at shoulder structures. This can potentially undermine the stability of the slopes and ultimately lead to an increase in the shoulder’s probability of failure.

The results also showed that the unpaved shoulder with less robust structure was more sensitive to the shoulder width reduction, compared with the studied paved shoulder. As evidenced
in Figure 11.48, decreasing the shoulder width from 10 ft to 2 ft results in 18% increase (28% - 10% = 18%) in the failure probability of the studied paved shoulder, while decreasing the shoulder width within the same range leads to 23 increase (34% - 11% = 23%) in the probability of failure for the unpaved shoulder evaluated in this study. Accordingly, operation of SHL units on pavements with narrow shoulder widths can potentially jeopardize the structural integrity and stability of roadway shoulders, specifically if unpaved.

![Figure 11.48: Influence of shoulder width on probability of failure of the pavement shoulder.](image)

11.8.3.7. **Influence of Moisture Management Capability on Shoulder Probability of Failure**

Moisture is primary climatic factor that play an important role in the structural integrity and stability of pavement shoulders. Evidently, the infiltration of water in shoulder structures due to heavy precipitation, surface water flow, or GWT fluctuation can adversely affect the orthogonal stiffness properties of pavement shoulders. This is mainly attributed to the stiffness softening behavior of unbound aggregate systems due to moisture intrusion effects. The changes in the stiffness properties of pavement shoulders is synonymous with the variation of the effective shear strength of sloped shoulders. Therefore, the stability of sloped pavement shoulders is not
monolithic under different environmental scenarios with various moisture intrusion levels. This emphasizes the need to consider the moisture level and saturation state of pavement shoulders during slope stability analyses.

It is often assumed that the roadway drainage systems are sufficient to ensure the protection of the base and subgrade soils, as well as the neighboring shoulders, from moisture intrusion. However, the capability of highway networks for efficiently draining the pavement foundations/shoulders, particularly under prolonged inundation conditions such as flooding events, is rather limited. One of the prime examples of such extreme moisture ingress conditions pertains to hurricane Harvey in Texas in 2017, in which segments of major highways were under water for nearly eight successive days. Exposing to such moisture conditions when coupled with poor moisture management performances, can substantially increase the risk of failure for roadway shoulders under heavy traffic loads. Accordingly, it deems necessary to investigate the influence of moisture management capability of roadway infrastructures, during demanding environmental scenarios such as flooding conditions and heavy rain events, on the stability of sloped pavement shoulders subjected to SHL vehicles.

This was the motivation for the authors to conduct another series of parametric analysis to account for the saturation state of pavement shoulders under different moisture management systems, and its impact on shoulders probability of failure under SHL applications. Accordingly, the following two case scenarios were investigated by incorporating different permutations for moisture management into the devised analysis framework:

- Scenario #1: Excellent moisture management capability, and
- Scenario #2: Poor moisture management capability.
In scenario #1, the pavement shoulder is dry and the GWT is assumed at the hydraulic equilibrium state due to efficient drainage system and excellent moisture management capability. The equations presented earlier in section 12.8.2. were used to calculate the FoS measures for scenario #1. However, for scenario #2, the relevant equations for FoS calculations were modified with consideration of the fully saturated pavement shoulders. In this scenario, the GWT is considered close to surface due to simultaneous effect of extreme moisture inundation and poor moisture management performance. Figure 11.49 illustrates the shoulder’s failure bolck at fully saturated state with GWT close to surface, as well as different forces acting on the failure block. As shown in Figure 11.49, the horizontal driving forces applied to the potential failure blocks due to pore water pressure should be included in stability analysis of sloped shoulders at saturated state.

![Figure 11.49: Sketch showing forces acting on the shoulder’s failure block at saturated state.](image)

In addition to pore water pressure, another required modification for calculation of FoS measures in scenario #2 pertains to proper incorporation of Mohr-Coulomb shear strength parameters at saturated condition. Based on the recommendations provided by U.S. Army Corps of Engineers (2003) and Cho (2010) for slope stability analysis, the Mohr-Coulomb angle of internal friction ($\phi_u$) for saturated subgrade soil under undrained condition was defined as zero; while the layer’s cohesion at saturated state ($c_u$) was characterized by $\mu_c = 3.3$ psi, and $\sigma_c = 1.0$ psi.
in the MCS algorithm. Additionally, for the base layer, the layer’s friction angle ($\varphi_u$) was characterized by $\mu_\varphi = 30$ degrees, and $\sigma_\varphi = 6$ degrees, while the cohesion of the unbound aggregate base layer was assumed to be zero (Zeng et al., 2016). Therefore, the FoS associated with the fully saturated pavement shoulder under poor moisture management systems can be calculated from Equation 4. Subsequently, the FoS distributions obtained from the MCS technique were then converted to “probability of failure ($P_f$)” measures to further comparison purposes.

\[
FoS = \frac{F_R}{F_S} = \frac{c_u L}{(W \sin \alpha + (F_{SHL}+F_{earth}+F_u \cos \alpha)}
\]  

(4)

The shoulder’s probability of failure associated with various moisture management capabilities are contrasted with each other for comparison purposes in Figure 1.50. The results pertain to the paved shoulder in SH 16 in San Antonio with shoulder slope of 1V:5H and shoulder width of 2 ft simulated under representative tires of SHL vehicle 28A-8T.

The post-processed results provided in Figure 1.50 clarified the influence of moisture management capability of roadway infrastructures on failure probability of shoulders under SHL vehicle operation. As evidenced in the plot, movement of the super heavy vehicle under flooding events with poor moisture management performance of the roadway can impose substantially higher failure risk for sloped shoulders, as compared to the potential risk against failure of shoulders with efficient drainage systems. This is in line with our expectations, because poor moisture management system of roadways under prolonged inundation conditions such as flooding or elevated GWT can dramatically accelerate the infiltration of water in shoulder structures. Hence, stiffness properties of shoulder structure adversely affected due to the softening behavior of unbound materials under extreme moisture intrusion conditions. This stiffness softening of unbound systems in turn translates into lower shear strength capacities, and therefore higher probability of failure for the roadway shoulders under SHL vehicles.
According to the sensitivity results presented in Figure 11.50, the elevated risk against failure of saturated shoulders under poor moisture management performance was more prominent in numerical permutations with heavier vehicle wheel loads. As evidenced in the plot, inclusion of 6 kips as wheel load into the slope stability analysis resulted in 11% and 9% as probability of failure under excellent and poor moisture management scenarios, respectively, indicating 2% difference in the calculated failure probability. However, incorporating wheel load of 12 kips in the analysis framework substantially increased the shoulder’s failure probability by 14% (52%-38%=14%) due to poor moisture management performance. Consequently, operation of SHL vehicles that carry heavy wheel loads can potentially jeopardize the stability of sloped shoulders under flooding conditions or heavy rain events, if the accommodating highway network lacks proper moisture management system for efficiently draining the pavement foundations and roadway shoulders.
11.8.3.8. Synthesis of Results on Probability of Failure of Pavement Shoulders under SHLs

The authors synthesized the relevant information obtained from the series of numerical simulations for all ten representative sites in this study to provide better insights on the probability of failure of pavement shoulders subject to SHL vehicle operations in overload corridors. The results were then post-processed and clustered for shoulders with similar structural properties and paving condition to more realistically represent the results achieved by slope stability analyses associated with different shoulder types. Additionally, the synthesized results pertaining to shoulders with poor drainage systems exposed to extreme moisture intrusion conditions during flooding/heavy rain events were provided to highlight the influence of moisture management capability of roadway infrastructures on failure probability of shoulders. It should be also noted that a narrow shoulder width equal to 2 ft with demanding SHL scenarios operating right at the edge of roadway shoulders were included in the slope stability analysis to consider more critical conditions.

Figure 11.51 provides the 3D color-coded plots developed in this study to represent the probability of failure of pavement shoulders under SHL vehicle operations, considering the synergistic influence of integral components such as wheel load, slope inclination, shoulder type, and moisture management capabilities. The areas highlighted in warm colors (red and yellow) indicate substantially higher probability of failure, in contrast to the regions highlighted in cold colors (blue and light blue) with lower failure probability. The developed contour maps shown in Figure 11.51 clarified the significance of wheel load magnitude and slope inclination in the probability of failure measures. As evidenced in the plots, operation of SHL vehicles with heavy wheel loads is highly detrimental to the structural integrity and stability of shoulders with steep slopes. Such detrimental impact was more pronounced in unpaved shoulders with less structural
and stress dissipation capacity, in comparison with paved shoulders with substantially higher structural capacity that dissipate the induced stresses in more efficient manner.
Figure 11.51: Contour Plots Showing the Synergistic Influence of Wheel Load and Slope Inclination on Probability of Failure of Pavement Shoulders Subjected to SHL Vehicles, for: (a) Paved Shoulder, (b) Unpaved Shoulder, and (c) Under Flooding Condition.
Figure 11.51(c) provides the average probability of failure for the studied paved/unpaved shoulders in the network at fully saturated state under flooding condition. The results captured the elevated risk against failure of sloped shoulders when subjected to simultaneous influence of SHL vehicles, extreme moisture intrusion conditions, and poor moisture management systems. Therefore, operation of SHL vehicles under demanding environmental scenarios such as heavy rain events, and during wet seasons with frequent wet-day occurrences, or during natural disasters such as flooding and hurricanes, can potentially jeopardize the stability of sloped shoulders.

