# Investigating the Impacts of Truck Platooning on Transportation Infrastructure in the South Central Region 

Project No. 19PITSLSU14<br>Lead University: Louisiana State University<br>Collaborative Universities: University of Texas at San Antonio; Texas A \& M University

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| APPROXIMATE CONVERSIONS TO SI UNITS |  |  |  |  |
| Symbol | When You Know | Multiply By | To Find | Symbol |
| LENGTH |  |  |  |  |
| in | inches | 25.4 | millimeters | mm |
| ft | feet | 0.305 | meters | m |
| yd | yards | 0.914 | meters | m |
| mi | miles | 1.61 | kilometers | km |
| AREA |  |  |  |  |
| $\mathrm{in}^{2}$ | square inches | 645.2 | square millimeters | $\mathrm{mm}^{2}$ |
| $\mathrm{ft}^{2}$ | square feet | 0.093 | square meters | $\mathrm{m}^{2}$ |
| $y d^{2}$ | square yard | 0.836 | square meters | $\mathrm{m}^{2}$ |
| $\mathrm{ac}^{2}$ | acres | 0.405 | hectares |  |
| $m i^{2}$ | square miles | 2.59 | square kilometers | $\mathrm{km}^{2}$ |
|  |  | VOLUME |  |  |
| fl oz | fluid ounces | 29.57 | milliliters | mL |
| gal | gallons | $3.785$ | liters | L |
| $\mathrm{ft}^{3}$ | cubic feet | $0.028$ | cubic meters | $\mathrm{m}^{3}$ |
| $y d^{3}$ | cubic yards | $0.765$ | cubic meters | $\mathrm{m}^{3}$ |
| NOTE: volumes greater than 1000 L shall be shown in $\mathrm{m}^{3}$ |  |  |  |  |
| MASS |  |  |  |  |
| oz | ounces | 28.35 | grams | g |
| lb | pounds | 0.454 | kilograms | kg |
| T | short tons (2000 lb) | 0.907 | megagrams (or "metric ton") | Mg (or "t") |
| TEMPERATURE (exact degrees) |  |  |  |  |
| ${ }^{\circ} \mathrm{F}$ | Fahrenheit | $5(\mathrm{~F}-32) / 9$ or (F-32)/1.8 | Celsius | ${ }^{\circ} \mathrm{C}$ |
| ILLUMINATION |  |  |  |  |
| fc | foot-candles | 10.76 | lux |  |
| fl | foot-Lamberts | 3.426 | candela/m ${ }^{2}$ | $\mathrm{cd} / \mathrm{m}^{2}$ |
| FORCE and PRESSURE or STRESS |  |  |  |  |
| lbf | poundforce | $4.45$ | newtons | N |
| $\mathrm{lbf} / \mathrm{in}^{2}$ | poundforce per square inch | 6.89 | kilopascals | $\mathrm{kPa}$ |
| APPROXIMATE CONVERSIONS FROM SI UNITS |  |  |  |  |
| Symbol | When You Know | Multiply By | To Find | Symbol |
| LENGTH |  |  |  |  |
| mm | millimeters | 0.039 | inches | in |
| m | meters | 3.28 | feet | ft |
| m | meters | 1.09 | yards | yd |
| km | kilometers | 0.621 | miles | mi |
| AREA |  |  |  |  |
| $\mathrm{mm}^{2}$ | square millimeters | 0.0016 | square inches |  |
| m | square meters | 10.764 | square feet | $\mathrm{ft}^{2}$ |
| m | square meters | 1.195 | square yards | $\mathrm{yd}^{2}$ |
| ha | hectares | 2.47 | acres | ac |
| km | square kilometers | 0.386 | square miles | $m i^{2}$ |
| VOLUME |  |  |  |  |
| mL | milliliters | 0.034 | fluid ounces | fl oz |
| L | liters | 0.264 | gallons | gal |
| m | cubic meters | 35.314 | cubic feet | $\mathrm{ft}^{3}$ |
| m | cubic meters | 1.307 | cubic yards | $y d^{3}$ |
| MASS |  |  |  |  |
| g | grams | 0.035 | ounces | oz |
| kg | kilograms | 2.202 | pounds | lb |
| Mg (or "t") | megagrams (or "metric ton") | 1.103 | short tons (2000 lb) | T |
| TEMPERATURE (exact degrees) |  |  |  |  |
| C | Celsius | 1.8C+32 | Fahrenheit | ${ }^{\circ} \mathrm{F}$ |
| ILLUMINATION |  |  |  |  |
|  |  | $0.0929$ | foot-candles | fc |
| $\mathrm{cd} / \mathrm{m}^{2}$ | candela/m² | 0.2919 | foot-Lamberts | $\mathrm{fl}$ |
| FORCE and PRESSURE or STRESS |  |  |  |  |
| N | newtons | 0.225 | poundforce | lbf |
| kPa | kilopascals | 0.145 | poundforce per square inch | $\mathrm{lbf} / \mathrm{in}^{2}$ |

## TABLE OF CONTENTS

TECHNICAL DOCUMENTATION PAGE ..... ii
TABLE OF CONTENTS ..... iv
LIST OF FIGURES ..... v
LIST OF TABLES ..... viii
ACRONYMS, ABBREVIATIONS, AND SYMBOLS ..... ix

1. INTRODUCTION ..... 1
2. OBJECTIVES ..... 4
3. LITERATURE REVIEW ..... 5
4. METHODOLOGY ..... 15
5. ANALYSIS AND FINDINGS ..... 40
6. CONCLUSIONS ..... 75
REFERENCES ..... 77

## LIST OF FIGURES

Figure 1. Total Domestic Freight Flows, 2012 - 2045 (1). ..... 1
Figure 2. Total Real Values of Domestic Freight Flows, 2012 - 2045 (1). ..... 1
Figure 3. Truck flows on the US National Highway System in 2016 (2) ..... 2
Figure 4. Truck flows forecasting on the US National Highway System in 2045 (2). ..... 2
Figure 5. Overall project methodology. ..... 4
Figure 7. Vehicle detection and tracking in aerial images ..... 20
Figure 8. Time-space diagram of the vehicles for ten minutes on the Southbound of I- 35 between Exit 237B and 238A, Austin, TX, during the morning peak time. ..... 22
Figure 9. Maximum longitudinal and transverse strain for different vehicle speeds. ..... 24
Figure 10. The simulated FEM model and meshing (61). ..... 26
Figure 11. Deflection measurements by the MDD embedded in various depth of the pavement (56) ..... 26
Figure 12. Effect of speed on deflection of different pavement (56) ..... 27
Figure 13. Sigmoidal function parameters effect (63). ..... 28
Figure 14. Increase of DSR function at a temperature and shifted temperature (66) ..... 29
Figure 15. The developed master curve using WLF (67). ..... 30
Figure 16. Schematic layout of the study ..... 31
Figure 17. Pavement seasonal temperature data for $\mathrm{IH}-35$. ..... 33
Figure 18. Developed master curve for the rich bottom layer porous friction coarse (PFC), stone matrix asphalt (SMA), stone-filled hot mix asphalt (SFHMA), SFHMA rut-resistance layer (RRL), and rich-bottom layer (RBL) ..... 35
Figure 19. Model development flowchart and critical points to assess rutting and fatigue damage. ..... 36
Figure 20. Tandem axle configuration and inflation pressure. ..... 37
Figure 21. Percentage change result visualization for Peak and Off-Peak hour ..... 42
Figure 22. Scenario Visualization (Peak hour flow) ..... 43
Figure 23. Scenario Visualization (Off-Peak hour flow). ..... 45
Figure 24. Time-space diagram of the vehicles for ten minutes on the Southbound of I-35, Austin, TX, during the morning peak period. ..... 46
Figure 25. Trajectory data example of ACC vehicles and their followers ..... 48
Figure 26. Flow-density plots for each lane with aggregation resolution of 30 seconds ..... 49
Figure 27. Flow-density plots for different platoon sizes. ..... 50
Figure 28. Flow-density plots for different market penetration rates of autonomous trucks (3-truck platoons) ..... 52
Figure 29. Tridem axle simulation using ABAQUS software. ..... 53
Figure 30. The field-measured deflections for the test truck. ..... 54
Figure 31. PFC layer surface deflection induced by the 34 kips tandem axle. ..... 54
Figure 32. Subbase layer surface deflection induced by the 34 kips tandem axle. ..... 55
Figure 33. PFC layer surface deflection induced by the 10.5 kips single axle (steering) ..... 55
Figure 34. Subbase layer surface deflection induced by the 10.5 kips single axle (steering). ..... 55
Figure 35. Tridem axle-loads distribution (the overweight axle-load frequencies are shown in red). ..... 56
Figure 36. Wandering considerations using 5 points (86). ..... 58
Figure 37. Dynamic loading and axle configuration for tandem axle. ..... 59
Figure 38. Lateral strain for the 40 kips tandem axle. ..... 59
Figure 39. Maximum principal tensile strain for a 40 kips tandem axle. ..... 60
Figure 40 . Vertical strain for the 63 kips tridem axle. ..... 60
Figure 41. Maximum principal tensile strain for a 63 kips tridem axle. ..... 60
Figure 42. Maximum principal strain at the bottom of SFHMA1' layer. ..... 61
Figure 43. Static loading and axle configuration for tridem axle. ..... 62
Figure 44. Vertical strain (LE22) on SFHMA1/2' for tridem axle. ..... 62
Figure 45. Vertical strain at the SFHMA1' layer surface for tridem axle. ..... 62
Figure 46. Vertical strain at the RBL layer surface for tridem axle. ..... 63
Figure 47. Vertical strain (LE22) at Subbase surface for tridem axle. ..... 63
Figure 48. Vertical strain (LE22) at subgrade surface for tridem axle. ..... 63
Figure 49. Maximum vertical strain at mid-depth of PFC, SMA, SFHMA1/2', SFHMA1', and RBL. ..... 64
Figure 50. The relative permanent deformation damage based on the normal distribution wandering for a tandem axle. ..... 64
Figure 51. Cost comparison of peak scenarios (corridor level analysis). ..... 70
Figure 52. Cost comparison of off-peak scenarios. ..... 71

Figure 53. Maintenance strategies for the normal wandering (top) and fixed-path wandering (bottom). .................................................................................................................................. 72

Figure 54. Reconstruction strategy based on pavement condition for A) Fatigue distress and B) Permanent deformation............................................................................................................ 73
LIST OF TABLES
Table 1. Scenario design result. ..... 17
Table 2. Statistical comparison among shifting methods conducted by Forough et al. (62). ..... 30
Table 3. The analyzed traffic data for different axle types. ..... 32
Table 4. Typical section of PP constructed in Texas (68). ..... 33
Table 5. The selected section for the study (The Texas PP Database, Texas Transportation Institute) (68). ..... 34
Table 6. Mix design specifications of the selected section (68). ..... 34
Table 7. Specifications of the static and the dynamic model. ..... 37
Table 8. Result of Simulation Scenario's (Peak hour) ..... 40
Table 9. Percent change from reference scenario (Peak hour). ..... 40
Table 10. Percent change from reference scenario (Off-Peak hour). ..... 41
Table 11. Percent change from reference scenario (Off-Peak hour). ..... 42
Table 12. Analysis levels for each variable. ..... 53
Table 13. Overloading representative axle loads and their frequencies. ..... 56
Table 14. Fatigue life ratio (FatigueLifefixed-path/FatigueLifenormal-distribution, 20-year) for single, tandem, and tridem. ..... 61
Table 15. Permanent deformation damage-ratio range for AC layers. ..... 65
Table 16. Anticipated service life for fixed-path truck platooning. ..... 65
Table 17. Estimation of crash cost per event by severity. ..... 68
Table 18. The relative percentage of crash severities. ..... 68
Table 19. Total cost of truck platooning impacts for peak scenarios at corridor level analysis. ..... 69
Table 20. Total cost of truck platooning impacts for off-peak scenarios at corridor level analysis.71
Table 21. Present cost for strain-value-based and distress-based approaches per mile over a 50-year life cycle costs.74

| ACRON | IATIONS, AND SYMBOLS |
| :---: | :---: |
| AADT | Annual Average Daily Traffic |
| ACC | Adaptive Cruise Control |
| AUC | Area Under the ROC Curve |
| CACC | Cooperative Adaptive Cruise Control |
| CAV | Connected and Automated Vehicle |
| $\mathrm{CO}_{2}$ | Carbon Dioxide |
| CPI | Consumer Price Index |
| CPI | Crash Potential Index |
| CRPC | Capital Region Planning Commission |
| DRAC | Deceleration Rate to Avoid a Crash |
| DSRC | Dedicated Short Range Communication |
| ECI | Employment Cost Index |
| FE | Finite Element |
| GEH | Geoffrey E. Havers Statistics |
| HDV | Heavy-duty Vehicles |
| HOV | High Occupancy Vehicle |
| IDM | Intelligent Driver Model |
| IRSA | Integrated Full-Range Speed Assistant |
| LMRS | Lane change Model with Relaxation and Synchronization |
| MPE | Mean Percentage Error |
| MPR | Market Penetration Rate |
| NGSIM | Next Generation Simulation |
| $\mathrm{NO}_{x}$ | Nitrogen Oxides |
| NPMRDS | National Performance Management Research Data Set |
| PATH | California Partners for Advanced Transit and Highways |
| PDO | Property Damage Only |
| $\mathrm{PM}_{10}$ | Particulate Matter |
| RMSPE | Root Mean Square Prediction Error |
| RRMSE | Relative Root Mean Square Error |

TTC
TET
TIT
TND
TTD
TTM
V2V
$\Pi$

Time to Collision
Time Exposed Time-to-collision
Time Integrated Time-to-collision
Total Network Delay
Total Time to Diverge
Total Time to Merge
Vehicle-to-Vehicle
Crash-Conflict Ratio

## EXECUTIVE SUMMARY

Freight and the efficient movement of freight is a critical component to the economy of the southern U.S. - particularly to states in Region 6 (AR, LA, NM, OK, and TX). Several of the largest freight distribution hubs (e.g., Houston, TX, New Orleans, LA, Oklahoma City, OK, and DallasFort Worth, TX) and most valuable truck corridors in the U.S. are located in this region. However, several challenges exist in supporting a strong freight economy: (1) efficiency of freight movement muffled by an infrastructure system in need of repair, limited capacity, and severe congestion and (2) mitigating negative community impacts.