The 3D contour plots presented in this chapter can be instrumental in assessing the potential failure risk of sloped shoulders subjected to SHL vehicles, with consideration of the vehicles wheel load, environmental factors, and unique characteristics of pavement shoulders in the network. The synthesized maps can also guide state DOTs during the SHL evaluation process when wide SHL trailers with multi-axle and multi-wheel arrangements impose safety/stability concerns during traveling in rural highway networks with narrow traffic lanes.

11.8.4. Summary of the Major Points

In this study, the authors developed a probabilistic approach to assess the stability of sloped pavement shoulders subjected to SHL vehicle movements. Advanced numerical techniques, in combination with MCS method, were deployed to extend the traditional LEM to a probabilistic methodology that accounts for the inherent uncertainties associated with shear strength properties of pavement unbound layers. The devised approach also accounts for the complex loading conditions attributed to SHL vehicles, dynamic nature of moving vehicles, tire-pavement interactions, environmental factors, and unique features of transportation systems for accurate assessment of pavement shoulders stability in overload corridors. The authors further performed a series of sensitivity analyses to investigate the influence of major parameters such as vehicle wheel
load, wheel offset from shoulder edge, shoulder slope, shoulder width, shoulder type, and moisture management capability, on stability of pavement shoulders subjected to SHL vehicles. The sensitivity results associated with ten representative sites evaluated in this study were then post-processed and clustered for shoulders with similar structural characteristics and paving conditions, to more realistically represent the findings obtained from slope stability analyses attributed to different shoulder types. The major findings of this study summarized as:

- The probabilistic analysis provided in this study underscored the significance of variability of influencing parameters and incorporation of their inherent uncertainties, for realistic assessment of the pavement shoulders stability under SHL vehicle operations. Therefore, relying on conventional deterministic methods that overlook the epistemic uncertainties in the analysis, in lieu of deploying probabilistic approaches, can be detrimental to the accuracy of the analysis of pavement shoulders subjected to SHLs with demanding loading conditions in overload corridors.

- The numerical simulation results highlighted the importance of vehicle wheel load on stability of sloped shoulders in this study. Analysis of 10 representative sites in overload corridors indicated that increasing the wheel load magnitude from 6 kips to 12 kips results in substantial increase in the probability of failure by 4 to 7 times, depending on the structural properties of roadway shoulders and presence of moisture.

- Shoulder width and slope inclination, are major factors that substantially contribute to the analysis of pavement shoulders under SHL vehicle operations. The sensitivity analysis provided in this study showed that the probability of failure increases with decreasing the shoulder width. This is primarily attributed to the more taxing stress paths imparted at shoulder structures by SHLs when the vehicle is in close proximity to the edge of shoulders.
with narrow widths. Additionally, the probability of failure measures showed an ascending trend with increasing the slope inclination, due to the lower shearing resistance along the slip surfaces in shoulders with higher side slopes that ultimately undermines the stability of sloped shoulders.

- Another noteworthy finding from this study pertains to the type of roadway shoulders and their sensitivity to the demanding loading conditions imposed by SHL vehicles. Based on the sensitivity analyses performed in this study, unpaved shoulders were found to be more sensitive to SHLs compared to paved shoulders, as in all numerical permutations, unpaved shoulders had a higher failure probability than the other counterparts. This is mainly attributed to the less structural and stress dissipation capacity of unpaved shoulders, in comparison with paved shoulders with more robust structures that dissipate the induced stresses in more efficient manner.

- Movement of super heavy vehicles during flooding/heavy rain events in highway networks with poor drainage systems can impose substantially higher failure risk for sloped shoulders, as compared to the potential risk against failure of shoulders with efficient drainage systems. This is in line with our expectations, since infiltration of water in shoulder structures during prolonged inundation conditions can adversely affect its shear strength capacities due to the stiffness softening behavior of unbound aggregate materials. This in turn translates into higher failure probability for the sloped shoulders when subjected to synergistic influence of SHLs, extreme moisture intrusion conditions, and poor drainage systems. Consequently, operation of SHL vehicles can potentially jeopardize the stability of sloped shoulders under flooding conditions or heavy rain events, if the
accommodating highway network lacks proper moisture management system for efficiently draining the pavement foundations and roadway shoulders.

- The results provided in this subtask substantiated the fact that unpaved shoulders with narrow widths and steep slopes are highly sensitive to the demanding loading conditions imposed by wide SHLs with multi-axle trailer units, particularly under demanding environmental scenarios such as flooding conditions and heavy rain events. Consequently, stability analysis of sloped shoulders should be an integral component in risk management studies of pavement facilities servicing SHLs in overload corridors.

- The 3D contour plots developed in this study facilitate the realistic prediction of failure risk of sloped shoulders under SHL vehicle operations, with consideration of the vehicle wheel load, environmental factors, and unique characteristics of pavement shoulders in the network. The synthesized maps can be also instrumental for state DOTs during SHL evaluation process to ensure safe operation of SHL vehicles in overload networks.
11.9. **Buried Utility Risk Analysis**

11.9.1. Introduction

Operation of SHL vehicles can negatively affect the structural integrity of utility facilities buried underneath the pavement surface, particularly if the buried utilities are aged or placed in shallow depth. The sensitivity of underground utility structures to heavy loads caused major concerns for utility stakeholders whether the existing utility facilities are structurally robust to withstand the demanding stress paths induced by non-conventional SHL vehicles with heavy tires and complex axle arrangements. A prime example of such concern pertains to the overload corridors in Texas with significant repair request records of damaged utilities. Review of the emergency repair requests for buried utility facilities in several districts across the state revealed 108 reported damages in corridors where heavy traffic loads were frequently routed (Kraus et al., 2014). Failure of underground utility facilities can interrupt supplying essential services to the public in various applications such as sewer lines, drain lines, water mains, gas lines, telephone and electrical conduits, oil and coal slurry lines, and heat distribution lines (Moser and Folkman, 2008). Essentially, the majority of these utility lines are regarded as lifelines of communities, and therefore, need to be remained in service to avoid further supply shortage effects.

Additionally, presence of underground utility systems should not intervene with the main role of highway facilities in traversing the traffic. This is of utmost importance because damaged underground utilities that are mainly located along and across the right-of-way can potentially disrupt the traffic flow, which, in turn, causes considerable inconvenience and waste of time for the traveling public. Consequently, it deems necessary to properly assess the potential risk against failure of buried utilities under SHL vehicle operations. The results can potentially provide means
to protect buried utility assets against failure under heavy traffic loads, and provide insights on risk management studies in overload corridors (Morovatdar et al., 2022a).

Since the 1990s, several researchers studied different methodologies to evaluate the detrimental impacts of heavy vehicles on pavement facilities (Gonzalez et al., 2021, Hajj et al. 2018, Chen et al. 2013, Dong and Huang 2013, Oh et al. 2011, Chatti el al. 2009, and Fernando 1997). However, there are a few studies in the literature associated with the risk assessment of buried utilities subjected to SHL vehicles. Kraus et al. (2014) evaluated the damage potential of buried pipes under SHL vehicles in Texas. The researchers, based on the laboratory test results and numerical simulation models, reported that the calculated damage ratios for both flexible and rigid pipes under SHL vehicle were less than one, indicating that the predicted vertical deformations of the pipes were less than the maximum allowable value at failure condition. In the numerical simulation phase, the researchers considered uniform distribution of contact stresses statically applied over a rectangular loaded area. The researchers also incorporated the pavement structure attributed to one unpaved roadway section, including base and subgrade layers, to simulate the pavement profile in their numerical models.

Hajj et al. (2018) developed an approach to assess the risk against failure in flexible and rigid buried structures due to SHL movement. The researchers adopted the available design procedures to analyze the failure risk of buried utilities under SHL vehicle–induced stresses. In the aforementioned study, the vertical stresses obtained from 3D-Move software were multiplied by a stress adjustment factor to estimate the resultant SHL-induced vertical stresses at the location of buried utility, due to the software limitation in modeling of soil-structure interactions and discontinuities within the medium.
The majority of the previous studies for buried utility risk analysis either rely on simplifying assumptions in the numerical simulation phase, or are based on limited sections and data points. The simplifying assumptions such as overlooking the soil-pipe interactions in lieu of realistic simulation of the interacting effects between the pipe and surrounding soil, and limitation of type of roadway facilities and pavement structures in the study, can potentially jeopardize the accuracy and reliability of the risk analyses of underground utility structures. Another anomaly persistent in the literature pertains to unrealistic simulation of the tire-pavement contact stresses using uniformly distributed load, rather than considering the non-uniform distribution of the contact stresses. Making such unrealistic assumptions can be detrimental to the accuracy of the analysis of structural impacts of SHL vehicles with demanding loading conditions.

The highlighted issues were the motivations for the authors to devise a protocol study to assess the potential risk against failure of the buried utilities under SHL vehicle operations in several FM roads, SH and US Highways in ten overload corridors. The proposed methodology accounts for the demanding loading conditions attributed to SHL vehicles, non-conventional axle assembly of SHLs with multi-axle trailer units, dynamic nature of moving vehicles with realistic tire-pavement interactions, realistic simulation of pipe-soil interactions, as well as unique characteristics of pavement facilities and buried utilities in the network.