Connected and automated vehicle (CAV) technologies offer potentially transformative societal impacts including significant mobility, safety, and environmental benefits. One CAV application of particular interest to the freight industry is truck platooning. Truck platooning describes as number of trucks equipped with CAV technology that closely follow one another in a "platoon". Benefits of truck platooning include energy savings from aerodynamic drag reduction, reduced highway congestion due to short following distances, and safety improvements from faster reaction times and automated support systems. However, the short following distances maintained between vehicles and more precise lane-keeping lead to a higher concentration of load being placed on the transportation infrastructure. It is unclear how these greater weight concentrations and new load configurations will impact the deterioration/damage to pavements. Addressing this uncertainty is critical, especially considering the current state of severe financial constraints in which not all state-owned infrastructures can be maintained.

The main objectives of this study are: (1) through a series of modeling case studies located in Region 6, the operational, environmental (fuel savings, and emissions), and safety impacts of various truck platooning implementations, configurations, and assumptions were quantified at both the corridor- and network-level, (2) impacts to the structural pavement resulting from these truck platooning implementations were investigated and quantified using finite element modeling (FEM), and (3) a feasibility study for implementation was performed comparing the (potential) operational, environmental (fuel savings, and emissions), and safety benefits of truck platooning with the (potential) cost associated with increased pavement loads. This was also compared with an equivalent "base case" with human-driven trucks.

The microsimulation corridor-level modeling was performed using python and Vissim software. The study area was selected from the I-10 freeway and the network was prepared in Vissim. All the designed scenarios were run with 5 different seeds to get the value of the performance indicators. An economic study was also performed to estimate the total cost of each scenario. The result showed that the truck platooning improves the operation, environment, and safety aspect of traffic in the off-peak hour. It has a deteriorating impact on traffic operation and safety in the peakhour if the truck platoon contains more than two trucks.

In addition to the microscopic analysis, a large-scale analysis of the impacts of truck platooning on congestion and traffic flow dynamics is conducted. Accordingly, a simulation model of I-35 is developed. The model was calibrated based on the trajectory data collected from a segment of I35 near Austin, TX. The findings show that as the size of platoon decreases, the scatter in fundamental diagram decreases and the traffic flow becomes more stable. Moreover, with a fixed platoon size, higher penetration rate of autonomous trucks results in smoother traffic and less scatter in fundamental diagram.

Finally, the impact of truck platooning on pavement were also addressed using the elastic and dynamic-viscoelastic finite element method (FEM) models. The mechanical response obtained from the simulations are implemented to predict the effects of platooning due to limited wandering (lateral movement of truck tires). Based on the results, it can be concluded that wandering pattern can have influential effect on the fatigue life and permanent deformation damage. Economic analysis shows that the fixed-path platooning can significantly increase the constructionmaintenance cost of the pavement.

Results of the case studies may inform CAV-related policy, planning, and integration strategies including changes to asset management and maintenance procedures.

## 1. INTRODUCTION

It is very critical to provide efficient and safe movement of freight for the economy of US states especially to the states in region 6 (Arkansas, Louisiana, New Mexico, Oklahoma, \& Texas) where there are several largest freight distribution hubs (e.g., Houston, New Orleans, Oklahoma City, and Dallas-Fort Worth). Accordingly, most valuable truck corridors in the U.S. are in this region.

According to a freight Flow Forecast Study that was published by FHWA in 2016, total domestic freight flow tonnage will grow by 6.0 billion tons over the forecast period from 2012 to 2045. This represents an increase of over 40 percent (as shown in Figure 1). Also, over the entire period of this forecast, the total real values of domestic freight flows will grow from $\$ 14.1$ trillion in 2012 to $\$ 22.5$ trillion in 2045, representing about $60 \%$ percent increase (as shown in Figure 2) (1).


Figure 1. Total Domestic Freight Flows, 2012 - 2045 (1).


Figure 2. Total Real Values of Domestic Freight Flows, 2012 - 2045 (1).

In addition, Figures 3 and 4 show truck flows on the US national highway system in 2012 and their forecast for the year 2045. Clearly, there will be a significant increase of the truck flows on the US national Highway system over the coming 25 years (2).


Figure 3. Truck flows on the US National Highway System in 2016 (2).


Figure 4. Truck flows forecasting on the US National Highway System in 2045 (2).

Unfortunately, there are several challenges that affect the efficiency of freight movement including: high fuel and labor costs, vehicular emissions, and traffic safety problems resulting from trucks drivers' errors (e.g., due to the need to drive for long distances and extended time).
Fortunately, emerging vehicle technology such as Connected and Autonomous Vehicle (CAVs) can help in minimizing these challenges. One CAV application of particularly interest to the freight industry is truck platooning. Truck platooning consists a number of trucks equipped with CAV technology that closely follow one another in a "platoon". Recent studies indicated that truck platooning may result in drastic changes in freight shipment operation and will have a great potential in addressing current challenges facing freight movement.
It is expected that truck platooning can help in reducing fuel consumption, emissions, labor costs. Also, it may provide traffic safety and traffic flow improvements. On the other hand, truck platooning may accelerate the pavement damage due to its greater weight concentrations.
Considering the literature review, very little studies concentrated on the safety aspect of truck platooning as well as impacts on Pavements. Indeed, there are several research issues that need to be considered, such as the effects of shorter headways on capacity or improved traffic flow stability, and its potential negative effects such as blocking at ramps. So, in our analysis, we will give a special focus on this safety aspect and impact on the pavement to fill this gap.

## 2. OBJECTIVES

The objectives of our project can be summarized as follows:

1. Examine the operational and environmental impacts of truck platooning on US highways. A series of modeling case studies located in Region 6 were developed using Vissim at both the corridor- and network-level.
2. Explore the impact of truck platooning on pavement. finite element (FE) modeling was used to quantify the impact on pavement
3. Conduct feasibility study and recommendations. An economic analysis was conducted comparing the (potential) operational and environmental benefits of truck platooning with the (potential) increases in maintenance costs associated with the limited tire wandering.

Figure 5 illustrates the overall methodology used in this study to achieve the abovementioned objectives. Each of these tasks will be explained in detail in the following sections.

Task 1: Stakeholder Engagement

Task 2: Literature Review

Task 3: Operational and Environmental Analysis (Corridor-level analysis)

## Task 4: Operational and Environmental Analysis (Network-level analysis)

Task 5: Pavement Analysis

Task 6: Feasibility Study and Recommendations

Figure 5. Overall project methodology.

## 3. LITERATURE REVIEW

### 3.1. Connected and Autonomous Vehicle

Calvert et al. investigated the impact of automated vehicles on traffic flow and capacity, especially during the transition period, through literature review and experimental analysis (3). They found that low-level automated vehicles in mixed traffic had a small negative impact on traffic flow and road capacity. The main reason behind the reduction was higher gap times maintained by automated vehicles. The influence of decreased lane changes did not show significant effects. Experiments showed that a positive effect was observed only when the penetration rate was above $70 \%$. The author argued that current knowledge and model development still lacking about appropriately capturing much of real driving behavior, especially in a lateral sense.

Rios-Torres \& Malikopoulos investigated the impact of Connected and Automated Vehicles (CAVs) on traffic flow at merging sections and developed a microscopic simulation framework to explore the implications on fuel consumption and travel time (4). The simulation study was done using two scenarios ( $0 \%$ and $100 \% \mathrm{mpr}$ ) and three-traffic flow conditions (low, medium, and congested). The result of the baseline case ( $0 \% \mathrm{mpr}$ ) showed a linear trend in free traffic conditions but scattered for the congested condition. Optimal coordination of CAVs significantly reduced the variations in traffic flow and density for different traffic conditions. Therefore, full penetration of CAVs contributed to more stable traffic patterns even when the traffic density was high. Also, the overall fuel consumption reduced by $100 \%$ CAV scenario in all traffic conditions, though the highest was for medium traffic. Travel time reduction was very close to low traffic in both scenarios but varied greatly for medium and high traffic conditions.

Martin-Gasulla et al. proposed a strategy for overcoming the potential deterioration of the performance measured when CAVs were introduced in the traffic stream at their early stages (5). The proposed strategy accounted for a dedicated lane for CAVs and preemption green time to sort CAVs for platooning. Empirical data were collected to calibrate the simulation model. The model showed that performance measures deteriorated with the introduction of CAVs and improvement was possible with platooning. Then the proposed strategy scenarios were implemented in the model. The maximum throughput achieved by CAVs seems promising for high penetration rates ( $44 \%$ more vehicles for $100 \%$ penetration rate). However, the coexistence with human-driven vehicles prevents them from achieving medium-to moderate improvements for low penetration rates, even reducing the capacity of the infrastructure. The proposed strategy effectively improved throughput and controls delay for most of the simulated scenarios, overcoming the potential loss in throughput for the first stages of CAV introduction into the traffic stream.

Moridpour et al. examined the differences in traffic characteristics in the vicinity of heavy vehicles and passenger cars (6). The trajectory dataset used in this study was made available by Cambridge Systematics Incorporated for the Federal Highway Administration (FHWA) as part of Next Generation Simulation (NGSIM) project. The section of I-80 was 503 m long and comprised of five main lanes with one auxiliary lane, one on-ramp, and one exit off-ramp. There were no lane restrictions for heavy vehicles. To better understand the influence of heavy vehicles on their surrounding traffic, they were classified into two separate classes based on their length. Heavy vehicles with a length of equal to or greater than 15 m were classified as heavy trucks, and those with a length of less than 15 m were considered as light trucks. The results showed larger front space gaps for heavy trucks compared with the front space gaps for light trucks and passenger cars.

The existence of larger front space gaps may be due to the limitations in the maneuverability of heavy trucks. Light trucks and passenger cars had an almost similar influence on their surrounding traffic characteristics, and therefore, the influence of heavy trucks on surrounding traffic was examined. The average travel times increased when the proportion of heavy trucks increased in each lane. The increase in the proportion of heavy vehicles increased the average travel times particularly in higher traffic densities and a larger proportion of heavy trucks. Furthermore, the number of passenger car lane-changing maneuvers per lane increased when more heavy trucks existed in that lane. The influence of heavy trucks on the number of passenger car lane-changing maneuvers was intensified in higher traffic densities and a larger percentage of heavy trucks. An increase in the proportion of heavy trucks intensified the likelihood of accidents and therefore reduced traffic safety. The existence of a larger percentage of heavy trucks could increase the likelihood of accidents and reduce traffic safety. To reduce travel time and improve traffic safety in highways/freeways, heavy trucks could be restricted from particular lanes especially during congestion.

### 3.2. CACC development and impact:

Schakel et al. investigated the effect of Cooperative Adaptive Cruise Control (CACC) on traffic flow stability through assessing the shockwave dynamics (7). A micro-simulation study was conducted with IDM + car-following model and CACC with varying penetration rates. An Integrated full-Range Speed Assistant (IRSA) controller and Acceleration Advice Controller (AAC) algorithm were implemented for the CACC system. The result was compared with the A270 experiment. The AAC algorithm produced a downstream moving shockwave similar to the second shockwave observed in the A270 experiments but could not reproduce the first upstream wave. Other differences between simulated and field data might come from the variability of headways. The result showed improved traffic flow stability as shockwaves were quickly damped.

Bergenhem et al. studied a vehicle-to-vehicle communication system that enables vehicles to drive in a platoon (8). A platoon according to SARTRE was a manually controlled lead vehicle with a number of automatically controlled (both longitudinally and laterally) following vehicles. A Vehicle-to-Vehicle (V2V) communication system prototype was proposed and described. The main communication channel was based on IEEE 802.11 p which operates at 5.9 GHz . A software was developed to be used in the V2V system; whose function was to forward messages between vehicles. Measurements of the performance of the prototype V2V system were presented. The performance of the V2V system was affected by the Line of Sight. Two antenna placements on the leading vehicle (LV) were tested; in front of the driver cabin and the rear on top of the container. The rear placement displayed superior results, especially for distances above 70 meters. The result showed for the front antenna, the number of failed round trip messages was 1740 and the percent of the successful round-trip messages was $84.7 \%$, and for the rear antenna, the number of failed round trip messages was 422 and the percent of the successful round trip messages was $96.3 \%$. The result showed the advantage of using the rear antenna over the front was $151 \%$ for a distance greater than 100 meters.

Segata et al. investigated the inter-vehicle communication technologies needed for automated platooning (9). The author also discussed the resulting challenges for beaconing protocols. The platooning controller was adopted where the inputs to the system were the leaders and the front vehicle's speed and acceleration. The idea was to divide the time after a beacon from the leader into slots and had each vehicle send its beacon in the time slot corresponding to its position in the
platoon. This slotted beaconing protocol was referred to as SLB and SLBP (with and without transmit power control respectively). The protocol versions of this static beaconing approach, with and without transmitting power control, were being referred to as STB and STBP, respectively. Calibration and validation were done using real experiments with four cars driving on a private road. A two-phase simulation was done, where in the first phase no communications were allowed, and thus overhead due to network simulation was removed, speeding up the initialization phase. In the second phase, communications were enabled when the platoons were formed properly. To assess the usability of the different protocols for platooning, a new metric the safe time ratio was defined which could then be computed for a set of possible delay requirements. In all the scenarios STB and SLB completely saturated the channel as they did not employed transmit power control and hence caused a huge amount of collisions. In comparison, STBP and SLBP were able to avoid complete channel saturation and drastically reduced the number of collisions in the channel, suggesting that transmit power control can give a huge benefit. The slotted approach resulted in a better utilization with increased channel busy ratio and lower collisions.

Gao et al. studied the performance of Dedicated Short Range Communication (DSRC) Vehicle-to-Vehicle $(\mathrm{V} 2 \mathrm{~V})$ communication in truck platooning scenarios through real-world experiments (10). Two commercial semi-trailer trucks were used in the tests and DSRC devices were mounted on the trucks. GPS antenna was also mounted on the left side of the truck and was connected to the DSRC radio. A test software was developed to test the DSRC performance and was designed to collect different performance metrics while varying the design parameters. Two types of tests were used: Dynamic test and Static test. The main findings were: In the baseline scenario, DSRC achieved nearly $100 \%$ delivery ratio at all data rates with any message sizes and rates, even at a distance as long as 78 m ; When a truck was turning, the outside antenna might be blocked by its trailer which affected delivery ratio, but the inside antenna normally worked very well; While on a straight line with complex terrain nearby, the delivery ratio could be low, especially with large message size and high data rates. The delivery ratio could even be lower than at curves; if the road was hilly, trucks could be misaligned (not parallel) with each other, resulting in a lower delivery ratio. However, in some situations, complex terrains may generate reflections that could improve the delivery ratio and reduce the adverse effects of hilly roads; using both side antennas alternately could normally improve delivery ratio significantly since the best performing antenna determined it at any moment.

Xiao et al. (2020) investigated the traffic flow impacts of converting High Occupancy Vehicle (HOV) lanes into CACC lanes regarding CACC MPRs on a complex freeway corridor with multiple interacting bottlenecks (11). Simulation studies were conducted on the SR-99 corridor which was $20-\mathrm{km}$ long with 16 on-ramps, 12 off-ramps, and 1 HOV lane. CACC vehicles were introduced and the left-most HOV lane was converted into a dedicated CACC lane. An enhanced car-following model, IDM + , and an extended lane-changing model, LMRS, was used.

### 3.3. Vehicle Platooning

Alam et al. showed the fuel reduction potential of heavy-duty vehicle platooning and the influence of a control system on fuel consumption (12). A simulation model was set up to quantify the fuel reduction of platooning vehicles and separate the effect of the control system (ACC) on fuel consumption. The result showed that the ACC system did not increase the fuel consumption and overall fuel reduction achieved through platooning. Also, different masses of trucks played a different role in fuel reduction, where following a lighter vehicle reduced more fuel consumption
than following a heavier vehicle. An experiment was conducted with two heavy-duty vehicles to verify the simulation result. The experimental result also showed significant fuel reduction achieved through vehicle platooning.

Liang et al. investigated the problem of when it is beneficial for a heavy-duty vehicle to drive faster to catch up and join a platoon (13). The author had proposed a formula to calculate the fuel consumption, break-even ratio, and incentive factor, which indicated when a platoon catch-up attempt would be beneficial. An example was established with two heavy-duty vehicles (HDVs) of 40 t each started driving at the same time at $80 \mathrm{~km} / \mathrm{h}$ with a position difference of 10 km and they had the destination 340 km and 350 km apart respectively. The result showed an incentive factor of 0.17, which indicated that it was beneficial for the following vehicle to catch-up the lead vehicle and form a platoon rather than driving alone. The formula derived result was verified using a simulation model from Scania. The result showed a similar pattern with the simulation result. There were differences between the result from the formula and the simulation, but they were mainly due to the simpler assumptions taken for deriving the formula.

Sugimachi et al. developed a method for autonomous platooning that uses information acquired from the front and rear trucks by inter-vehicle communication (14). A comparison between longitudinal control methods was done using a simulation study. F model was referred to as the controller that utilized the distance from only the front vehicle where the FR model was referred to the controller that utilized the distance from both front and rear vehicle. FR model outperformed and ensured better string stability than the F model. The effectiveness of the FR control method was experimentally evaluated. The proposed method showed a precise performance that was required for autonomous platooning.

Liang et al. investigated the influence of traffic on merging maneuvers of heavy-duty vehicles trying to form a platoon (15). A simulation framework was used to simulate two heavy-duty vehicles merging maneuvers on a 50 km long straight two-lane road. Different speeds of the HDVs and different traffic densities were simulated and the distance it took for the vehicles to merge and form a platoon, as well as the average speeds both vehicles maintained, were measured. The result showed that for light traffic the merging point was delayed insignificantly. For medium traffic, the maximum increased merging distance was $46 \%$ compared to the ideal merging distance. For heavier traffic, the merging distances further increased, and the maximum increased value was $66 \%$ compared to the ideal case.

Lioris et al. presented a queueing model to predict the throughput change of an intersection due to vehicle platooning (16). A simulation confirmed the predicted result. A fluid dynamic model was developed and formalized to predict the capacity and throughput increase of an intersection with vehicle platooning. A mesoscopic simulation was done using PointQ software on a roadway network consisting of 16 intersections. With an infinite capacity queue, the network supported throughput increased by any factor, but with a finite capacity queue, the maximum throughput of the intersection decreased significantly. Cycle time may be reduced to reduce queue length and delay. However, in reality, cycle time reduction is not possible, but if platooning gives a gain in throughput, some of the gains may be used to reduce the cycle time, and hence reduce queue length and delay.

### 3.4. Truck platooning: field testing

Browand et al. investigated the field test result of two tandem trucks to evaluate the fuel reduction potential of platooning (17). Two tests were performed with various inter platoon spacing; with two trucks equipped with automatic transmissions to maintain a fixed inter platoon distance. The results of the first test were not reliable, as rain canceled most of the data taken on that day. To minimize the effect of runway slope and wind, platoons were run on both north and south bounds, and the results were averaged. The results were compared with a base case, and it showed fuel reduction achieved for both lead and trail trucks.

Tsugawa presented experiment and simulation results of automated truck platoon incorporated with configuration and sensing and control systems (18). Experiments were conducted with four trucks along an expressway equipped with DSRC for V2V communication. Both micro and macro simulations were performed to estimate aerodynamic drag reduction and $\mathrm{CO}_{2}$ emission, respectively. The control and sensing system performed well in all results. The experiment result showed reduced fuel consumption. The microsimulation result showed aerodynamic drag reduction and the macrosimulation result showed CO 2 reduction with a high penetration rate of platoons.
$\mathrm{Lu} \&$ Shladover investigated the result from field testing of a vehicle platooning system, considering the practical string stability for different maneuvering conditions (19). Three trucks were successfully platooned for different inter platoon gaps while equipped with Dedicated ShortRange Communication (DSRC) equipment for vehicle-to vehicle (V2V) communications. The DSRC communication system had sufficient capabilities to support V2V communication applications. A maximum of $14 \%$ fuel reduction was achieved in the third truck of the platoon. Energy-saving and reducing fuel consumption was the most important benefit of platooning.

### 3.5. Truck platooning: Simulation

Yeo et al. proposed a new behavioral algorithm with improved car-following and lane-changing dynamics for oversaturated freeway flow which showed very good results when comparing with the field data (20). The model was implemented on Aimsun. The model was validated with both vehicle trajectory data and macroscopic detector data. The result showed good agreement with real traffic data.

Schakel et al. proposed a new lane-change model, which can be integrated with a car-following model for microsimulation (21). They included both relaxation and synchronization phenomena in designing the lane change model. The model was calibrated and validated for both free flow and congested traffic. It was very accurate when representing lane distribution and the onset of congestion. Also, the model was easier to calibrate as it has only seven parameters.

Zhao et al. used an API interface of microscopic traffic simulator Vissim to construct the ACC and CACC simulation framework in mixed traffic (22). ACC and CACC equipped vehicles were considered and incorporated in the Vissim model. They simulated the model with platooning capabilities and found increased capacity with an increasing penetration rate of CACC platoons. The model was not calibrated, so the result was not reliable. However, it was the first step to incorporate different automated vehicles in the microsimulation study. The study only proposed the car-following model; however, the lane-changing model was missing.

Lu et al. calibrated a microsimulation model with field data on Aimsun and Vissim. I-66 eastbound section was selected as a freeway section for the simulation study (23). Field data were collected
using different sensors and compatibility of data was checked using flow conservation theory. Calibration criteria for the microsimulation model were GEH, RRMSE, accumulated flow, and fundamental diagrams. Simulation results showed that the two models matched the field data at critical locations reasonably well. Even with the extensive collected data, the calibration process was very laborious and required many iterations.

Lu et al. proposed an enhanced car-following lane-changing model which can incorporate connected automated vehicle for microsimulation study (24). The model was implemented on Aimsun. Loop detector data from a 13-mile long corridor was used to calibrate the model. The calibrated model accurately replicated the locations and throughputs of the freeway bottlenecks. Also, the simulated flows satisfied the calibration criteria of GEH distribution.

Xiao et al. developed a realistic and collision-free car-following model for Adaptive cruise control (ACC)-Cooperative adaptive cruise control (CACC) vehicles and tested the model in a wide range of scenarios to explore model performance and collision possibilities (25). The model assumed that a human driver resumes vehicle control either according to his or her assessment or after a collision warning asks the driver to take over. The empirical ACC-CACC car-following models presented by Milanés and Shladover were used for the simulation study. However, these models could not achieve collision-free operation in the full speed range, but the proposed multi-regime car-following models for ACC and CACC systems extended the empirical ACC-CACC models with human intervention. An advanced version of the Intelligent Driver Model, IDM + , was used as the car-following model in the loop of human control. Five representative traffic scenarios were simulated: stop and go, hard brake, cut-in, cut-out, and approaching. The result of the simulation study showed that the proposed models were collision-free under typical traffic situations and most safety-critical scenarios.

Ramezani et al. expanded their previous studies on car-following and lane-changing models to develop a microsimulation model for truck platooning and evaluate the traffic impact of heavy trucks on the I-701 corridor with diverging sections (26). They incorporated field data for time gap settings and calibrated the model to realistically simulate truck CACC operations. The results for the two scenarios were the base case with a $0 \%$ penetration rate and a $100 \%$ penetration rate of CAV. The results showed that traffic operations were improved due to truck platooning. One limitation of this study was that the authors only compared $100 \% \mathrm{mpr}$ with the base case $(0 \% \mathrm{mpr})$ and did not consider fuel consumption

Kan et al. calibrated two driving behavior models, PATH and LMRS-IDM+ model, which were intended to reproduce the traffic dynamics of large scale and complex freeway corridors (27). A case study was conducted using real-world data at a complex 13-mile long section of the northbound SR99 freeway corridor. PATH and LMRS models were implemented on AIMSUN and MOTUS respectively. The driver behavior models accurately replicated the locations and throughput of the freeway bottlenecks. Both models outperformed the proprietary driver behavior model commercially available in AIMSUN.

### 3.6. Truck platooning: Impact on Operation

Jo et al. quantified the benefits of travel time savings with the application of truck platooning (28). The data used in the analysis include network and origin-destination data of 252 traffic in Korean freeway networks. The integration of micro and macro simulations was done to estimate the capacity and travel time to better capture the impact of truck platooning. Inputs of the simulations
were platoon size, inter platoon distance, intra-platoon distance, and penetration rate with 160 scenarios. The simulation result showed a positive impact of truck platooning in reducing transportation costs.

One limitation of this study is that the simulation model was not calibrated with field experiments. Simulation models should be calibrated and validated to enhance the reliability of the result. Only capacity and travel time savings were quantified, but fuel savings, environmental impact, and safety issues were not considered in the result. Also, while calculating the travel time saving, different costs for truck platooning like operating, automation, and infrastructure costs were not considered.

Yang et al. simulated the impact of large-scale truck platooning on traffic safety and efficiency especially within critical traffic locations like merging and diverging areas (29). Traffic data of A15 corridors with merging, diverging and weaving areas were used in the analysis. Microscopic simulation using Vissim was used then to simulate truck platooning in the A15 corridor. Wiedemann 99 model was developed for the car-following behavior of drivers. The result showed, with the increase of platoon intensity, traffic efficiency and safety decreased in both merging and diverging areas. The platoon would increase the traffic flow only in a minimum intensity scenario. One limitation of this study was that the traffic data used were not compatible to validate all the Vissim model parameters, as according to Fan et al. (2012), lane changing, minimal gap acceptance, safety distance, standstill distance (CC0), headway time (CC1) and the threshold for entering 'following' (CC3) parameters should be validated. Only two types of vehicles cars and trucks were used in the analysis, which may bias the result. Another limitation was that human factors were not considered in this study which may affect the results.

Calvert et al. examined the effect of truck platooning on traffic flow and traffic performance (30). LMRS-IDM + model was employed to incorporate truck platooning. Traffic simulation was conducted on a 56.6 km motorway corridor from Dutch motorway A67. They used four variables including traffic state, truck gap settings ( $0.3,0.5,0.7$ seconds), platoon size ( 2 and 3 ), and share of equipped trucks or penetration rate $(20 \%, 50 \%$, and $80 \%)$ to evaluate total traffic performance, the performance of traffic at ramps, and the ability of trucks to remain in a platoon. For this study area, truck platooning was found to have a small negative effect on traffic flow, which is negligible, but a higher negative effect for near congested or congested traffic flow. The authors suggested not allowing truck platooning in a saturated traffic state. It was suggested also that as no positive effect was found, the main concern for truck platooning should be emission and energy consumption. The main drawback of this study was that the result obtained was for the European context where vehicles were required to drive on the outside lane when not performing overtaking maneuverers, which is contrary to common practice in the US.

Wang et al. investigated the benefits and risks associated with truck platooning on freeway operations, especially in diverging sections (31). A truck acceleration model was proposed and integrated with the car-following model IDM + to simulate truck platooning. The enhanced microsimulation model LMRS-IDM+ was implemented in MOTUS. Input variables were platoon size, intra-platoon distance, traffic intensity, and penetration rate. The result showed that platooning caused significant merging problems in the diverging section (on-ramp). Proposed solutions for this problem were platoon yielding, platoon change lanes, and larger intra-platoon spacing for vehicle cut-ins. Truck platooning increased road capacity, but the increase was only
significant at high penetration rates. Vehicles unable to merge on time were deleted from the simulation, which limited the validity of the benefit of platooning.