11.9.2. Current State of Practice

The Utility Accommodation Rules (UAR) of TxDOT’s Administrative Code is commonly used by utility stakeholders in Texas to meet the requirements for design, installation, and relocation of underground utility facilities on the state right-of-way. Based on the specifics provided in Texas UAR, underground utilities crossing the highways within the right of way should be installed with a minimum depth of cover in accordance with Rule §21.40 pertaining to “Underground Utilities”.
Depth of cover is one of the key parameters for the protection of buried utilities, which is defined as the vertical distance between the roadway surface and the crown of buried utility. Table 11.10 provides the relevant information on the minimum depth of cover for crossing utilities, based on the 2021 version of Texas UAR. As indicted in Table 11.10, the minimum required depth of cover for water and sanitary sewer lines located within the right-of-way ranges between 30 in. (2.5 ft) and 60 in. (5 ft), depending on their relative location to pavement structure, (i.e., outside/under the pavement structure) and the encasement condition (i.e., encased/un-encased). It is worth noting that Texas UAR recommends the minimum depth of cover for longitudinal installations as 36 in.

Table 11.10: Depth of Cover Requirements for Underground Utilities, based on Texas UAR

<table>
<thead>
<tr>
<th>Location</th>
<th>Encasement</th>
<th>Minimum Depth of Cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside Pavement Structure</td>
<td>Encased</td>
<td>30 in. (24 in. for existing lines may be authorized), or 36 in. for un-encased section of encased lines (30 in. for existing lines may be authorized)</td>
</tr>
<tr>
<td></td>
<td>Un-encased</td>
<td>48 in. outside paved areas (reduction may be authorized if reinforced concrete slab is used)</td>
</tr>
<tr>
<td>Under Pavement Structure</td>
<td>Encased</td>
<td>18 in. or one-half the pipe diameter, whichever is greater, underneath the pavement structure (12 in. or one-half the pipe diameter may be authorized)</td>
</tr>
<tr>
<td></td>
<td>Un-encased</td>
<td>60 in. under the pavement surface or 18 in. under pavement structure, whichever is greater</td>
</tr>
</tbody>
</table>

The adequacy of the current standard specification described in Texas UAR to protect buried utility assets against failure remains questionable, particularly, under heavy traffic loads. This is because the current guideline for installation and design of buried utilities is primarily empirical in nature, and hence, needs further modifications to account for the structural properties of pavement facilities, as well as demanding loading scenarios induced by heavy traffic loads. Evidently, passage of a SHL vehicle with heavy axles and tires over a thinly surfaced FM road can potentially induce more taxing stress paths within the pavement structure and on the buried
utilities, compared to the passage of a standard truck over a well-designed US highway, with more stress dissipation capacities. Therefore, using the same minimum depth of cover for both cases leads to erratic design solutions, and can potentially compromise the structural integrity of buried utilities in overload corridors with large volume of SHL vehicle operations.

In addition to these stressed shortcomings, issues such as overlooking the influence of utility characteristics in Texas UAR, coupled with the lack of a methodology to realistically consider the pipe-soil interactions, motivated the authors to explore a new protocol to assess the risk against failure of buried utilities, considering the SHL characteristics, and unique features of pavement facilities and buried utilities in overload networks. The results can further upgrade the SHL evaluation procedure to ensure safe operation of SHLs when the proposed route accommodate underground utility facilities.

**11.9.3. Proposed Methodology for Buried Utility Risk Analysis**

This section elaborates on the proposed approach for risk assessment of buried utilities subjected to SHL vehicles. The proposed procedure consisted of three main segments, including: (1) field testing, (2) calculation of the SHL-induced vertical deflection of utility using 3D FE modeling, and (3) determination of FoS against failure of buried utilities. The site-specific information on pavement layer properties and SHL vehicle characteristics were determined from extensive filed data collection efforts using NDT equipment, as well as P-WIM deployments, in ten representative sites. The buried utility as-built plans were further instrumental in obtaining the relevant information on the type and characteristics of buried pipes such as depth of cover, wall thickness, inside diameter, outside diameter, and the material properties. Additionally, the authors conducted extensive data mining on the permit records and SHL vehicle plans to complement the information on the axle and tire characteristics of the SHLs operating in Texas overload corridors. GVW, axle
weight, axle configuration, wheel load, and tire pressure, were the most relevant traffic load parameters included in the analysis.

Subsequently, the post-processed information on pavement layers properties, buried utilities, and SHL-vehicle loading conditions, were in turn incorporated into a 3D FE system for determination of the vertical deflection of utility at the crown under SHL vehicle movement. Figure 11.52 provides a schematic view of a typical pavement structure accommodating underground utilities, as well as the deformed pipe under SHL movement.

![Schematic View of Buried Utility Facility and Deformed Pipe under SHL Vehicle Movement.](image)

Figure 11.52: Schematic View of Buried Utility Facility and Deformed Pipe under SHL Vehicle Movement.

The last stage of the analysis pertains to the characterization of FoS against failure of the buried utilities. To calculate FoS in this subtask, the SHL-induced vertical deflection of pipe, computed from the numerical simulations, was compared with the pipe’s maximum allowable deformation recommended by widely accepted buried utility design procedures, to assess the potential failure risk of the buried pipe. The maximum vertical deflection equal to 5% of inside diameter was incorporated in the analysis as failure threshold for flexible pipes, while 2% deflection criterion, as typically adopted in design practice, was considered for risk analysis of rigid pipes (Moser and Folkman, 2008, ASTM Standard, 2011, and Kraus et al., 2011; 2014).
Consequently, the FoS against failure for flexible and rigid pipes under SHL vehicle movement was calculated from Equation 1 as:

$$\text{Factor of Safety (FoS)} = \frac{(U_y)_{\text{max}}}{(U_y)_{\text{SHL}}} = \frac{(0.05 \text{ or } 0.02)D_{\text{in.}}}{U_y}$$

where, $(U_y)_{\text{max}}$ is the maximum allowable vertical deflection of pipe that is equal to 5% of inside diameter for flexible pipe, and 2% of inside diameter for the rigid pipe, $D_{\text{in.}}$ is inside diameter of pipe, and $(U_y)_{\text{SHL}}$ is the pipe vertical deformation at the crown induced due to SHL movement.

### 11.9.3.2. Buried Utility Characteristics

The authors conducted data mining on documents and as-built plans provided by TxDOT open records coordinators in different districts in Texas to extract the relevant information on buried utility characteristics such as utility type, utility size, utility material, depth of cover, soil condition, as well as type of overlying pavement. Figure 11.53 provides an example of buried utility plans associated with State Highway 359 in Laredo District. The underground utility demonstrated in Figure 11.53 pertains to the water pipes in City of San Diego in Duval County, with pipe diameter of 10 in., and depth of cover as 2.59 ft. It is also worth noting that historical records of repair requests for damaged utilities in Texas Districts indicated that 53 out of 108 reported damaged cases were attributed to water and sewer lines, while the remaining 55 damaged cases were categorized under communication, oil and gas, culverts, and other types of underground utility facilities (Kraus et al., 2011). Such alarming occurrence of failure in water and sewer lines in the studied network motivated the authors to emphasize on this type of underground utilities.
Figure 11.53: Example of Buried Utility Plans for Water Lines in State Highway 359 in Laredo (from Personal Correspondence with TxDOT, April 2020).

Review of the available literature, coupled with our research team’s inspection survey in the network, were also instrumental in complementing the required information on the characteristics of buried utilities within TxDOT right-of-way. Table 11.12 provides a summary of the relevant information on material properties, structural characteristics, as well as dimensions and wall thicknesses, for both flexible and rigid pipes, deployed in water and sewer lines. This information was a direct input into the analysis of buried utility structures in this study.
Table 11.11: Buried Utility Characteristics used in the Risk Analysis

<table>
<thead>
<tr>
<th>Pipe Type</th>
<th>Material</th>
<th>Elastic Modulus (psi)</th>
<th>Poisson’s Ratio</th>
<th>Nominal Pipe Size (in.)</th>
<th>Average Outside Diameter (in.)</th>
<th>Average Inside Diameter (in.)</th>
<th>Wall Thickness (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexible</td>
<td>Poly-Vinyl Chloride (PVC)</td>
<td>400,000 psi</td>
<td>0.4</td>
<td>10</td>
<td>10.5</td>
<td>9.6</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>18</td>
<td>18.7</td>
<td>17.0</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
<td>24.8</td>
<td>22.5</td>
<td>0.7</td>
</tr>
<tr>
<td>Rigid</td>
<td>Concrete</td>
<td>2,900,000 psi</td>
<td>0.2</td>
<td>10</td>
<td>11.8</td>
<td>10.0</td>
<td>0.9</td>
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<td></td>
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<td>18</td>
<td>21.0</td>
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<td></td>
<td>24</td>
<td>28.2</td>
<td>24.0</td>
<td>2.1</td>
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</tbody>
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11.9.3.3. **Numerical Simulations**

Realistic simulation of SHLs moving at the pavement surface and their effect on utilities buried underneath the pavement structure involves several aspects of complexity in terms of the analysis approach, loading conditions, vehicle speed incorporation, tire-pavement interactions, and pipe-soil interactions. In this study, ABAQUS FE program was used for the calculation of the induced vertical deflection of buried utilities under SHL vehicle movement, with considerations of the complex loading conditions, SHL-vehicle dynamics, and non-uniform tire-pavement contact stress distributions. Moreover, the authors used the “embedded constraint” offered in ABAQUS to realistically simulate the buried utilities and their interactions with the surrounding subgrade soil.

Figure 11.54 provides an example of the simulated SHL nucleus, as well as the buried utilities, for calculation of the SHL-induced vertical deflection of pipe using 3D finite element modeling. The representative set of 4×4 tires shown in Figure 11.54 pertains to the most critical SHL case, i.e., 28A-8T with demanding axle assembly and tire arrangement, which was selected as the SHL case study in this analysis. Another noteworthy information associated with the numerical simulations was modeling of the buried utilities in longitudinal direction along the
model with no offset from the load centerline to consider more critical scenarios when the SHL vehicle moves along the longitudinally installed utilities. It should be also noted that relatively low vehicle speed \((v = 20 \text{ mph})\) was assigned to the explicitly modeled tire elements to account for the slow-moving nature of SHLs in this study. Detailed information on FE modeling, tire element characteristics, boundary conditions, tire-pavement contact stresses, and frictional interactions, are provided in Chapter 8.

![Simulation of Pavement Structure and Buried Pipe under SHL Vehicle Movement, using 3D FE Models.](image)

**11.9.3.4. Parametric Analysis of Buried Utility Risk Assessment**

The authors conducted a series of parametric analyses to evaluate the influence of major parameters such as wheel load magnitude, utility offset from load centerline, depth of cover, pipe diameter, pipe type, and type of roadway facilities, on the vertical deformation of buried pipes under SHL vehicle movement. The pipe vertical deformation obtained from the numerical simulations were in turn converted to “FoS” measures to facilitate the quantitative comparison of different case scenarios.

Figure 11.55 provides an overview of the analysis parameters, as well as the ranges/permutations, incorporated into the devised parametric study. As illustrated in Figure 11.55, wheel load ranged all the way from 6 kips to 12 kips, utility offset from load centerline changed from zero (no offset) to 12 ft, depth of cover ranged between 3 ft to 7 ft, while different pipe diameters, i.e., 10 in., 18 in., and 24 in., different pipe types, i.e., flexible and rigid, and various
types of roadway facilities, i.e., FM, SH, and US, were incorporated in the analysis. Review of design procedures and available resources, such as Texas UAR, TxDOT documents, and buried utility plans, coupled with our team’s experience in relevant projects, with consideration of unique features of roadway infrastructures and traffic loading conditions in the surveyed network, provided the basis for the ranges selected for each analysis parameter. The following sections provide the post-processed results and relevant discussions on the performed parametric analysis.

![Figure 11.55: Numerical Permutations Incorporated in the Parametric Analysis of Buried Utility Risk Assessment.](image)

11.9.4. Analysis of Results and Discussions

11.9.4.1. Influence of SHL-Vehicle Wheel Load on Buried Utility Failure Risk

This section provides the synthesized results pertaining to underground utility structures in one of FM roadways with thin surface and base layers, i.e., FM 468, to showcase the analysis procedure developed in this study for risk assessment of buried utilities subjected to SHLs with multi-axle trailer units. Various load magnitudes on SHL-vehicle tires, i.e., 6 kips, 8 kips, 10 kips, and 12 kips, were also incorporated into the analysis to account for the potential risk against failure of
buried utilities under different loading scenarios. The buried utility evaluated in this section comprised of an 18 in. flexible pipe with inside diameter of 17 in. and 0.5 in. wall thickness, positioned at 3 ft from the top of pavement surface.

Figure 11.56 demonstrates the numerical simulation results attributed to vertical deflection contour plots for buried pipe under different SHL-vehicle wheel loads. As evidenced in Figure 11.56, the maximum vertical deflection of the pipe was 283 mils under studied SHL case with wheel load magnitude of 6 kips. The induced vertical displacement of the pipe was more substantial for numerical simulations with higher wheel load magnitudes than 6 kips. This is as expected, since heavier wheel loads can impose more taxing stresses on the buried utilities, leading to an increase in the pipe vertical deformation. Based on the results provided in Figure 11.56, inclusion of wheel loads as 8 kips, 10 kips, and 12 kips into the analysis resulted in significant vertical displacement for the studied pipe as 471 mils, 740 mils, and 1,062 mils, respectively. Such substantial pipe deformation under heavy wheel loads highlights the importance of wheel load magnitude and its structural impacts on buried utility assets in OW highway networks.

Figure 11.56: Influence of SHL-Vehicle Wheel Load on Pipe Vertical Deflection at the Crown.
The pipe vertical deflections obtained from the numerical simulations were in turn converted to “FoS” measures, by contrasting the induced deflections with the 5% deflection criterion typically allowed in practice. Figure 11.57 shows the calculated values for FoS against failure of the buried pipe in FM 468, associated with different load magnitudes of SHL-vehicle tires to provide a broader insight on the sensitivity of FoS to loading magnitude. The descending nature of FoS measures in Figure 11.57 clarifies the influence of wheel load magnitude on the structural integrity of buried utilities. Based on the post-processed results provided in Figure 11.57, increasing the wheel load from 6 kips to 12 kips results in significant decrease in the FoS from 3.0 to 0.8, indicating 3.8 times decrease. The figure also shows that, for the studied buried pipe in FM 468, the numerical simulations with 11 kips and 12 kips wheel load magnitudes had a FoS lower than one, indicting failure condition. This emphasizes the need for alternative mitigation strategies to protect buried utility facilities against failure in similar occasions, and to ensure safe operation of SHL vehicles that carry extremely heavy wheel loads.

The results substantiated the fact that operation of SHL vehicles with heavy wheel loads can potentially compromise the structural integrity of buried utilities in overload corridors, considering the non-conventional loading conditions of SHLs with numerous axles and tires. Therefore, analysis of the wheel load magnitude should be an integral component in risk analysis of buried utilities subjected to SHLs with multi-axle trailer units in overload corridors.
11.9.4.2. Influence of Utility Offset from Load Centerline on FoS against Failure

Figure 11.58 shows the variations of FoS with changes in the lateral distance of buried utilities with respect to traffic load centerline. An 18 in. flexible pipe with inside diameter of 17 in. and 0.5 in. wall thickness, with depth of cover of 3 ft was considered in the numerical simulation models. Additionally, vehicle wheel load of 12 kips, and pavement profile of FM 468, were incorporated in the sensitivity analysis performed in this subtask.

Based on the numerical simulation results provided in Figure 11.58, decreasing the transverse distance between the pipe axis and the centerline of the applying load under SHL tires moving at the pavement surface significantly increases the risk against failure of buried utilities. As evidenced in Figure 11.58, decreasing the lateral distance between the buried pipe and the load centerline from 12 ft to zero (right at the centerline) results in significant decrease in the FoS by nearly 4.4 times (i.e., from 3.5 to 0.8). Such trend is in line with our expectations, since when the buried pipe is positioned symmetrically with respect to the SHL tires with no offset from the load centerline, the overlapping stress distributions induced on the pipe structure due to adjacent SHL tires are more pronounced, compared to the buried pipe located at a distance away from the load centerline.
centerline. This can result in higher pipe deformation and higher failure risk, and therefore lower FoS measures, when the buried pipes are located right at the load centerline. Consequently, it deems necessary to install underground utility structures at an appropriate distance from the SHL vehicle routes to mitigate the risk against failure of buried utilities in overload corridors.

![Figure 11.58: Influence of Lateral Distance between Buried Utility and Load Centerline on FoS.](image)

**11.9.4.3. Influence of Depth of Cover on FoS against Buried Utility Failure**

Figure 11.59 shows the sensitivity of FoS to the changes in the depth of cover for buried utilities. The results attributed to various roadway types, i.e., FM 468, SH 123, and US 281, were also contrasted with each other for comparison purposes in Figure 11.59. Vehicle wheel load of 12 kips, and 18 in. flexible pipe with inside diameter of 17 in., were the identical parameters incorporated in the sensitivity analysis performed in this subtask. As evidenced in the plot, the shallower the buried pipe, the lower the calculated FoS against failure. This is mainly attributed to more taxing stress distribution patterns imposed on shallow-buried pipes under SHL movement. Higher induced stresses in turn translates into lower FoS against failure in utilities buried at lower cover depth. Consequently, underground utility structures that are located at shallow depths are highly sensitive to the demanding loading conditions imposed by SHL vehicles.
Another noteworthy observation from Figure 11.59 pertains to the type of roadway facilities and its influence on the buried utility risk assessment in overload corridors. As evidenced in Figure 11.59, the studied utility in FM roadway was found to be more sensitive to SHL vehicle movement compared to the same utility simulated in SH and particularly US highway, as in all numerical permutations, the FM roadway has the lowest FoS against utility failure, followed by SH, and ultimately US highway. This is mainly attributed to the fact that structurally deficient FM roadways have lower stress dissipation capacities, compared to pavement sections in SH and US highways with more robust structures that dissipate the stresses within the pavement layers and buried utilities in more efficient manner. Therefore, protocols for risk assessment of buried utilities in OW networks should properly account for the influence of pavement profile and type of roadway facilities to improve the accuracy of results.