### 3.7. Traffic Safety

Li et al. evaluated the impacts of the Cooperative Adaptive Cruise Control (CACC) system on reducing the rear-end collision risks on freeways (32). The car-following model was based on the Intelligent Driver Model, IDM. Two surrogated safety measures, Time Integrated TTC (TIT) and Time Expose TTC (TET), were used to quantify the collision risks. The safety effects of the CACC system were theoretically analyzed based on both TTC definition and linear stability. The simulation study was conducted to compare the result with the theoretical analysis. The theoretical and simulation results conformably indicated that the CACC system benefited dramatically from the reduction of rear-end collision risks. When the two key factors, the desired time headway, and the time delay were set properly (such as 0.6 s and 0.3 s , respectively), the TET and TIT could be reduced by more than $90 \%$. The sensitivity analysis of TTC and the length of the CACC platoon also indicated there were no significant differences between these two factors. Furthermore, the safety effects of the CACC system weakened, with the decrease of the penetration rates on the market and the increase of time delay between platoons. Results also showed that the additional proportions of traffic flow per hour increased when the desired time headway declined to some extent.

Zhao \& Lee analyzed rear-end collision risk of cars and heavy vehicles on freeways using a modified crash potential index (CPI) which incorporates driver reaction time and vehicles maximum deceleration capacity (33). They proposed a CPI equation with modified DRAC (the deceleration to avoid crashes) to incorporate the driver's reaction time, as the reaction time of cars and heavy vehicles vary. Vehicle trajectory data were collected from the US-101 freeway NGSIM project and loop detector data were collected from westbound Gardiner Expressway in Toronto, Canada. Vissim model was calibrated using observed vehicle trajectory data to replicate the traffic condition on the location of loop detector data. The model was validated by comparing traffic conditions with loop detector data with RMSPE and MPE criteria. The errors were very low. CPIs were calculated and compared for both observed and simulated data. Observed data showed mean CPI for the HV-Car pair was highest. Simulated data showed CPIs were consistently higher for the Car-Car pair and Car-HV pair than the HV-Car pair for the crash scenario. The non-crash scenario showed a similar result to the observed data. Finally, a binary logit model was developed to predict crashes based on the CPI value. The result showed that the CPI was statistically significant for all vehicle pairs and predicted the crash with higher accuracy (AUC $=1.0$ ). Observed trajectory data in the location of loop detector data could not be used. Also, the driver's visibility was not considered. Considering these may reduce the model's performance but will reflect real-world behavior. Also, the car and heavy vehicles driving behavior differences should be incorporated in the simulation study.

Tu et al. examined the longitudinal safety impacts of vehicle's degradation from Cooperative Adaptive Cruise Control (CACC) to Adaptive Cruise Control (ACC) mode in a CACC fleet (34). To capture the realistic microscopic dynamics of ACC and CACC vehicles, models proposed by the California Partners for Advanced Transit and Highways (PATH) were implemented for simulation. To evaluate the safety impacts of degradation, the Time Integrated Time-to-collision (TIT) indicator was used in for safety evaluation. Extensive simulations experiments were conducted in MATLAB R2018a. Four experiments such as different driving states (i.e., cruise,
deceleration, and acceleration), one vehicle degradation, two adjacent vehicle degradation, and two non-adjacent vehicle degradation at different positions of a 10 CACC vehicle fleet were simulated. For the driving state, the result showed the deceleration state brought collision risk to the degraded vehicle with a TIT value of 7.8. Impacts of one degraded vehicle at different positions showed the maximum value of TIT appears in the second position. The TIT value drops as the degradation position moved to the middle of the fleet and reaches a minimum value at the fifth position. Then the risk increased gradually to the end of the fleet. When two adjacent vehicles degraded at the same time, the result showed a similar trend to that in one vehicle degradation scenario. The middle positions were still safer. When two non-adjacent vehicles degraded at the same time, the safety (TIT value) further decreased than the adjacent scenario. The main focus was to evaluate safety in the longitudinal direction, and the lateral direction was not considered in the simulation.

### 3.8. Gaps in Previous Studies

Autonomous vehicles are defined as "those in which operation of the vehicle occurs without direct driver input to control the steering, acceleration, and braking and are designed so that the driver is not expected to constantly monitor the roadway while operating in self-driving mode." (NHTSA) Connected vehicles are equipped with communication technologies to exchange information with other users on the road. Connected and autonomous vehicles are a new and transformative technology, and they are rapidly evolving and developing capabilities (35). Cavs are still in the transition period. In this period, CAVs will have a negative effect on traffic flow. According to Calvert et al., a positive effect of CAVs on traffic flow and capacity was observed only when the penetration rate was above $70 \%$ (3). With the increase of the market penetration rate, the positive effect will increase. Rios-Torres \& Malikopoulos showed that with a full market penetration of CAVs, the traffic patterns would be more stable even with high traffic density (4).

One of the most important applications of CAVs is their platooning capabilities. Platooning means several vehicles following a leader vehicle with reduced headway to reduce fuel consumption and increase the traffic flow (12). Introducing platooning at the transition period of CAVs will negatively affect traffic flow, but with the increase of market penetration rate the positive effect will increase. Truck platooning will be the future of transport with certain benefits like reduction of fuel consumption, emission reduction, and increase the economy of scale (36). Browand et al. investigated the field test result of two tandem trucks, found fuel reduction capacity of platoons for both leads, and trail trucks (17). Lu \& Shladover investigated the result from field-testing of a vehicle platooning system and the test achieved a $14 \%$ fuel reduction for the third truck of the platoon (19).

The impact of truck platooning on traffic flow, capacity, and safety are still unclear. Wang et al. investigated the impact of truck platooning in a diverging section with a simulation experiment (31). The result showed that truck platooning increased road capacity, but the increase was only significant at high penetration rates. Jo et al. quantified the benefits of travel time savings with the application of truck platooning and found a positive impact on reducing transportation costs (28). In the study of Calvert et al., no positive effect was found of truck platooning on traffic flow (30). Authors suggested that the main concern for truck platooning should be emission and energy consumption. Yang et al. showed a simulation result of large-scale truck platooning within a critical traffic location (29). The simulation result showed a negative effect of increased platoon intensity on traffic safety and efficiency. Li et al. evaluated the impacts of a Cooperative Adaptive

Cruise Control (CACC) system with simulation experiments and found that it dramatically reduced the rear-end collision risks (32). Tu et al. simulated and examined the longitudinal safety impacts of vehicle's degradation from Cooperative Adaptive Cruise Control (CACC) to Adaptive Cruise Control (ACC) mode in a platoon (34). As the platoon vehicles will be CACC equipped, degradation from CACC to ACC for any following vehicle will reduce the safety substantially. Very few papers dealt with the safety aspect of truck platooning.

## 4. METHODOLOGY

### 4.1. Operational and Environmental Analysis at Corridor-Level

### 4.1.1. Stakeholder Engagement

We have done many communications with many stakeholders in the US to get the required data for this project.
1- We have contacted National Performance Management Research Data Set (NPMRDS) in Jan. 2020. In Feb. 2020, they have provided us with the link to download GIS shapefiles of TMC segments, which includes AADT data. We signed up for an account and got access to their massive data downloader tool. This tool includes daily data from 2011 to 2020 for 24 -hour period with $10,15-$ and 60 -minute interval. For every record, it includes speed, historical average speed, reference speed, travel time, and data density values. We can select the data using TMC code and download it in CSV format.
2- We have contacted Capital Region Planning Commission (CRPC) in Feb 2020. In late March 2020, they provided us with the data through email. The dataset includes GIS shapefiles of TMC segments for Baton Rouge with AADT data.

We contacted Texas Transportation Institute and Texas Department of Transportation to obtain Traffic data (volume, speed, classifications etc.) and Pavement structure data (layers thicknesses, materials properties, etc.) to assist with finite element simulations.

### 4.1.2. Data

The data set used in this task of the analysis was obtained from the National Performance Management Research Data Set (NPMRDS). This data set contains GIS shapefiles of TMC segments which includes AADT data.

This dataset provides massive daily data from 2011 to 2020 for a 24 -hour period with 10,15 , and 60 -minute intervals. For every record, it includes speed, historical average speed, reference speed, travel time, and data density values.

The output provides two files, one containing Speed and travel time data and the other containing TMC segment data.

- Speed/Travel time file includes Speed, Historical Average Speed, Reference Speed, Travel Time, and Data Density Values for every time period.
- TMC segment data includes useful information like the number of through lanes and AADT.

Traffic count data for 2017-2018 were collected for the Baton Rouge area from the Capital Region Planning Commission (CRPC). This count data was used to estimate the vehicle input of our model and validate the model.

### 4.1.3. Study area

We have selected a freeway segment with diverging and merging sections on I-10 road in Baton Rouge, Louisiana, which is a heavily utilized truck corridor situated in Region 6. We have
collected GIS shapefiles of the segment that was used to create the roadway network in Vissim. It is approx. 6.95 km ( 4.3 miles) corridor with 8 merging and 8 diverging sections (as shown in Figure 6).


Figure 6. Study area (a freeway section in I-10 road).

### 4.1.4. Scenario Design

The effects of truck platooning were investigated considering the following variables:

1. Platoon size ( $2,3,4$, and 5)
2. Inter-platoon distance ( 50 m , and 100 m )
3. Intra-platoon distance ( $0.3 \mathrm{~s}, 0.5 \mathrm{~s}$, and 0.7 s )
4. Market Penetration rate ( $25 \%, 50 \%$, and $100 \%$ )
5. Time period (Peak and Off-peak hour)

To model truck platooning in Vissim, we integrated python with Vissim to code the CAV platooning part.

Considering these factors, the results of the experimental design showed that a total of 144 scenarios need to be examined. However, using a fractional factorial design method, the number of simulation scenarios can be reduced to only 36 scenarios as shown in Table 1.

Table 1. Scenario design result.

| Time period | MPR | Platoon size | Inter platoon distance | Intra platoon distance |
| :---: | :---: | :---: | :---: | :---: |
| Reference peak hour | 0 | - | - | - |
| Peak hour | 25 | 2 | 100 | 0.3 |
| Peak hour | 25 | 3 | 100 | 0.3 |
| Peak hour | 25 | 2 | 100 | 0.5 |
| Peak hour | 25 | 3 | 100 | 0.5 |
| Peak hour | 25 | 2 | 100 | 0.7 |
| Peak hour | 25 | 3 | 100 | 0.7 |
| Peak hour | 50 | 2 | 100 | 0.3 |
| Peak hour | 50 | 3 | 100 | 0.3 |
| Peak hour | 50 | 2 | 100 | 0.5 |
| Peak hour | 50 | 3 | 100 | 0.5 |
| Peak hour | 50 | 2 | 100 | 0.7 |
| Peak hour | 50 | 3 | 100 | 0.7 |
| Peak hour* | 100 | 2 | 100 | 0.3 |
| Peak hour* | 100 | 3 | 100 | 0.3 |
| Off-peak hour | 25 | 3 | 50 | 0.3 |
| Off-peak hour | 25 | 4 | 50 | 0.3 |
| Off-peak hour | 25 | 5 | 50 | 0.3 |
| Off-peak hour | 25 | 3 | 50 | 0.5 |
| Off-peak hour | 25 | 4 | 50 | 0.5 |
| Off-peak hour | 25 | 5 | 50 | 0.5 |
| Off-peak hour | 25 | 3 | 50 | 0.7 |
| Off-peak hour | 25 | 4 | 50 | 0.7 |
| Off-peak hour | 25 | 5 | 50 | 0.7 |
| Off-peak hour | 50 | 3 | 50 | 0.3 |
| Off-peak hour | 50 | 4 | 50 | 0.3 |
| Off-peak hour | 50 | 5 | 50 | 0.3 |
| Off-peak hour | 50 | 3 | 50 | 0.5 |
| Off-peak hour | 50 | 4 | 50 | 0.5 |
| Off-peak hour | 50 | 5 | 50 | 0.5 |
| Off-peak hour | 50 | 3 | 50 | 0.7 |
| Off-peak hour | 50 | 4 | 50 | 0.7 |
| Off-peak hour | 50 | 5 | 50 | 0.7 |
| Off-peak hour* | 100 | 4 | 50 | 0.3 |
| Off-peak hour* | 100 | 5 | 50 | 0.3 |

### 4.1.5. Performance Indicators

To align the microscopic analysis with the project objective, the following surrogate measures were considered:

- Surrogate measures for operational impacts: Total Network delay (TND), Time to merge (TTM) and diverge (TTD)
- Surrogate measures for environmental impacts: Fuel consumption, Total emission of $\mathrm{CO}_{2}$, $\mathrm{NO}_{\mathrm{x}}$, and $\mathrm{PM}_{10}$
- Surrogate measures for safety impacts: Time Integrated Time to Collision (TIT)


## Total network delay (TND):

Total network delay is defined as the total time delay of vehicles compared to free-flow conditions. It indicates the severity of congestion.

$$
\begin{equation*}
\mathrm{Tnd}=\sum_{k=0}^{n}\left(t t_{k}-t t_{f f}\right) \tag{1}
\end{equation*}
$$

where:
$t t_{k}=$ total travel time of vehicle k ; and
$t t_{f f}=$ free-flow travel time.

## Time to merge/diverge (TTM, TTD):

Time to merge/diverge indicates the average time that all merging/ diverging vehicles require to merge/diverge from the point they enter the merging/diverging section to the point they leave the merging/diverging section.

$$
\begin{equation*}
\operatorname{Ttmd}=\sum_{k=0}^{n}\left(t_{B}^{k}-t_{A}^{k}\right) \tag{2}
\end{equation*}
$$

where:
$t_{B}^{k}=$ time vehicle k passes location B ; and
$t_{A}^{k}=$ time vehicle k passes location A .
For the merging section, location A will be on the merging section, and location B will be on the main freeway.

## Fuel Consumption and Emission:

The Vissim software provides an emission model to calculate the emissions of some or all vehicles in a simulation run. The Emission Model DLL Interface of Vissim is written in C/C++ which contains specific functions. During a simulation run, Vissim calls the DLL code for each affected vehicle in each simulation time step to calculate the emissions of the vehicle. Vissim passes the current state of the vehicle (speed, acceleration, gradient) to the DLL and the DLL computes the emissions in the current time step and passes these values back to Vissim to be shown in evaluations.

In this study, the fuel consumption of all vehicles was aggregated over the whole simulation run. Emission value was also aggregated for $\mathrm{CO}_{2}, \mathrm{NO}_{\mathrm{x}}$, and $\mathrm{PM}_{10}$ pollutants only. The unit for fuel consumption is gallons and for emission is grams.

## Time Integrated Time to Collision (TIT):

According to Hyden, Time to Collision (TTC) value is the time that remains until a collision between two vehicles would have occurred if the collision course and speed difference remain unchanged (37).

The TIT expresses the entity of the TTC lower than a safety threshold.

$$
\begin{align*}
& \operatorname{TTCi}(k)=\left\{\begin{array}{r}
\frac{x_{i-1}(k)-x_{i}(k)-L_{i-1}}{v_{i}(k)-v_{i-1}(k)}, \text { if } v_{i}(k)>v_{i-1}(k) \\
\infty, \text { if } v_{i}(k) \leq v_{i-1}(k)
\end{array}\right. \\
& \operatorname{TIT}(k)=\sum_{i=1}^{N}\left[\frac{1}{T T C i(k)}-\frac{1}{T T C *}\right] \cdot \Delta k, \forall 0<\operatorname{TTCi}(k) \leq T T C * \\
& \operatorname{TIT}=\sum_{k=1}^{T} \operatorname{TIT}(k) \tag{3}
\end{align*}
$$

where:
$\operatorname{TTCi}(\mathrm{k})=$ time to collision for the i vehicle at k time step
$x i(k)=$ Location of the $i$ vehicle at $k$ time step
$\operatorname{vi}(\mathrm{k})=$ Velocity of the i vehicle at k time step
TTC* $=$ threshold Time to collision
TIT $(\mathrm{k})=$ Tiem integrated time to collision at k time step

### 4.2. Operational and Environmental Analysis at Large-Scale

This section discusses the development of the requirements for the large-scale analysis of the impacts of truck platooning on congestion, emissions, and traffic flow dynamics.

### 4.2.1. Data

Vehicle trajectory data have been a great source to understand driver behavior. Unfortunately, due to the difficulties associated with collecting vehicle trajectory data at a corridor level, there exist very limited vehicle trajectory datasets (e.g., NGSIM I-80, NGSIM I-290, and HighD). None of these datasets, however, contains any automated vehicles (at any levels of automation, from Level 1 to Level 5). According, in order to better capture the impacts of platooning on congestion, emissions, and traffic flow dynamics, the proposal team collected their own corridor-level vehicle trajectory data. The dataset also contains some instances of three Level 1 automated vehicles (operating under full-range Adaptive Cruise Control) in a platoon formation. The details of the data collection approach and the extracted data is provided below.

Data Collection Methodology: Vehicles' trajectory can be extracted from the video frames recorded in the bird's-eye view from a segment of the roadway (figure 7.a). In every video frame, the vehicles' location can be estimated for a fixed coordinate system and reference point on the ground. Every video recording is converted to a sequence of images (i.e., frames) separated at a constant rate over time (e.g., 25 frames per second). Tracking the vehicle's location over the sequence of images enables extracting the vehicle's trajectory over time. The vehicle trajectory extraction can be performed in four steps: image stabilization, vehicle detection, vehicle tracking, and trajectory construction. In the image stabilization step, all the images are transformed to match a reference field of view. Then the vehicles are detected in every image and tracked over the sequence of images. Finally, the vehicle's location and trajectories are constructed by converting the image coordinates to the adopted reference coordinates on the ground. These steps are further elaborated in the following sections.

Image Stabilization: The location of every vehicle in an image is estimated by converting its position on the image map to the fixed coordinate system picked on the ground. Consequently, it is essential to find the mapping function between the image coordinate to the adopted ground coordinate. Image stabilization is the process of converting the field of view of all the images (i.e., frames) to a reference image for which the mapping function to the ground coordinate is known. The image stabilization is performed in three steps; first detecting the key features in both reference and input images, second, finding the matching features between the two images, and third, estimating transformation between them. There exist different algorithms for good key features detection in images such as Harris corner detector (38), Scale-Invariant Feature Transform (SIFT) (39), Speeded up Robust Feature (SURF) (40), and Oriented FAST and Rotated BRIEF (ORB) (41). Among these algorithms, SIFT and SURF are the top-performing feature detectors in terms of scale and transformation. The second step is matching the features between the reference image and input image. One naive approach is to compare every feature in the reference image with every feature in the input image to find the best matching pairs; however, this would be very timeconsuming and impractical for a video data collection at a high frame rate. Instead, to improve the computation speed, the Fast Library for Approximate Nearest Neighbors (FLANN) matcher can be utilized to match features between the images (42). The final step is finding the perspective transformation, specifically the homography, between the reference and input images considering the best matching features. There are chances that some of the feature matches are incorrect. The Random Sample Consensus (RANSAC) algorithm is a technique to find the model parameters from a dataset with many outliers through an iterative process (43). RANSAC can be used to estimate the homography transformation between two images considering the matched key features.

Object Detection: Object detection techniques in computer vision can be used to identify and locate the vehicles in the aerial images. The classical approach for object detection is to identify the informative regions in the image that contain objects of interest, then extract semantic and representative features from them, and classify the objects in those regions. Deep neural networks (DNN) made a significant performance breakthrough in object detection due to the capacity of the convolutional neural networks (CNN) to learn more complex features than shallower models. There is multiple popular CNN based object detectors such as R-CNN (44) and RetinaNet (45). The weights and parameters of a pre-trained CNN based object detector can be fine-tuned by training on a dataset of aerial images with known vehicle annotations. The trained model can be used in the vehicle detection process to identify and locate vehicles on the aerial images. The input


Figure 7. Vehicle detection and tracking in aerial images.
to the vehicle detection is a stabilized image, and the output is the coordinates of the bounding boxes enclosing the vehicles in the image. Figure 7.b presents the detected vehicles and their visualized bounding boxes.

Object Tracking: Tracking is the process of linking the new detections to previous observations. The tracking methodology proposed for this study includes data association and track maintenance. Data association is associating the detected vehicles in the current image frame to the vehicles identified in the previous ones (Figure 7.c). The track maintenance is in charge of initiating new tracks, maintaining the tracks, and deleting them. The track maintenance initiates tracks with unique ids to all the vehicles detected in the first image frame. After that, for every image, all the newly detected vehicles are compared with the existing tracks using the data association. The tracks are updated as new detection is associated with them. A new track is constructed for any new observation that is not associated with the current tracks. Moreover, if a track is not updated in the previous n frames, the track maintenance deletes that track. A track object maintained by the track maintenance contains both the unique id of the track and the coordinates of the bounding box of its last observation.

Trajectory Construction: All the aerial images are transformed and stabilized, considering a reference image before extracting the vehicle trajectories. The trained vehicle detector model locates the vehicles on the stabilized images, and the resulting bounding boxes are used in the tracking of the vehicles from one frame to another. The bounding boxes represent the vehicle's location in image coordinates (i.e., row and column of pixels). These coordinates need to be converted to a fixed ground coordinate system (e.g., meters or feet) for trajectory extraction. Every pixel is located by its row and column number in the image map. The pixel coordinate can be transformed into a cartesian coordinate system by taking axes parallel to the columns and rows of the image map and knowing the pixel size on the ground. The pixel size on the ground depends on the flight elevation and is the key to the mapping function between the two coordinate systems. The front bumper can be taken as the vehicle's location on the roadway, and the trajectory of the vehicle is the list of its location over space and time. Besides, a Kalman filter can be applied to reduce the noise in the vehicle's location estimates due to noisy bounding boxes from image stabilization and vehicle detection processes.

### 4.2.2. Data Collection in Austin, TX:

One of the primary motivations of this data collection was to observe how recent advancements in vehicle technology and ADAS impacts traffic flow dynamics and to utilize that knowledge in the simulation. However, it is not straightforward to distinguish vehicles using ADAS from the birdseye view. The accessible solution to the problem was to use probe vehicles during the data collection. Accordingly, this study evaluated the impacts of the most common yet demonstrating ADAS feature, Adaptive Cruise Control (ACC). ACC is also a core feature amongst all platooning applications. The trajectory data was collected in Austin, TX (see Figure 8).

For the data collection in Austin, a platoon of three probe vehicles, including two Toyota Prius and one Toyota Avalon, were used under ACC for data collection. The platoon leader was following an arbitrary vehicle on the roadway in front of it using ACC. The other two vehicles were also following their leaders with ACC. A total of five runs were performed along the study roadway segment. Note that all vehicles had full-range ACC. The data was collected on the southbound of Interstate Highway 35 between Exit 237B and Exit 238A in Austin, TX. A single


Figure 8. Time-space diagram of the vehicles for ten minutes on the Southbound of I-35 between Exit 237B and 238A, Austin, TX, during the morning peak time.
stretch of 500 feet roadway was recorded for 2 hours between 07:30 AM and 09:30 AM on a Friday.

The key contribution of this data collection was to introduce the first collected comprehensive trajectory dataset from both ACC and human-driven vehicles. One of the main features of this dataset is the continuity in data recording for over two hours during the morning traffic peak hours. The continuity in data collection ensures that no information or interaction between the vehicles is lost. Figure 9 presents the examples of the time-space diagram of the trajectories extracted using aerial videography on a roadway segment of approximately 500 feet over 10 minutes. According to this figure, some of the trajectories are not continuous, which is due to the lane-changing in most cases. However, there are few cases that the vehicle detector has failed to detect a vehicle in the image, causing a discontinuity in its trajectory. In this study, two actions were applied to address these types of errors. First, the false-negative error in detection was mitigated in the tracking process by the combination high frame rate (i.e., 25 fps and 30 fps ) and maintaining track of the vehicle for five consecutive frames after the last time it was seen. Second, the false-positive error was reduced by eliminating the trajectories with less than three data points. The quality of data is then further improved by manually addressing the false-negative and false-positive detections. More details on this data collection effort and findings of the study can be found in (46).

### 4.3. Pavement Structural Analysis

### 4.3.1. Introduction

Over the past few years, autonomous truck platooning technology (a set of connected heavy-duty trucks travelling closely at specific headway intervals) has been tested and would be utilized in near future. Connected autonomous vehicles (CAV) can be implemented as a contribution to sustainable freight and potentially offers road safety improvements, elevated economic aspects, and environmental preservation.

However, autonomous trucks platooning can negatively affect AC pavements service life due to limited tire wandering as the trailing trucks closely follow the leading truck's pathway. Failure to take account of wandering can impose significant maintenance costs on national and state road network. This study aims at evaluating the detrimental effects of wandering. From a structural point of view, a limited wandering pattern leads to concentration of induced maximum stress (or strain) over a narrower area beneath the tires. Hence, these areas become highly prone to fatigue and permanent deformation.

Although numerous studies have been conducted to evaluated various aspects of AVs, the effects of platooning on pavement condition have not been studied thoroughly. In this task the following tasks are conducted to take into account the effective features of AV platoons:

- Traffic data will be analyzed to provide an estimation for the truck share, axle load magnitude, and spectrum.
- Material characterization will be used to develop the master curves and Prony series parameters to account for viscoelastic behavior of asphalt materials.
- Finite Element Method (FEM) Models will be implemented to assess the effect of truck platooning effect on pavement.
- MEPDG method is implemented to evaluate the impact of fixed-path wandering on fatigue and permanent deformation damage.


### 4.3.2. Background on Traffic Loading Impact on Pavement

Generally, numerous factors can affect the mechanical response of asphalt concrete (AC) pavements. In this section, the most significant factors are reviewed based on previous research.

Vehicle Speed and Dynamic Loading Impacts: A NCHRP report developed by Gillespie et al. (1993) demonstrated that structural response of AC pavements is influenced by the speed of the passing load (i.e., loading time) and also the consequential dynamic loading impacts. The authors believed that for higher speeds, the effects of reduced loading time can be counterbalanced by the increased dynamic load impact (47).

In a study conducted by Sarkar, a FEM model was used to numerically assess the impact of speed and AC characteristics on critical pavement responses (48). The assessment was based on different loading axles types including single, tandem and tridem axles. It was concluded that decrease in moving speed does not necessarily increase the measured critical strains and in some cases, higher speed induces higher structural responses.


Figure 9. Maximum longitudinal and transverse strain for different vehicle speeds.
Overloading: Overloading is one of the crucial topics considered in pavement design and many researchers have aimed at evaluating the overloading damage on transportation infrastructures. In a study by Pais et al., the effect of overloaded was determined using the truck factors for both different pavement layer thicknesses and subgrade stiffness moduli (49). Based on the results, the overloading can double the pavement cost. Another research demonstrated that the increased number of overloaded trucks up to $20 \%$ can lead to a $50 \%$ reduction in the fatigue life of asphalt pavement (50). Gungor et al. measured the impacts of overweight (OW) trucks on road infrastructures (including pavement) to determine realistic permit fee system based on prediction algorithms (51).

Zaghloul et al. implemented a three-dimensional, dynamic finite element program (3D-DFEM) to analyze pavement through "static, linear elastic analysis and dynamic, nonlinear analysis". 3DDFEM predictions were compared with the results of a multi-layer analysis. Also, the measured pavement deflections were compared with the 3D-DFEM predictions for dynamic verifications (52).

Wheel-path Wandering: Load-induced distresses such as fatigue and rutting have significant effect on pavement condition and are considered as fundamental parameters in pavement thickness design. Since the stress and strain magnitudes in a certain depth are inversely related to horizontal distance to loading point, wheel path wandering affects the induced damage. Many studies have been conducted to assess the effects of tire wandering. Noorvand et al. implemented "AASHTOWare Pavement ME Design" to assess the effect of truck wheel-path on the long-term performance of pavement structures for exclusive and partial use by autonomous trucks. The evaluations were conducted based on rutting and fatigue cracking MEPDG analysis (53).