![Figure 11.59: Influence of Depth of Cover on FoS against Buried Utility Failure for Different Roadway Types.](image)

The sensitivity results provided in Figure 11.59 also indicates that decreasing the depth of cover from 7 ft to 3 ft results in significant decreases in the FoS measures from 3.5, 3.4, and 2.8 to 1.1, 1.0, and 0.8 for the buried pipes in representative US, SH, and FM roadways, respectively.
Figure 11.59 also shows that, in all three studied roads, the buried pipes at 3 ft cover depth either closely approached or surpassed the failure threshold (i.e., FoS = 1). Accordingly, the synthesized analysis results suggest a minimum depth of cover as 4 ft to satisfy the deflection limit of buried pipes under SHL vehicle movement in OW highway networks.

11.9.4.4. Influence of Depth of Cover on FoS against Failure for Different Pipe Diameters

Figure 11.60 illustrates the synergistic influence of depth of cover and pipe diameter on the calculated FoS values. Three different pipe diameters including, i.e., 10 in., 18 in., and 24 in., with corresponding wall thicknesses as 0.3 in., 0.5 in., and 0.7 in., respectively, were included in the analysis. Additionally, vehicle wheel load of 12 kips, and pavement profile of FM 468, were incorporated in the sensitivity analysis performed in this subtask.

The results provided in Figure 11.60 underscores the significant role of pipe geometric properties in the buried utility risk studies. Based on the sensitivity results shown in Figure 11.60, pipes with smaller diameter were found to be more sensitive to SHL vehicle movement compared to the larger diameter pipes. As evidenced in the plot, the FoS against failure decreased by using smaller diameter pipes. This observation is in line with the previous findings documented in the literature (Rajani et al., 2000, Hu et al., 2007, and Kerwin et al., 2020). For the range of input parameters incorporated in this analysis, the results revealed that the buried pipes with 10 in. diameter had the lowest FoS, while the 18 in. pipes and particularly 24 in. pipes showed higher FoS values. This is mainly attributed to the fact that pipes with smaller diameters and lower wall thicknesses have a lower moment of inertia than the larger pipes with higher cross-sectional moment of inertia. This in turn translates into lower bending capacity, and therefore, lower FoS against longitudinal bending failure for smaller pipes under heavy traffic loads.
Accordingly, structural effects of SHL vehicles on pipes with smaller diameters are more substantial, compared to larger pipes. Such detrimental impact was more pronounced for pipes buried at shallow depths. Therefore, operation of SHL vehicles can potentially jeopardize the structural integrity of shallow-buried utility facilities comprised of pipes with small diameter and thin walls located across the overload corridors.

![Figure 11.60: Influence of Depth of Cover on FoS against Failure for Different Pipe Diameters.](image)

**11.9.4.5. Influence of Depth of Cover on FoS against Failure for Different Pipe Types**

Figure 11.61 illustrates the synergistic influence of depth of cover and pipe type on the calculated FoS values. 18 in. flexible and rigid pipes, with depth of cover of 3 ft were considered in the FE models. Additionally, vehicle wheel load of 12 kips, and pavement profile of FM 468, were incorporated in the sensitivity analysis performed in this subtask.

Based on the sensitivity results presented in Figure 11.61, the calculated FoS values drastically decreased as the pipe material changed from “concrete” for rigid pipe to “PVC” for flexible pipe, due to the lower material stiffness properties. Such deviation between the two studied pipes was more substantial at lower depth of cover. As evidenced in the plot, inclusion of depth of cover of 7 ft in the analysis resulted in FoS as 2.8, and 3.2 for flexible, and rigid pipes, respectively,
indicating 0.4 difference in the calculated FoS. However, the FoS values for the same pipes placed at 3 ft cover depth were found to be 0.8 for flexible pipe, and 1.8 for the studied rigid pipe, reflecting major difference, as high as 1.0, in the FoS values. Therefore, protocols for risk assessment of buried utilities should take into consideration the material properties of buried utilities, besides depth of cover, to achieve reliable results.

![Factor of Safety (FoS) vs Depth of Cover (ft)](image)

Figure 11.61: Influence of Depth of Cover on FoS against Failure for Different Pipe Types.

11.9.5. Summary of the Major Points

In this study, the authors devised a methodology to assess the potential risk against failure of the buried utilities under SHL vehicle operations. Field-derived information on truck traffic, pavement profile, and utility structure, in combination with advanced numerical techniques, were used to bridge the existing gap in the literature for realistic assessment of the failure risk in underground utility structures, considering the traffic loading conditions, and unique features of roadway infrastructures in overload corridors. The devised approach also accounts for the demanding loading conditions and non-conventional axle assembly of SHLs with multi-axle trailer units, non-uniform tire-pavement contact stresses, realistic simulation of pipe-soil interactions, as well as unique characteristics of buried utilities, to improve the accuracy of the utility risk analyses. The
authors further conducted a series of parametric analyses to investigate the influence of major parameters such as wheel load magnitude, utility offset from load centerline, depth of cover, pipe diameter, pipe type, and type of roadway facilities, on the FoS against failure of buried pipes subjected to SHL vehicle movements. The sensitivity results for different numerical permutations were then post-processed and clustered for roadways with similar structural characteristics, to more realistically represent the findings obtained from buried utility risk analyses attributed to different roadway types. The major findings of this study summarized as:

- Operation of SHL vehicles can potentially jeopardize the structural integrity of utilities buried underneath the pavement structures. Consequently, risk analysis of buried utilities should be an integral component in risk management studies of transportation facilities servicing SHLs in overload corridors.

- The numerical simulation results underscored the significant role of vehicle wheel load in the buried utility risk studies in overload corridors. The parametric analysis performed in this study indicated that increasing the wheel load magnitude from 6 kips to 12 kips leads to significant reduction in the FoS against failure by “1.5 ~ 3.8” times, depending on the structural properties of buried utilities and overlying pavement structure, as well as depth of cover. Therefore, risk assessment protocols for buried utilities should properly account for the load magnitude on each individual SHL tires and its contribution to the damages imparted on underground utility facilities.

- Relative location of buried utilities with respect to moving SHL tires substantially affects the level of risks against failure. Analysis of different numerical permutations in this study showed that underground utility structures that are longitudinally installed under the pavement structure with “minimal to no” transverse offset from the load centerline are
highly sensitive to the demanding loading conditions imposed by heavy vehicles. Such sensitivity was found to be more pronounced for utility structures placed at shallow depths, due to more taxing stresses induced on shallow-buried utilities under SHL movement. Accordingly, it is imperative to install underground utilities with an appropriate distance from the SHL routes and at a sufficient depth of cover to mitigate the risk against failure of buried utilities in overload corridors.

- The synthesized results obtained from a comprehensive parametric analysis performed in this study suggested a minimum depth of cover as 4 ft to satisfy the maximum allowable deflection limit (2%-5% of inside diameter) of buried pipes under SHL vehicle movement in OW highway networks.

- Geometric characteristics and material properties of underground utilities are major factors that substantially contribute to the risk analysis of buried utilities under SHL vehicle operations. The sensitivity analysis provided in this study indicated that the FoS against failure decreases by using smaller diameter pipes with lower wall thicknesses. This is in line with our expectations, since pipes with smaller diameters and thinner walls have a lower moment of inertia, and therefore, lower bending capacity under heavy traffic loads, compared to larger pipes with higher bending capacities. Furthermore, the calculated FoS measures drastically decreased with changing the pipe material from “concrete” for rigid pipe to “PVC” for flexible pipe, due to the lower material stiffness properties. Therefore, operation of SHL vehicles can potentially jeopardize the structural integrity of utility facilities comprised of flexible pipes with small diameters and thin walls.

- The utilities buried underneath the pavement structure of FM roadways were found to be more sensitive to SHL vehicle movement, in contrast with the same utilities simulated in
SH and US highways. This is mainly attributed to the fact that structurally deficient FM roadways have lower stress dissipation capacities, compared to pavement sections in SH and US highways with more robust structures that dissipate the stresses within the pavement layers and buried utilities in more efficient manner.

- Protocols for risk assessment of buried utilities in OW networks should properly account for the synergistic influence of location, type, dimension, wall thickness, material properties, and depth of cover of buried utilities, with consideration of roadway structural characteristics, for realistic assessment of the potential risk against failure in utility facilities under SHL vehicles. Hence, relying on simplifying assumptions that tend to overlook the influence of major analysis parameters can potentially jeopardize the buried utility risk studies in OW corridors.

- The results provided in this subtask substantiated the fact that shallow-buried utilities consisted of flexible pipes with small diameters and thin walls that are longitudinally installed under pavement structures have considerable potential to be damaged under the passages of SHL trailer units that carry extremely heavy axles and tires. This emphasizes the need for alternative mitigation strategies to protect buried utility facilities against failure in similar occasions, and to ensure safe operation of SHL vehicles in overload corridors.
Chapter 12: Conclusions and Recommendations

This chapter summarizes the extensive efforts made in this study to characterize the structural impacts of SHL vehicles on transportation infrastructures such as highways, pavements, roadway shoulders, and underground utilities. The primary goal of this study was to mechanistically quantify the damages and structural impacts imparted on transportation facilities due to SHL applications. The proposed analysis protocol consisted of multiple categories, including the analysis of loss of pavement service life, analysis of slow-moving nature of SHLs, analysis of acceleration/deceleration forces, analysis of roadway geometric features, stability analysis of sloped pavement shoulders, and buried utility risk assessment. The secondary goal of the research was to synthesize the field data and numerical analysis to provide further insights into necessary adjustments to upgrade and overhaul the current mechanistic-empirical (ME) pavement design protocols, considering the complex nature of SHLs with taxing loading conditions.