In a work done by Tamura et al., a path tracking controller for a semitrailer-like vehicle was implemented by the means of time scale transformation and linearization. The vehicle's controller was designed to follow arbitrary paths consisting of arcs and lines (54).

Erlingsson et al. evaluated the impact of lateral wandering standard deviation characteristics on rut development. The rutting calculations were based on two methods including mechanistic empirical (M-E) evaluation of the permanent strain in all layer of the structure and time hardening approach in bonded layers (55).

FEM Analysis of AC Pavements: Researchers have conducted many studies to develop micromechanical FEM models. Al-Rub et al. implemented FE material constitutive behaviors including viscoelastic-viscoplastic, elasto-viscoplastic, coupled viscoelastic, viscoplastic, viscodamage. The results demonstrated that 2D plain strain models provide an overestimated rut values compared to 3D models (Mshali \& Steyn, 2020). Darabi et al. implemented numerical algorithms to simulate the fatigue and healing behavior of asphalt concrete by the means of "Abaqus via the user material subroutine UMAT" (56). In a research conducted by Ban et al, a 3D FEM simulation was developed based on Schapery's nonlinear viscoelastic constitutive model. A UMAT FE software was implemented to evaluate the asphalt pavement undergoing heavy truck loads $(57,58)$.

Ambassa et al. developed a viscoelastic finite element method (FEM) model to determine asphalt concrete pavement damage due to multiple-axle moving traffic loads. The model was evaluated for different scenarios including different pavement structures, passing load velocities, load configurations, and AC layer temperature (59).

Al-Qadi et al. conducted a comparison between the measured responses and the finite element using linear elastic theory. Results demonstrated that "the elastic theory overestimates pavement responses at low temperatures but significantly underestimates these responses at high temperatures". An improved prediction was also suggested by the means of the bonding conditions at the interfaces and also implementing a viscoelastic method (60).

Assogba et al. evaluated the effects of vehicular speed and overloading on the induced dynamic loads and developed a three-dimensional finite element model to calculate the stress and strain below the asphalt concrete (61). The model was further validated by comparing these strain and stress values with field measured dynamic strains (using the embedded Fiber Bragg Grating sensors). As shown in Figure 10, the FEM analysis was conducted using implicit method and DLOAD user subroutine; this model was analyzed using eight-node brick element with reduced integration and the pavement dimensions were considered to be $80 \times 20 \times 20$ meters (length $\times$ width $\times$ depth). Due to difficulty of simulating the real stress contact in study's programming subroutine, only the vertical tire-pavement contact stress was considered in the analysis.

Based on the results, the authors concluded that lower vehicular speed induces great increase in the loading duration and also increases the shock effect of tire load. The overloading effect highly depends on the overload extent. Moreover, their fatigue analysis demonstrated that the concurrent effects of overloading and decreased vehicular speed can significantly reduce the service life of the pavement.


Figure 10. The simulated FEM model and meshing (61).
Mshali and Steyn measured the field elastic deflection using a multi-depth deflectometer (MDD) at different speeds and on different pavement sections while considering preset wandering scenarios (56). A traffic speed deflectometer (TSD) with tire inflation pressure of 700 kPa was implemented to apply the dynamic loading simulation. Testing temperature and ambient condition were not controlled during their field testing; however, tests were performed at a fixed day-time and for the same duration. The TSD speed were considered to be $2,5,10,20,30,40,60,80$ and100 $\mathrm{km} / \mathrm{h}$ and the wandering offsets were considered 192.5 and 500 mm from the center of the dual tire axle (Figure 11).

As shown in Figure 12, the results demonstrated that the type and structure of pavement had significant effect on the proposed calculated speed adjustment factors (SAF) and these factors cannot be generalized merely as a function of speed and load amplitude.


Figure 11. Deflection measurements by the MDD embedded in various depth of the pavement (56).


Figure 12. Effect of speed on deflection of different pavement (56).
The difference between the static and dynamic loading can be characterized by the oscillating motion of the combined suspension-tire system, which makes the dynamic load to deviate from the mean amplitude $(57,58)$.

Time-temperature Superposition (TTSP): Based on time-temperature superposition (TTSP) phenomena for a viscoelastic material, a viscoelastic parameter at two different temperatures can be equal if the test frequency is altered using TTSP equations.

The shift factor $a_{T}$ is used to determine frequency or loading time shift and it is defined as:

$$
\begin{equation*}
a_{T}=\frac{0}{T} \tag{4}
\end{equation*}
$$

where:
$\omega_{0}=$ frequency on master curve for a defined reference temperature; and
$\omega_{T}=$ frequency with equal property value at temperature T
Sigmoidal function is one of most conventional techniques that can be used to develop master curves. This function is also used by the MEPDG pavement design (Figure 13) $(59,62)$. The sigmoidal function can be defined as following (63):

$$
\begin{equation*}
E(t)=\delta+\frac{\alpha}{1+e^{\beta+\gamma\left(\log T_{R}\right)}} \tag{5}
\end{equation*}
$$

where:
$E(t)=$ Cyclic modulus;
$\delta=$ Minimum modulus value;
$\alpha=$ Span of modulus value;
$\beta, \gamma=$ Shape parameters; and
$T_{R}=$ Referenced temperature.


Figure 13. Sigmoidal function parameters effect (63).
Williams et al. discussed the dependence of the shift factor temperature $T$ relative to the reference temperature as $a(T)_{R}$ and defined it as Williams-Landel-Ferry (WLF) shift factor (64):

$$
\begin{equation*}
\log a(T)_{R}=-\frac{C_{1}\left(T-T_{R}\right)}{\left(C_{2}+T-T_{R}\right)} \tag{6}
\end{equation*}
$$

where:
$T=$ Shifted temperature (or the selected temperature); and
$C_{1}$ and $C_{2}=$ Empirical coefficient.
Log-Linear Shift factor is one of the simplest functions to determine the asphalt concrete shift factors (not generally for the binders) (65), which has only one degree of freedom. Log linear function can be defined as:

$$
\begin{equation*}
\log a_{T}=C_{1}\left(T-T_{R}\right) \tag{7}
\end{equation*}
$$

Modified Kaelble method is somehow similar to WLF shift factor and it is mostly used for asphalt concrete mixtures. Modified Kaelble shift factor is defined as:

$$
\begin{equation*}
\log a(T)_{R}=-\frac{C_{1}\left(T-T_{R}\right)}{C_{2}+\left|T-T_{R}\right|} \tag{8}
\end{equation*}
$$

Arrhenius method is a one-coefficient shift factor, which is mostly used for asphalt concrete mixtures and can be determined as follows:

$$
\begin{equation*}
\log a_{T}=A\left(\frac{1}{T}-\frac{1}{T_{R}}\right) \tag{9}
\end{equation*}
$$

TTSP Validation: In a study performed by Texas DOT, Glover et al. aimed to develop an improved technique for asphalt long-term pavement performance using dynamic shear rheometer (DSR) test. DSR is considered to perform the measurements with angular velocity of $0.005 \mathrm{rad} / \mathrm{s}$ at $15{ }^{\circ} \mathrm{C}$. Authors proposed a time-temperature superposition shift to $44.7^{\circ} \mathrm{C}$ and $10 \mathrm{rad} / \mathrm{s}$ to introduce a method that is easily accessible to standard laboratory equipment. Also, the effect of polymer modification and aging on shift factor of some asphalt concrete specimens (Glover et al., 2001). Based on the results provided in this study, the comparison of DSR function $\left(\frac{G^{\prime}}{\left(\eta / / G^{\prime}\right)}\right)$ at 15 ${ }^{\circ} \mathrm{C}$ and $0.005 \mathrm{rad} / \mathrm{s}$ show satisfactory agreement with the DSR function values at $10 \mathrm{rad} / \mathrm{s}$ and 44.7 ${ }^{\circ} \mathrm{C}$ for several asphalts (Figure 14). This fact implies the reasonable capability of TTSP to simulate AC behavior at a specific temperature and frequency using results obtained at a more preferable temperature and respective frequency (which is determined using shift factor functions).


Figure 14. Increase of DSR function at a temperature and shifted temperature (66).
Walubita et al. conducted uniaxial static-direct loading test with a strain-controlled condition to evaluate master curve development techniques including Arrhenius, Williams-Landel-Ferry (WLF), and a proposed sum of square error (SSE) method (67). In this research, the controlled strain was limited to 200 micro strain and relaxation tests were conducted at 3 different temperatures $\left(10,20\right.$, and $\left.30^{\circ} \mathrm{C}\right)$. The authors compared the experimental data to the implemented techniques and concluded that Arrhenius and the Williams-Landel-Ferry (WLF) satisfactory results (r-squared=0.997) if appropriate constants were used (Figure 15).


Figure 15. The developed master curve using WLF (67).
Forough et al. evaluated 5 master curve development shifting techniques, including Numerical, Log-Linear, WLF, Modified Kaelble, and Arrhenius. They tested 72 AC specimens with different aggregate gradation, asphalt content, aging conditions at four different temperatures. Experimental data was obtained using direct tension relaxation test at a controlled strain level of 200 microstrain (62). According to the calculated relaxation modulus in different conditions and using mean normalized error (MNE), Arrhenius shift function had the least MNE value in all temperatures, binder percent, and etc. moreover, the high r-square value (approximately $98 \%$ ) demonstrates the capability of TTSP technique (Table 2).

Table 2. Statistical comparison among shifting methods conducted by Forough et al. (62).

| Testing condition | Technique | No. of samples | $R^{2}$ | SER | $r_{i}(\boldsymbol{*})$ | MNE | ADG |
| :--- | :--- | :---: | :---: | :---: | :---: | :---: | :---: |
| All the experimental combinations | Log-Linear | 288 | 0.9815 | 0.1375 | 54.17 | 13.9595 | 1.1341 |
|  | WLF | 288 | 0.9827 | 0.1365 | 56.25 | 13.8237 | 1.1293 |
|  | Modified Kaelble | 288 | 0.9823 | 0.1370 | 54.86 | 13.9412 | 1.1329 |
|  | Arrhenius | 288 | 0.9848 | 0.1238 | 67.36 | 12.8633 | 1.1178 |

### 4.3.3. Methodology

The effect of truck platooning was evaluated using the mechanical strain outputs obtained from dynamic FEM simulation. In order to develop the FEM model, axle weight, material characteristics, and temperature data are implemented. Further details are provided in model development section. Figure 16 illustrates a schematic layout of this study. Pavement characteristics including layers thickness, modulus of elasticity, Poisson ratio, and dynamic modulus; traffic analysis including axle loads spectrum-distribution; platooning specification including wheel wandering, axle configuration, and truck speed as well as the seasonal temperature were attained.


Figure 16. Schematic layout of the study.
In order to simulate the viscoelastic behavior of the AC layers, the truck speed, pavement temperature, and dynamic modulus data are analyzed. The viscoelastic behavior is defined in terms of time domain Prony series, which is discussed in the pavement characteristics section. Finally, the MEPDG method is implemented to quantify the fatigue and permanent deformation damage based on the ABAQUS outputs (resulting mechanical strains).

Traffic Analysis: In this analysis, the traffic data including traffic distribution and truck axle loading spectrum for IH-35 was gathered from TxDOT database prepared by Texas Transportation Institute. Table 3 shows the analyzed traffic data in terms of axle type (i.e., steering, non-steering single axle, tandem, and quad) and axle distribution (the share of each axle load magnitude compared to total traffic volume). For each axle type the red and green spectrum illustrated higher and lower percentage share, respectively.

Table 3. The analyzed traffic data for different axle types.

| Steering <br> Axle <br> Load <br> (kips) | Percent | Non- <br> Steering <br> Single <br> Axle <br> Load | Percent | Tandem <br> Axle <br> Load <br> (kips) | Percent | Tridem <br> Axle <br> Load <br> (kips) | Percent | Quad <br> Axle <br> Load <br> (kips) |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| 3 | 0.7 | 3 | 6.4 | 6 | 0.0 | 12 | 15.4 | 12 | 0.0 |
| 4 | 0.4 | 4 | 5.6 | 8 | 0.0 | 15 | 8.4 | 15 | 0.0 |
| 5 | 0.6 | 5 | 6.7 | 10 | 4.9 | 18 | 10.0 | 18 | 0.0 |
| 6 | 0.9 | 6 | 5.2 | 12 | 6.7 | 21 | 7.1 | 21 | 0.0 |
| 7 | 2.4 | 7 | 6.0 | 14 | 7.7 | 24 | 5.2 | 24 | 0.0 |
| 8 | 3.5 | 8 | 6.5 | 16 | 8.6 | 27 | 3.4 | 27 | 0.0 |
| 9 | 7.9 | 9 | 6.5 | 18 | 9.1 | 30 | 6.4 | 30 | 0.0 |
| 10 | 12.5 | 10 | 5.5 | 20 | 8.8 | 33 | 6.5 | 33 | 0.0 |
| 11 | 22.3 | 11 | 6.3 | 22 | 8.2 | 36 | 5.0 | 36 | 0.0 |
| 12 | 19.6 | 12 | 4.9 | 24 | 7.6 | 39 | 4.7 | 39 | 0.0 |
| 13 | 21.6 | 13 | 6.8 | 26 | 8.4 | 42 | 7.8 | 42 | 0.0 |
| 14 | 6.2 | 14 | 6.6 | 28 | 7.7 | 45 | 8.5 | 45 | 0.0 |
| 15 | 0.7 | 15 | 5.9 | 30 | 7.6 | 48 | 3.1 | 48 | 0.0 |
| 16 | 0.4 | 16 | 4.8 | 32 | 7.1 | 51 | 1.8 | 51 | 0.0 |
| 17 | 0.2 | 17 | 5.4 | 34 | 4.7 | 54 | 2.8 | 54 | 0.0 |
| 18 | 0.1 | 18 | 5.4 | 36 | 1.9 | 57 | 1.8 | 57 | 31.8 |
| 19 | 0.0 | 19 | 2.6 | 38 | 0.7 | 60 | 0.6 | 60 | 31.8 |
| 20 | 0.0 | 20 | 1.7 | 40 | 0.2 | 63 | 0.6 | 63 | 0.0 |
| 21 | 0.0 | 21 | 0.6 | 42 | 0.1 | 66 | 0.0 | 66 | 0.0 |
| 22 | 0.0 | 22 | 0.3 | 44 | 0.0 | 69 | 0.3 | 69 | 0.0 |
| 23 | 0.0 | 23 | 0.1 | 46 | 0.0 | 72 | 0.3 | 72 | 0.0 |
| 24 | 0.0 | 24 | 0.1 | 48 | 0.0 | 75 | 0.0 | 75 | 0.0 |
| 25 | 0.0 | 25 | 0.0 | 50 | 0.0 | 78 | 0.0 | 78 | 0.0 |
| 26 | 0.0 | 26 | 0.0 | 52 | 0.0 | 81 | 0.0 | 81 | 0.0 |
| 27 | 0.0 | 27 | 0.0 | 54 | 0.0 | 84 | 0.0 | 84 | 0.0 |
| 28 | 0.0 | 28 | 0.0 | 56 | 0.0 | 87 | 0.0 | 87 | 31.8 |
| 29 | 0.0 | 29 | 0.0 | 58 | 0.0 | 90 | 0.0 | 90 | 0.0 |
| 30 | 0.0 | 30 | 0.0 | 60 | 0.0 | 93 | 0.0 | 93 | 0.0 |
| 31 | 0.0 | 31 | 0.0 | 62 | 0.0 | 96 | 0.0 | 96 | 1.6 |
| 32 | 0.0 | 32 | 0.0 | 64 | 0.0 | 99 | 0.0 | 99 | 0.0 |
| 0.0 | 33 | 0.0 | 66 | 0.0 | 102 | 0.0 | 102 | 3.1 |  |

Temperature: The temperature data for the $\mathrm{IH}-35$ is measured at $0,3.5,11$, and 22 inches deep using multi depth temperature probes. Hence, temperature at any depth can be extrapolated using the collected data. The average seasonal temperatures are illustrated in Figure 17. The pavement temperature further was used to develop the TTSP relationship.