To achieve research objectives, initially, the research team developed a comprehensive database of SHLs in demanding corridors of energy sectors in Texas, based on the calibrated filed data collected by P-WIM units in ten representative sites. Two sets of data were collected in the summer and winter times to analyze the seasonal variation of the heavy truck traffic in this study. A supplementary database of SHLs with multi-axle trailers was also developed based on the most recent permit records issued by TxDMV Motor Carrier Division in both Eagle Ford and Permian Basin regions. Along with the traffic characterization efforts, the research team performed a series of non-destructive tests such as GPR and FWD to better understand the structural capacity of ten representative sites. The research team then developed a methodology for clustering unconventional trucks carrying super heavy loads, and further developed an algorithm for the determination of the number of influencing tires extrapolated outside of the nucleus axle of the
multi-axle trailers. The selection of the number of tires and axle assemblies was primarily based on the hyperbolic effect and the geospatial influence zones in structural analysis of pavements.

The relevant information on loading conditions attributed to SHL units, coupled with pavement structural properties, were in turn incorporated in a series of numerical simulations for the determination of pavement responses subjected to taxing traffic conditions. A finite element code was further developed that makes adequate provisions for advanced modeling of moving SHL vehicles to accurately calculate the induced pavement responses under complex loading conditions. The numerical simulation models account for the viscoelastic nature of asphalt layers, non-uniform distribution of tire-pavement contact stresses, various vehicle operational speeds, and acceleration/deceleration forces, to realistically simulate the dynamic nature of moving SHLs.

A methodologically sound approach for the calculation of the site-specific pavement damage equivalency was further developed in this study. The site-specific damage factors for each load group were calculated using material properties in summer and winter seasons to quantify the influence of seasonal variation of material properties on the damage accumulation in overload corridors. In another effort, the research team developed a framework for the determination of the remaining service life of the pavement sections in this study. To achieve this objective, the research team developed an algorithm to backtrack the current traffic information to pre-energy developing era to quantify the energy developing impacts and freight transportation on highway facilities.

Ultimately, by using the field-derived databases and the devised analysis algorithms, an all-encompassing protocol was established for the mechanistic characterization of the loss of pavement service life imparted by SHL vehicle operations, considering the unique nature of super heavy trucks with complex axle assemblies. The multi-tier framework consisted of provisions to account for the influence of acceleration/deceleration, slow-moving nature of the trailers, roadway
geometric features at curved segments, and demanding environmental scenarios such as flooding conditions and elevated temperatures in summer seasons, on the longevity of pavement facilities. A series of parametric analyses were then conducted to provide a better understanding of major analysis parameters and their contribution to distress accumulation under SHL passage, with consideration of the most demanding loading conditions, vehicle loading scenarios, site-specific traffic makeup, unique features of transportation facilities, and environmental impacts.

As part of this study, a novel probabilistic approach was also devised to assess the stability of sloped pavement shoulders subjected to SHLs. Advanced numerical techniques, in combination with Monte-Carlo simulations, were deployed to extend the traditional LEM to a probabilistic methodology that accounts for the inherent uncertainties associated with shear strength properties of pavement unbound layers. Furthermore, the potential risk against failure of the buried utilities under SHL vehicle operations was also investigated. The proposed approach aimed to bridge the existing gap in the literature for realistic assessment of the failure risk in underground utility structures, considering the pipe-soil interactions, non-conventional loading conditions, and unique features of roadway infrastructures in overload corridors.

The results were then synthesized in risk management heat maps to facilitate the practical implementation of research findings. The synthesized contour maps can be further instrumental for pavement design and analysis Divisions in state DOTs during SHL evaluation process in having a mechanistic means for the approval (or rejection) of SHL permits, considering the loading conditions, climatic factors, and characteristics of pavement facilities in the network. This is of great benefit to state DOTs to ensure safe operation of SHL vehicles in overload networks, by minimizing the imparted damage and mitigating the safety concerns for the traveling public across SHL routes. The best practice recommendations presented in this study can also guide highway
agencies in adopting proper M&R strategies to preserve the existing transportation networks in overload corridors. This can potentially protect state assets by reduction or elimination of reconstruction costs associated with premature failure of transportation facilities subjected to heavy truck traffic.

The following sections provide noteworthy observations and a summary of major findings pertaining to structural impacts of SHL vehicles in overload highway networks. The conclusions are accompanied by suggestions for feature work and research potential in the area of analysis and design of pavement structures subjected to taxing loading conditions under SHL trailers.

12.1. CONCLUSIONS

A summary of major findings of the study is provided in the following:

- Prior to the development of field experimental plans, an online survey was conducted among the districts located in the overload corridors of East and Southeast Texas. 94% of the respondents indicated that their transportation infrastructure had been adversely affected by heavy truck traffic.

- Based on the analysis of the questionnaire regarding the types of pavement distresses, 82.4% were attributed to rutting, 82.4% to potholes, and 76.5% to fatigue cracking in the overload zones.

- A comparative analysis between portable and stationary WIM systems confirmed the reliability, quality, and consistency of the traffic information obtained by P-WIM units. Accordingly, the P-WIM system is a verified and viable alternative to costly and labor-intensive stationary WIM systems for collecting accurate and reliable site-specific traffic data required for ME pavement design and analysis.

- Based on the post-processing of the P-WIM traffic data, on average 64% of the truck traffic
in FM roads exceed the Texas legal axle load limits. The State Highways and US Highways in this study also had alarming OW percentages of 36% and 45%, respectively. Additionally, nearly 25% of the recorded traffic mix in FM roadways were attributed to passages of SHL vehicles. The average of the traffic makeup in SH and US highways in the studied network were 20%, and 7%, respectively, attributed to SHL vehicles.

- Analysis of the P-WIM data indicated that the majority of the studied sites accommodated SHL vehicles with significantly heavy tires, as the maximum recorded ATHWL was as high as 14.1 kips. Several SHL trucks under Class 9, 10, and 13 with GVW ranging from 250 kips to 364 kips were captured in the field.

- Review of the most recent permit records of SHLs in Eagle Ford Shale and Permian Basin regions indicated that more than 8,000 SHL permits with GVW in excess of 250 kips were issued during the 2017-2020 period, with the maximum recorded as high as 1.8 million (lb). Based on the information retrieved from available permit records, SHL units can comprise of specialized trailers with a significant number of axles as high as 45, while the most common SHL trailers consisted of 15 axle lines.

- Based on relevant data mining techniques, SHLs were categorized into three distinct groups: Category (I): FHWA class 4-13 super heavy trucks, Category (II): SHLs with evenly-distributed multi-axle trailers, and Category (III): SHLs with separate dollies of multi-axle trailers. The synthesis of the available data such as P-WIM databases, SHL plans, and the permit records, etc., were the basis for the clustering information. It was found that SHLs classified under category (I) carry more demanding axles and tire loads compared to the SHLs with multi-axle trailers in categories (II) and (III), and hence they can be more detrimental to the service life of pavement structures.
• Operation of heavy truck traffic in the surveyed network adversely affected FM roads more than the SH and US highways. This is in line with our visual field observations and forensic distress identification reports of FM roads in the overload corridors.

• Considerations of the complex and unique nature of SHLs, inherent aleatory variability of influencing parameters, and incorporation of epistemic uncertainties in the analysis of pavements subjected to SHLs can protect pavement facilities from premature failure in overload corridors.

• The research team developed a novel approach for the estimation of the site-specific damage factors with considerations of environmental factors on the material properties of layers, and the type of the pavement facility in this study. The developed mechanistic approach confirmed that the modified EALF values were substantially higher than traditional industry-standard axle load factors currently employed by the pavement design industry. This can result in underestimation of the potential damages imparted by heavy vehicles in overload zones.

• The EALF values based on surface deflection and rutting criteria were highly sensitive to the increasing axle load magnitude in numerical simulations. This behavior was anticipated, as the distresses associated with heavy truck operations are essentially attributed to the load magnitude rather than the load repetitions. Therefore, rutting and surface deflection criteria led to higher damage factors in comparison with fatigue criterion.

• Overlooking the influence of the type of roadway facility in damage quantification can potentially incur systematic errors for accurate assessment of the service life of the pavements in overload corridors. Therefore, it deems necessary to cluster similar highways,
in terms of functionality and structural layer profile, to realistically represent the damages imparted by SHL vehicle operations.

- The detrimental effect of SHLs was higher for pavements with lower structural capacity. This is in line with expectations due to the fact that heavy axle loads are more destructive to less robust pavement profiles such as FM roads than to US roadways. Based on the analysis provided in this research, FM roadways had the highest damage factors, followed by SH, and ultimately US highways. Therefore, the algorithms for the quantification of pavement damage should have the flexibility to adjust when the transition of the SHL from SH or an interstate highway to a FM road occurs.