Figure 17. Pavement seasonal temperature data for IH-35.
Pavement Characteristics and Road Section Description: In this research, the studied pavement section is located in interstate highway 35 (IH-35) in San Antonio. The selected section in one of the typical sections designed to evaluate the Texas perpetual pavements (PP) $(68,69)$. The details of typical and the studied pavement section are provided in Tables 4 and 5, respectively. The section is comprised of totally 21.5 -inch thick asphalt mixture layers and a 6 -inch $3 \%$ lime-treated subbase.

Table 4. Typical section of PP constructed in Texas (68).
Layer Designation, Materials, and Functions Thickness


Table 5. The selected section for the study (The Texas PP Database, Texas Transportation Institute) (68).

| Layer\# | Thickness (Inches) | Layer Material |
| :---: | :---: | :--- |
| 6 | 1.5 | PFC |
| 5 | 2.0 | SMA |
| 4 | 2.0 | $1_{2}^{\prime \prime}$ SFHMA |
| 3 | 12.0 | $1^{\prime \prime}$ SFHMA (RRL) |
| 2 | 4.0 | RBL ( $1 / 2^{\prime \prime}$ Superpave) |
| 1 | 6.0 | $3 \%$ Lime Treated Subgrade Material |
| 0 | $\infty$ | In-situ Subgrade Soil |

As demonstrated in Table 5, the pavement structure of the selected section is a combination of porous friction course (PFC), stone matrix asphalt (SMA), stone-filled hot mix asphalt (SFHMA), SFHMA rut-resistance layer (RRL), and rich-bottom layer (RBL). Mix design specifications of each layer is provided in Table 6.

Table 6. Mix design specifications of the selected section (68).

| Layer\# | Mix/Material | Average In-Service Typical Material Characteristics |
| :---: | :---: | :---: |
| Layer 1 <br> (Optional) | PFC | 6.0-6.1 $\%$ PG 76-22S $+0.0-1.0 \%$ lime $+0.3-0.4 \%$ cellulose fibers + igneous/limestone aggregates ( 19 mm NMAS open-graded) (Avg OAC $=6.0 \%$ ) |
| Layer 2 | SMA | $5.9-6.8 \%$ PG 76-22S $+5.0-11.0 \%$ mineral filler $+0.0-1.5 \%$ lime $+0.0-0.4 \%$ cellulose fibers $+0.0-4.5 \%$ fly ash + igneous $/$ limestone aggregates $(12.5 \mathrm{~mm}$ NMAS gap-grade $)($ Avg $\mathrm{OAC}=6.0 \%)$ |
| Layer 3 | $\begin{aligned} & \text { 3/4-inch SF } \\ & \text { (HMAC) } \end{aligned}$ | $4.2-5.2 \%$ PG 76-22 $+0.0-1.5 \%$ lime $+0.0-1.0 \%$ anti-strip + limestone aggregates ( 19 mm NMAS dense to coarse graded) (Avg OAC $=4.4 \%$ ) |
| Layer 4 | 1-inch SF <br> (HMAC) <br> (RRL) | $4.0-4.5 \%$ PG 70-22 + 0.0-1.5 \% lime $+0.0-0.5 \%$ anti-strip + limestone ( 25 mm NMAS coarse-graded with low fines) (Avg OAC $=4.2 \%$ ) |
| Layer 5 | RBL | 4.2-6.1 \% PG 64-22 + 0.0-1.5 \% lime $+0.0-0.5 \%$ anti-strip + limestone aggregates ( 12.5 mm NMAS dense-graded) (Avg OAC $=5.4 \%$ ) |
| Layer 6 | Base | Cement ( $2.0 \%$ ) or lime ( 3.0 to $6.0 \%$ ) treated subgrade soil materials (lime is typically added in liquid slurry form) |
| Subgrade |  | Compacted natural in-situ soil material |

Master Curves: The studied pavement section is comprised of five complex asphalt layers. The dynamic modulus data for all layers where obtained, analyzed, and used to develop the master curves and Prony series. Master-curves are used to predict the pavement modulus of elasticity or $E^{*}$ at the required temperatures (seasonal averages) using Arrhenius shift-factor equation:

$$
\begin{equation*}
\log \left(E^{*}\right)=\delta+\frac{\operatorname{Max}-}{1+e^{\beta+\gamma\{\log (t)-\log T\}}} \tag{10}
\end{equation*}
$$

Figure 18 demonstrates the developed master curves for all of the 5 AC layers, which were obtained based on the dynamic modulus data conducted at different temperatures.


Figure 18. Developed master curve for the rich bottom layer porous friction coarse (PFC), stone matrix asphalt (SMA), stone-filled hot mix asphalt (SFHMA), SFHMA rut-resistance layer (RRL), and rich-bottom layer (RBL).

Prony Series: As discussed before, viscoelastic materials physical characteristics are highly dependent on the temperature and loading rate (frequency). Depending on the material properties and testing temperature, viscoelastic materials behave linearly at very small strains (up to 100 microstrain) (70). Linear viscoelasticity can be mathematically characterized using various proposed models. The generalized Maxwell model (or the Maxwell-Wiechert model) has been used frequently used to characterize linear viscoelasticity (71,72). Mazurek and Iwaski evaluated the accuracy of different models to characterize AC viscoelasticity within the linear region and concluded that the generalized Maxwell model has the highest accuracy compared to other models $\left(r^{2}=0.995\right)(73)$. The generalized Maxwell model is basically comprised of a spring being attached to a set of Maxwell components in parallel and can be mathematically defined as (74):

$$
\begin{equation*}
E(t)=E_{0}+E_{1} e^{-\frac{t}{\tau_{1}}}+E_{2} e^{-\frac{t}{\tau_{2}}}+\cdots \tag{11}
\end{equation*}
$$

where:
$E_{i}$ : Elastic modulus constant of the spring i;
$\tau_{j}$ : Relaxation time for the dashpot j ; and
$t$ : Loading time.

## FEM Model Development:

In this study, three-dimensional finite element models are developed using Dassault SIMULIA ABAQUS 2017 to simulate the movement of real rotating truck wheel on the surface of a pavement section. Based on the structural layer configuration, axle weight and distribution analysis, viscoelastic-elastic characteristics, and temperature data, 2 separate FEM models were considered to assess fatigue and permanent deformation (rutting) damage (Figure 19). The specifications of each model are provided in Table 7.


Figure 19. Model development flowchart and critical points to assess rutting and fatigue damage.
Mesh Size: Mesh size selection has a significant effect on finite element modeling and each model requires a specific mesh size value to deliver acceptable results. Some research aim at mesh size optimizations and recommended a range of 15 to 25.4 mm for AC pavement simulation (75-77). However, mesh size is unique for each model and should be determined using field or experimental data.

In this research, the size of mesh was determined in order to increase the accuracy while keeping the analysis time within a practical range. To this end, the meshing near the tire loading area was set to be finer while a coarser mesh was used for the out-of-wandering area. a $2 \times 2$-inch and $2 \times 4$ mesh sizes were selected for the loading area ( 40 -inch width) and the out-of-wandering area, respectively.

Table 7. Specifications of the static and the dynamic model.

|  | Fatigue damage analysis | Permanent deformation damage <br> analysis |
| :---: | :---: | :---: |
| Dimension | 100 -meter long model | $300 \times 300$-inch model |
| Material Characteristics | Viscoelastic material | Elastic material |
| Loading Type | Dynamic loading | Static loading |
| Other Considerations | Finer mesh in the rolling <br> area | Uniform meshing |

Tire Configuration: In order to the assess the effects of axle configuration, 3 different configurations including single, tandem (double tire), and tridem (double tire) are evaluated. Based on the values considered in the MEPDG and also the axle distance for regular truck type 3S2, the axle distance was determined equal to 4 ft (center to center distance). Also, there are recommendations for tire wheel distance (in double tire configuration) and tire side-clearance (78) and the suggested tire side-clearance values vary from 1 to 2 inches. In this model a side-clearance of 1.5 inches (or a 12.5 -inch center to center side distance) is selected. Figure 20 shows tire configuration for a tandem axle and internal pressure arrows to simulate the tire inflation pressure.


Figure 20. Tandem axle configuration and inflation pressure.
Fatigue Analysis: The wheel load related cracking or fatigue is one of the fundamental factors that is considered in mechanistic-empirical (M-E) design of pavements. Fatigue occurs due to the tensile strain at the surface and bottom of the asphalt mixture layer, which gradually leads to longitudinal and alligator cracking, respectively. Based on the Palmgren-Miner hypothesis, the accumulated fatigue damage can be calculated using the equation below (79):

$$
\begin{equation*}
F D=\sum_{1}^{k} \frac{n_{i, j, k, l, m}}{N_{i, j, k, l, m}} \tag{12}
\end{equation*}
$$

where:
$n_{i, j, k, l, m}=$ Number of applied axle load for condition $i, j, k, l, m$;
$N_{i, j, k, l, m}=$ Number of allowed pass of axle load to reach fatigue cracking failure for condition $i, j, k, l, m$;
$i=$ the specific month or any considered time-span to account for temperature or moisture variations;
$j=$ time of the day;
$k=$ axle load configuration;
$l=$ axle load magnitude; and
$m=$ wheel path lateral wandering condition. (79)
Finn et al. implemented an alteration of AI pavement design approach to predict the number of standard load axles to reach fatigue cracking failure. Finn et al.'s proposed equation benefits the fatigue life prediction by taking into account the AC mixture's volumetric properties. The equation can be defined as follows:

$$
\begin{equation*}
N_{f}=K_{1} \times \varepsilon_{t}^{-3.9492} \times E^{-1.281} \tag{13}
\end{equation*}
$$

where:
$N_{f}$ : Number of allowed load axles to reach fatigue cracking failure;
$K_{2}, K_{3}$ : Constant fitting parameters equal to 3.291 and 0.854 , respectively; and
$K_{1}$ : Fitting parameter related to pavement thickness and mixture volumetric parameters.

$$
\begin{align*}
& K_{1}=0.00432 \times k_{1}^{\prime} \times C  \tag{14}\\
& C=10^{M}  \tag{15}\\
& M=4.84 \times\left(\frac{V_{b}}{V_{a}+V_{b}}-0.69\right) \tag{16}
\end{align*}
$$

where:
$V_{b}=$ air volume divided by the total volume of the mixture; and $V_{a}=$ Volume of the mix divided by the total volume of the mixture.
For bottom-up fatigue cracking:

$$
\begin{equation*}
k_{1}^{\prime}=\frac{1}{0.00039 \frac{0.003602}{1+e^{11.02-3.49 \times h a c}}} \tag{17}
\end{equation*}
$$

For top-down fatigue cracking:

$$
\begin{equation*}
k_{1}^{\prime}=\frac{1}{0.01+\frac{12}{1+e^{15.676-.8186 \times h_{a c}}}} \tag{18}
\end{equation*}
$$

where:
$h_{a c}=\mathrm{AC}$ thickness in inches.

Permanent Deformation Analysis: In NCHRP 1-37A, Permanent deformation in a pavement structure is measured by adding up the plastic deformation in all substructures. To this end, the plastic strain is determined using the vertical resilient strain, number of axle pass for each axle group, and the pavement temperature. $k_{1}$ is the coefficient to account for the confinement condition of viscoelastic material with increase in depth.

$$
\begin{align*}
& P D=\sum_{i=1}^{n} \varepsilon_{p}^{i} h^{i}  \tag{19}\\
& \frac{p}{v}=k_{1} \times 10^{-3.4488} \times T^{1.5606} \times N^{0.479244}  \tag{20}\\
& k_{1}=\left(C_{1}+C_{2} \text { depth }\right) \times 0.328196^{\text {dept }}  \tag{21}\\
& C_{1}=-0.1039 \times h_{a c}^{2}+2.4868 \times h_{a c}-17.342  \tag{22}\\
& C_{2}=0.0172 \times h_{a c}^{2}-1.7331 \times h_{a c}+27.428 \tag{23}
\end{align*}
$$

where:
$T=$ Pavement temperature (at mid-depth);
$N=$ Number of loading cycles;
$k_{1}=$ Depth coefficient;
$\varepsilon_{p}=$ Plastic strain; and
$\varepsilon_{v}=$ Elastic vertical strain at mid-depth of the AC layer.
Generally, the constitutive models are implemented to interpolate and assess the deformation inelasticity and time-temperature dependency behavior of materials (80).