- The numerical modeling results underscored the significance of realistic simulation of moving SHL vehicles and tire-pavement interactions for proper quantification of the damages imparted on pavement structures. Consequently, the protocols for analysis and design of pavements accommodating SHLs should properly account for the non-uniform distribution of tire-pavement contact stresses, rather than relying on simplifying assumptions such as using uniformly distributed loads.

- Accurate assessment of pavement damage should include seasonal variation of the material properties of pavement layers. This is primarily attributed to the viscoelastic behavior of the asphalt layers and the variations of stiffness properties of granular layers due to freeze-thaw effects or changes in the saturation state of the unbound granular layers due to moisture infiltration or evapotranspiration during the service life of pavements. Therefore, incremental assessment of the damages, rather than single value average damage factor, better represent the detrimental influence of SHLs in highway networks.
• Post-processing the FWD data for summer and winter seasons showed that the back-calculated layers modulus values in summer were significantly lower compared to winter time modulus values. This is primarily attributed to the viscoelastic behavior of the asphalt layer, and the stiffness softening of the granular base and subgrade layers due to moisture ingress in multi-layer pavement structure in wet seasons.

• The damages imparted by SHL vehicle during hot summer months were found to be substantially higher compared to the damage imparted by the same vehicle in cold winter months. Similarly, intrusion of moisture in granular layers due to poor drainage conditions in structurally deficient roadways was a major contributor to premature failure of pavements subjected to SHL vehicles. Therefore, pavement damage mechanisms should be able to account for the cyclical nature of material properties due to environmental conditions.

• Comparisons between the predicted and field-measured rut depths in this study indicated that the proposed methodology for the estimation of pavement remaining life is capable of accurately assessing the incremental progression of distresses during its design/service life.

• Accurate account of traffic characteristics is the precursor for reliable assessment of damages imparted on pavements in overload corridors. Site-specific axle load spectra data provides the primary ME traffic data input for accurate and optimal pavement design and analysis. Incorporating the site-specific axle load spectra in the pavement life analysis routines rather than using agency default database in numerical simulations leads to remaining life analysis drastically improved the agreement between the simulations and field distress measurements.

• Remaining service life analysis of the three studied FM roadways indicated that despite
several remedial measures since 2008, almost all sections are approaching the predefined distress limits that define the service life of pavement sections. This is primarily attributed to the fact that these roadways with extremely thin surface treated layers were not designed to withstand current traffic loading conditions with large volumes of heavy truck traffic operations.

- Mechanistic characterization of the loss of pavement service life underscored the significance of SHL operations and their role to jeopardize the longevity of pavement facilities. The numerical simulation of several FM, SH, and US highways in Texas energy sectors indicated that operation of SHL and OW vehicles in FM roadways can impart substantial PLRs as high as 55%, and 62%, respectively, over the 20-year design life. The greatest loss of pavement service life due to heavy vehicle movements ranged between 33% and 38% for the State Highways, and between 25% and 32% for US Highways.

- The destructive effect of moving SHLs was substantially higher under heavier wheel load magnitudes. Analysis of 10 representative pavement sections in this study indicated that increasing the load magnitude on SHL tires from 6 kips to 12 kips resulted in drastic increase in the imparted pavement damages by 7-12 times, depending on the tire arrangement, axle assembly, roadway structural characteristics, and climatic conditions. It was also found that increasing the maximum wheel load in the same range leads to drastic increase, by 3-4 times, in the imparted loss of pavement life during the 20-year design life. Accordingly, design and analysis of pavement facilities servicing overload corridors should properly account for the heaviest wheel load of SHLs.

- The numerical simulation results highlighted the importance of moisture management capability of roadway infrastructures during extreme moisture conditions such as flooding
events, when combined with taxing loading scenarios. Operation of SHLs in roadways with frequent heavy rain events and poor moisture management performances, imparted substantially higher pavement life consumption, compared to the case scenarios with excellent moisture management systems that can drain the infiltrated water in a more efficient manner. This is primarily attributed to the reduction in stiffness properties of granular layers due to moisture infiltration under prolonged inundation conditions such as flooding or elevated GWT during the service life of pavements with poor drainage systems.

- Overlooking the dynamic nature of traffic demands in transportation networks can potentially jeopardize damage quantification and PLR studies in overload corridors. Therefore, pavement analysis protocols should consider such variations in traffic demands and annual growth rates, for proper estimation of the projected traffic over design period; otherwise, it may prove more challenging to accommodate traffic demand fluctuations in transportation networks in overload corridors.

- Slow-moving state of the SHLs can potentially result in higher level of damages imparted on the pavement facilities, compared to the same vehicle traveling at conventional speed conditions. This is primarily attributed to the viscoelastic nature of the asphalt layer, and the loading rate dependency of the materials properties in flexible pavements.

- Inclusion of acceleration/deceleration of SHLs in the pavement analysis resulted in substantially higher damages, compared to the same vehicle traveling at a constant speed. This is mainly due to the dynamic nature of the analysis, and the added longitudinal and tangential forces at tire-pavement interface under speeding and braking actions. Therefore, pavement structures, particularly at intersections, junctions, stop–go sections, or speed hump areas, can potentially experience excessive distresses due to sudden variations in
SHL vehicle speed. Consequently, analysis of acceleration and deceleration of SHLs should be an integral component in risk management studies of pavement facilities servicing the overload corridors.

- Realistic simulation of roadway geometric features and the induced tire-pavement contact stresses is the precursor for reliable assessment of the damages imparted on pavement sections by SHLs during turning movements. The 3D numerical simulations showed that the maximum shear and vertical stresses imparted at the tire-pavement interface under a 15-kips SHL tire moving at horizontal curve section were 46%, and 34%, respectively, higher compared to straight segments. This is mainly due to the centrifugal effects and the localized stress concentration under turning/cornering tires.

- Overlooking the influence of geometric design of roadways at curved sections results in underestimation of damage factors by 16% to 68%, depending on the SHL axle assembly and roadway type. Therefore, damage quantification algorithms should manifest such sensitivity to roadway curvature for accurate assessment of distresses at horizontal curves.

- Operation of SHLs at curved segments resulted in significantly higher pavement life consumption during the 20-year design life by 18% to 42%, compared to the PLR imparted by the same SHLs typically operating along the straight paths without vehicle turning/maneuvering actions. Consequently, mechanisms for the quantification of pavement life reduction should be capable of accounting for the excessive distress accumulation when transition of SHL from straight sections to curve sections occurs.

- Super-elevation and curve radius are the key geometric design parameters that substantially contribute to the performance of pavements at bend segments. It was found that the
accumulated damage under SHLs is more pronounced at sharp horizontal curves with high super-elevation rates, due to more taxing tire-pavement contact scenarios.

- The stability analysis of pavement shoulders indicated that unpaved shoulders with narrow widths and steep slopes are highly sensitive to the demanding loading conditions imposed by wide SHLs with multi-axle trailer units, particularly under demanding environmental scenarios such as flooding conditions and heavy rain events. Consequently, stability analysis of sloped shoulders should be an integral component in risk management studies of pavement facilities servicing SHLs in overload corridors.

- The probabilistic slope stability analysis provided in this study highlighted the significance of shear strength properties of pavement unbound layers and their inherent variabilities for realistic assessment of the shoulders stability. Consequently, relying on conventional deterministic methods that overlook the intrinsic uncertainties in the analysis, in lieu of deploying probabilistic approaches, can be detrimental to the accuracy of the analysis of pavement shoulders subjected to SHLs with demanding loading conditions.

- The buried utility risk analysis showed that shallow-buried utilities consisted of flexible pipes with small diameters and thin walls that are longitudinally installed under pavement structure with “minimal to no” transverse offset from the load centerline can be damaged under SHL units that carry extremely heavy tires. Accordingly, it is necessary to place such underground utilities with an appropriate distance from SHL routes and at a sufficient depth of cover to mitigate the risk against failure of buried utilities. The sensitivity results suggested a minimum depth of cover as 4 ft to satisfy the maximum allowable deflection limit of buried pipes under SHL vehicle movement.
The results of this study underscored the significance of super-heavy trailer operations and their detrimental effects on the structure, longevity, and stability of pavement facilities in several US Highways, State Highways, and FM roadways throughout the OW transportation networks. Accurate quantification of such detrimental impacts is the prelude to adopting commensurate strategies to preserve the existing transportation facilities in overload corridors. In line with this necessity, a series of synthesized color-coded plots were developed in this research to provide a mechanistic means for the quantification of the SHL detrimental impacts. The devised risk management heat maps can be further instrumental by stakeholders/state DOTs for load restriction decision making or adopting proper mitigation strategies, which result in provision of safe traffic passage, as well as preservation of transportation infrastructure, under demanding loading/environmental scenarios.

12.2. RECOMMENDATIONS FOR FUTURE WORK

The author suggests the following items to be considered as potential research areas for future studies associated with the analysis and design of transportation infrastructures subjected to non-conventional SHL vehicles.

- The structural impact of SHL vehicles on the bridges can be included in the future researches. It is worth evaluating how the passages of super heavy trucks with GVW exceeding million pounds can be detrimental to the structural integrity of bridges. This is of great importance, since the performance of bridge structures depends not only on the axle load magnitudes of the passing vehicles, but also more importantly on the GVW.

- As a continuation of this research, future studies can deploy the analysis frameworks devised in this dissertation for further implementation in representative pavement sections across the overload corridors and energy sector regions such as Barnette Shale and Permian
Basin. The results can be helpful in having a broader insight on the structural impacts of SHL vehicles in different transportation networks.

- The mechanistic algorithms developed in this study are generic in nature. However, the robustness and accuracy of the analysis highly depend on proper characterization of the site-specific traffic data and determination of pavement layer properties in the studied network. Hence, by obtaining realistic data, the developed protocols can be used by other pavement design practitioners or other state highway agencies across the nation to address the existing concerns associated with the pavement deterioration due to heavy truck traffic movements in the US.
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Appendix A: Visual Inspection Results associated with each Site

Detailed information pertaining to the visual inspection conducted on the selected sites is provided in this section.

*Laredo District – Farm-to-Market (FM) 468*

The first visual inspection survey was conducted on Farm-to-Market (FM) 468 in LaSalle County of the Laredo District. Upon initial inspection, FM 468 appeared to be in a severely distressed state exhibiting multiple distress types. The first distress present on both K1 and K6 lanes was severe flushing on both right and left wheel paths throughout the entire section, as shown in Figure A.1. The flushing rated 3, according to TxDOT’s 2018 PMIS Pavement Rater’s Manual, begins in the intersection with FM 469 and continues past the Dimmit County line. The second documented distress was shallow to severe rutting throughout most of the inspected section. The rutting illustrated in Figure A.2, ranged from .25in to .50in in both wheel paths and covered approximately 70% of the wheel paths area. Moreover, an area with a large patch appearing to be milled out and replaced, as shown in Figure A.3, was also reported. However, the patch already seems to be disintegrating. Alligator cracking is also developing in the wheel paths covering an area of approximately 30%. In addition, raveling and potholes were also spotted in certain places of the inspected area. Heavily loaded vehicles coupled with a thin asphalt layer are believed to be the reasons for the many distresses present and the poor ride quality of FM 468.
Figure A.1: Severe Flushing in FM 468 - La Salle County.

Figure A.2: Shallow Rutting in FM 468 - La Salle County.

Figure A.3: Patches in FM 468 - La Salle County.
**Laredo District – US Highway 83**

US highway 83 displayed multiple distress types; raveling among the most significant. The raveling rated 2 was present throughout most of the inspected section, particularly in between the right and left wheel paths and towards the right shoulder. As illustrated in Figure A.4 and highlighted by the red oval one can identify areas where a significant amount of aggregate is missing. Shallow rutting was also reported in the inspected section ranging from .25in to .313in and covering approximately 40% of the wheel paths area. Additionally, as illustrated in Figure A.5, flushing rated 2 can be spotted in certain parts of the section along with alligator cracking.

![Figure A.4: Raveling and Missing Aggregate in US 83 -Dimmit County.](image-url)
San Antonio District – State Highway (SH) 16

In State Highway (SH) 16 shallow and deep rutting conditions in both wheel paths were observed throughout the entire pavement section. The rutting ranged from .25in to .625in and covered approximately 90% of the wheel paths area. Severe flushing rated 3, was also present on both K1 and K6 lanes and both right and left wheel paths, as shown in Figure A.6. Additionally, the raveling rated 2, can be spotted primarily on the right wheel path and towards the right shoulder, illustrated in Figure A.7. As of 2019, work to reconstruct SH 16 between Tilden and Jourdanton is under progress.
Figure A.7: Raveling and Rutting in SH 16-Atascosa County.

San Antonio District – Farm-to-Market (FM) 99

FM 99 also appeared to be in a poor and distressed condition exhibiting shallow rutting, flushing, and pothole formation. Shallow to severe rutting is present throughout the entire inspected section ranging from .25in to .50in. in both wheel paths and covering approximately 65% of the wheel paths area, as shown in Figure A.8. Flushing was also detected on both K1 and K6 lanes and rated at a level 2. Nonetheless, as illustrated in Figure A.9, the most significant distress was the severity of the potholes present in FM 99. Several large potholes were documented in the small inspection area however, while driving on the rest of FM 99 the inspectors encountered numerous potholes and failures. Heavily overloaded vehicles have adversely damaged much of FM 99 and have compromised its structural integrity.
Similarly, FM 624 was also in a distressed condition exhibiting shallow to deep rutting, flushing, and pothole formation. The research team documented shallow and deep rutting throughout most of the inspected section ranging from .25in to .625in in both wheel paths and covering approximately 60% of the wheel paths area as shown in Figure A.10. Moreover, severe flushing was also documented on both K1 and K6 lanes and rated at a level 3. As illustrated in Figure A.11, several large potholes were also identified and logged.
Corpus Christi District – US Highway 281

US 281 also exhibited multiple distresses such as shallow rutting, alligator cracking, and longitudinal cracks. Shallow rutting was reported through most of the inspected section ranging from .25 in to .375 in. in both wheel paths and covering approximately 40% of the wheel paths area as illustrated in Figure A.12. Alligator cracking was also documented primarily on the right wheel path and covering approximately 30% of the wheel paths area. However, the most
significant distress was the sealed and non-sealed longitudinal cracks present throughout the R1 lane and covering an approximate area of 30%. Additionally, the longitudinal cracking has led the right shoulder to separate and begin to sink in. Figures A.13 and A.14 respectively, show the severity of the longitudinal cracks present.

Figure A.12: Rutting in US 281 – Live Oak County.

Figure A.13: Longitudinal Cracks in US 281 – Live Oak County.
Corpus Christi District – State Highway (SH) 72

Upon inspection, SH 72 appeared to be in a fair condition as illustrated in Figure A.15. Since its recent reconstruction in 2013-2014, the only observable distresses that have initiated are shallow rutting and moderate flushing. Moreover, if the current OW traffic persists, SH 72 will soon start to develop more severe rutting.

Figure A.15: Pavement Conditions of SH 72 – Karnes County.
Corpus Christi District – BU 181/SH 123

Similarly, to the other highways, the pavement distresses documented in BU 181/SH 123 were also shallow rutting and flushing. The research team logged shallow rutting throughout most of the inspected section ranging from .25in to .375in primarily on the right wheel path and covering approximately 50% of the wheel paths area as shown in Figure A.16. Flushing was also spotted as illustrated in Figure A.17 and rated at a level 2.

Figure A.16: Shallow Rutting in BU 181/SH 123 – Karnes County.

Figure A.17: Flushing in BU 181/SH 123 – Karnes County.
Yoakum District – SH 119

SH 119 in the Yoakum District appeared to be in a structurally sound and fair condition as shown in Figure A.18. Since its recent lane widening, the only distress that has started to develop is shallow rutting. Despite drilling operations and pipe line construction, SH 119 has remained in an adequate condition.

Figure A.18: Pavement Conditions of SH 119 - Dewitt County.

Yoakum District – US 183

The final visual inspection survey was conducted on US highway 183, the major pavement distress documented was the seal and non-sealed longitudinal cracking throughout both K6 and K1 lanes, illustrated in Figure A.19 and A.20 respectively. The longitudinal cracks covered an approximate area of 95%. In addition, shallow rutting was also recorded ranging from .25in to .375in and covering approximately 30% of the wheel paths area.
Figure A.19: Pavement Conditions of US 183 - Gonzales County.

Figure A.20: Longitudinal Cracks of US 183 - Gonzales County.
Curriculum Vita

Ali Morovatdar completed graduate work at the University of Tehran in transportation and roadway engineering in 2016. Besides completing his master’s studies, he began working as a supervisor civil engineer with one of the most well-known engineering consulting firms to gain practical skills. In June 2018, he joined the doctoral program in Civil Engineering at the University of Texas at El Paso. Obtaining a GPA of 4/4, he has been honored to award the Anita Mochen Loya Scholarship for two consecutive years in 2018 & 2019. During his PhD studies, he has being actively involved in a number of research projects funded by FHWA through TxDOT emphasizing on transportation infrastructures such as Roadways and Highway Pavements. Ali Morovatdar is a Standing Member of the Highway Construction Committee (HCC) of ASCE - T&D, and an active research member of the Center for Transportation Infrastructure Systems and the Transportation Leadership Council. He has been privileged to serve as a teacher assistant and co-instructor to deliver undergraduate and graduate courses. He has contributed to several scholarly publications; including book chapters, project technical reports, conference proceedings, and journal articles published by TRR, ASCE, Elsevier, Springer, Taylor & Francis, and FHWA. He is an active peer reviewer for several prestigious journals. He has also presented at several international conferences around the world such as TRB, Geo-Congress, ASCE ICTD conferences, GAP 2019, AM3P 2020, Geo-China 2021, and BCRRA 2022. Recently, he has been honored to serve the ASCE Pavements 2021 Conference as a moderator to deliver technical sessions. His research interests include Mechanistic-Empirical (ME) pavement design and analysis, characterization of ME traffic data, analysis of OW and Super Heavy Load (SHL) vehicles, non-destructive evaluation of transportation infrastructures using field testing, geotechnical engineering, finite element analysis, and application of statistical analysis in pavement engineering.

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