Overloading and Load Limits: The maximum load magnitudes for the interstate highways (including the $\mathrm{IH}-35$ ) are limited to 20,34 , and 80 kips for single, tandem, and the gross vehicle weight (GVW) (81). There is no limitation for tridem axles; however the bridge formula can be used to calculate the allowable axle loading for any number of consecutive axles (82):

$$
\begin{equation*}
W=500\left(\frac{L N}{N-1}+12 N+36\right) \tag{24}
\end{equation*}
$$

where:
$L=$ distance between the farthest (extreme) axles (ft.);
$N=$ Number of axles considered in an axle loading group; and
$W=$ The total weight allowed to be induced on the axle loading group (lb.).

## 5. ANALYSIS AND FINDINGS

### 5.1. Results of Corridor Level Analysis

The results of truck platooning scenarios are presented in Tables $8,9,10, \& 11$. In Tables $8 \& 10$, the simulation result of truck platoons in peak and off-peak hours are recorded. In Tables $9 \& 11$, the percentage change of the platoon scenario results from the reference scenario were estimated.

Table 8. Result of Simulation Scenario's (Peak hour).

| Time <br> period | MPR | Platoon <br> size | Intra <br> platoon <br> distance | TND | TTM | TTD | Fuel | Emission <br> CO2 | Emission <br> NOx | Emission <br> PM10 | TIT |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| reference | 0 | 0 | 0 | 1992.1 | 794.7 | 6381.7 | 3962.5 | 16.26 | 43.36 | 18227.5 | 282.119 |
| peak | 25 | 2 | 0.3 | 3432.5 | 825.6 | 7744.7 | 3992.05 | 16.47 | 43.92 | 18363.43 | 498.494 |
| peak | 50 | 2 | 0.3 | 1972.8 | 827.9 | 6217 | 3953.05 | 16.44 | 43.84 | 18184.03 | 269.741 |
| peak | 100 | 2 | 0.3 | 2335.7 | 910 | 5974.8 | 3951.65 | 16.44 | 43.84 | 18177.59 | 373.161 |
| peak | 25 | 3 | 0.3 | 2613.2 | 838.9 | 7759.2 | 3981.8 | 16.56 | 44.16 | 18316.28 | 343.406 |
| peak | 50 | 3 | 0.3 | 1975.9 | 779.2 | 7098.4 | 3968.65 | 16.47 | 43.92 | 18255.79 | 265.8 |
| peak | 100 | 3 | 0.3 | 4039.2 | 832.6 | 7312.4 | 4014.5 | 16.38 | 43.68 | 18466.7 | 552.981 |
| peak | 25 | 2 | 0.5 | 1934.8 | 763.8 | 6981.2 | 3918.4 | 16.26 | 43.36 | 18024.64 | 286.814 |
| peak | 50 | 2 | 0.5 | 2339 | 869.9 | 6071.7 | 3923.1 | 16.29 | 43.44 | 18046.26 | 347.584 |
| peak | 25 | 3 | 0.5 | 1836 | 876.2 | 7033.7 | 3960.2 | 16.35 | 43.6 | 18216.92 | 257.033 |
| peak | 50 | 3 | 0.5 | 2543.4 | 911.6 | 6464.7 | 3915.95 | 16.23 | 43.28 | 18013.37 | 391.923 |
| peak | 25 | 2 | 0.7 | 2177 | 907.5 | 6297.9 | 3895.55 | 16.17 | 43.12 | 17919.53 | 313.317 |
| peak | 50 | 2 | 0.7 | 1094.5 | 639 | 6432.2 | 3855.85 | 16.05 | 42.8 | 17736.91 | 140.76 |
| peak | 25 | 3 | 0.7 | 2320.3 | 751.6 | 6464.9 | 3897.5 | 16.08 | 42.88 | 17928.5 | 287.276 |
| peak | 50 | 3 | 0.7 | 2960.6 | 1058.9 | 5960.7 | 3875.6 | 16.08 | 42.88 | 17827.76 | 393.009 |

Table 9. Percent change from reference scenario (Peak hour).

| Time <br> period | Platoon <br> size | Intra <br> platoon <br> distance | MPR | TND | TTM | TTD | Fuel | Emission <br> CO2 | Emission <br> NOx | Emission <br> PM10 | TIT |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| peak | 2 | 0.3 | 25 | 72.31 | 3.89 | 21.36 | 0.75 | 1.29 | 1.29 | 0.75 | 76.7 |
| peak | 2 | 0.3 | 50 | -0.97 | 4.18 | -2.58 | -0.24 | 1.11 | 1.11 | -0.24 | -4.39 |
| peak | 2 | 0.3 | 100 | 17.25 | 14.51 | -6.38 | -0.27 | 1.11 | 1.11 | -0.27 | 32.27 |
| peak | 3 | 0.3 | 25 | 31.18 | 5.56 | 21.59 | 0.49 | 1.85 | 1.85 | 0.49 | 21.72 |
| peak | 3 | 0.3 | 50 | -0.81 | -1.95 | 11.23 | 0.16 | 1.29 | 1.29 | 0.16 | -5.78 |
| peak | 3 | 0.3 | 100 | 102.76 | 4.77 | 14.58 | 1.31 | 0.74 | 0.74 | 1.31 | 96.01 |
| peak | 2 | 0.5 | 25 | -2.88 | -3.89 | 9.39 | -1.11 | 0 | 0 | -1.11 | 1.66 |
| peak | 2 | 0.5 | 50 | 17.41 | 9.46 | -4.86 | -0.99 | 0.18 | 0.18 | -0.99 | 23.2 |
| peak | 3 | 0.5 | 25 | -7.84 | 10.26 | 10.22 | -0.06 | 0.55 | 0.55 | -0.06 | -8.89 |
| peak | 3 | 0.5 | 50 | 27.67 | 14.71 | 1.3 | -1.17 | -0.18 | -0.18 | -1.17 | 38.92 |
| peak | 2 | 0.7 | 25 | 9.28 | 14.19 | -1.31 | -1.69 | -0.55 | -0.55 | -1.69 | 11.06 |
| peak | 2 | 0.7 | 50 | -45.06 | 19.59 | 0.79 | -2.69 | -1.29 | -1.29 | -2.69 | -50.11 |
| peak | 3 | 0.7 | 25 | 16.48 | -5.42 | 1.3 | -1.64 | -1.11 | -1.11 | -1.64 | 1.83 |
| peak | 3 | 0.7 | 50 | 48.62 | 33.25 | -6.6 | -2.19 | -1.11 | -1.11 | -2.19 | 39.31 |

The visualization of truck platoon scenario results is presented in Figures 21, 22, and 23. In Figure 21, the percentage change result for peak and off-peak hours are shown. In Figures 22 \& 23, the change of performance indicators (PI) with MPR (market penetration rate/ \% equipped truck) for peak and off-peak hours are presented.

Table 10. Percent change from reference scenario (Off-Peak hour).

| Time <br> period | MPR | Platoon <br> size | Intra <br> platoon <br> distance | TND | TTM | TTD | Fuel | Emission <br> CO2 | Emission <br> NOx | Emission <br> PM10 | TIT |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| reference | 0 | 0 | 0 | 1261.8 | 690.4 | 5045.7 | 3242.7 | 13.77 | 36.72 | 14916.42 | 179.81 |
| Off-peak | 25 | 4 | 0.3 | 1390.5 | 695.9 | 4988.2 | 3262.25 | 13.89 | 37.04 | 15006.35 | 169.28 |
| Off-peak | 50 | 4 | 0.3 | 1030.6 | 702 | 5552.2 | 3212.65 | 13.74 | 36.64 | 14778.19 | 122.51 |
| Off-peak | 100 | 4 | 0.3 | 1053.2 | 702 | 4648.7 | 3235.25 | 13.77 | 36.72 | 14882.15 | 139.39 |
| Off-peak | 25 | 5 | 0.3 | 807.4 | 666.7 | 4915 | 3212.8 | 13.8 | 36.8 | 14778.88 | 98.916 |
| Off-peak | 50 | 5 | 0.3 | 1089.9 | 671 | 4453.8 | 3279.65 | 13.77 | 36.72 | 15086.39 | 116.25 |
| Off-peak | 100 | 5 | 0.3 | 1370 | 792.3 | 4734.3 | 3261.9 | 13.83 | 36.88 | 15004.74 | 189.65 |
| Off-peak | 25 | 4 | 0.7 | 957.6 | 665.7 | 5363.1 | 3262.4 | 13.8 | 36.8 | 15007.04 | 110.41 |
| Off-peak | 50 | 4 | 0.7 | 1000.2 | 639 | 4540.3 | 3127.05 | 13.38 | 35.68 | 14384.43 | 128.12 |
| Off-peak | 25 | 5 | 0.7 | 1109.5 | 750.6 | 4929.3 | 3112.65 | 13.38 | 35.68 | 14318.19 | 165.69 |
| Off-peak | 50 | 5 | 0.7 | 1153.5 | 671 | 4517.1 | 3219.45 | 13.62 | 36.32 | 14809.47 | 151.73 |
| Off-peak | 50 | 3 | 0.7 | 783.6 | 726.5 | 4183.2 | 3191.2 | 13.62 | 36.32 | 14679.52 | 106.99 |
| Off-peak | 25 | 3 | 0.7 | 1060.2 | 684.3 | 3933.8 | 3162.9 | 13.41 | 35.76 | 14549.34 | 135.82 |
| Off-peak | 50 | 5 | 0.5 | 1166.2 | 671 | 4319.1 | 3256.1 | 13.74 | 36.64 | 14978.06 | 162.34 |
| Off-peak | 25 | 4 | 0.5 | 1063.2 | 722.4 | 4884.1 | 3249.4 | 13.8 | 36.8 | 14947.24 | 136.86 |
| Off-peak | 50 | 4 | 0.5 | 863.6 | 671 | 4579.6 | 3167.5 | 13.5 | 36 | 14570.5 | 115.48 |
| Off-peak | 25 | 5 | 0.5 | 1065.4 | 662.7 | 5784.2 | 3198.35 | 13.68 | 36.48 | 14712.41 | 135.12 |
| Off-peak | 50 | 3 | 0.5 | 1168.5 | 694.9 | 4402.2 | 3216.1 | 13.68 | 36.48 | 14794.06 | 176.12 |
| Off-peak | 25 | 3 | 0.5 | 990.5 | 662.7 | 3925.8 | 3171.4 | 13.53 | 36.08 | 14588.44 | 127.59 |
| Off-peak | 50 | 3 | 0.3 | 1122.1 | 639 | 4592.7 | 3250.3 | 13.8 | 36.8 | 14951.38 | 144.05 |
| Off-peak | 25 | 3 | 0.3 | 1282.5 | 828.4 | 4091.2 | 3218.95 | 13.68 | 36.48 | 14807.17 | 181.59 |

Table 11. Percent change from reference scenario (Off-Peak hour).

| Time <br> period | Platoon <br> size | Intra <br> platoon <br> distance | MPR | TND | TTM | TTD | Fuel | Emission <br> CO2 | Emission <br> NOx | Emission <br> PM10 | TIT |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Off-peak | 3 | 0.3 | 25 | 1.64 | 19.99 | -18.92 | -0.73 | -0.65 | -0.65 | -0.73 | 0.99 |
| Off-peak | 3 | 0.3 | 50 | -11.07 | -7.44 | -8.98 | 0.23 | 0.22 | 0.22 | 0.23 | -19.89 |
| Off-peak | 4 | 0.3 | 25 | 10.2 | 0.8 | -1.14 | 0.6 | 0.87 | 0.87 | 0.6 | -5.86 |
| Off-peak | 4 | 0.3 | 50 | -18.32 | 1.68 | 10.04 | -0.93 | -0.22 | -0.22 | -0.93 | -31.87 |
| Off-peak | 4 | 0.3 | 100 | -16.53 | 1.68 | -7.87 | -0.23 | 0 | 0 | -0.23 | -22.48 |
| Off-peak | 5 | 0.3 | 25 | -36.01 | -3.43 | -2.59 | -0.92 | 0.22 | 0.22 | -0.92 | -44.99 |
| Off-peak | 5 | 0.3 | 50 | -13.62 | -2.81 | -11.73 | 1.14 | 0 | 0 | 1.14 | -35.35 |
| Off-peak | 5 | 0.3 | 100 | 8.58 | 14.76 | -6.17 | 0.59 | 0.44 | 0.44 | 0.59 | 5.47 |
| Off-peak | 3 | 0.5 | 25 | -21.5 | -4.01 | -22.2 | -2.2 | -1.74 | -1.74 | -2.2 | -29.04 |
| Off-peak | 3 | 0.5 | 50 | -7.39 | 0.65 | -12.75 | -0.82 | -0.65 | -0.65 | -0.82 | -2.05 |
| Off-peak | 4 | 0.5 | 25 | -15.74 | 4.63 | -3.2 | 0.21 | 0.22 | 0.22 | 0.21 | -23.89 |
| Off-peak | 4 | 0.5 | 50 | -31.56 | -2.81 | -9.24 | -2.32 | -1.96 | -1.96 | -2.32 | -35.78 |
| Off-peak | 5 | 0.5 | 25 | -15.57 | -4.01 | 14.64 | -1.37 | -0.65 | -0.65 | -1.37 | -24.85 |
| Off-peak | 5 | 0.5 | 50 | -7.58 | -2.81 | -14.4 | 0.41 | -0.22 | -0.22 | 0.41 | -9.72 |
| Off-peak | 3 | 0.7 | 25 | -15.98 | -0.88 | -22.04 | -2.46 | -2.61 | -2.61 | -2.46 | -24.46 |
| Off-peak | 3 | 0.7 | 50 | -37.9 | 5.23 | -17.09 | -1.59 | -1.09 | -1.09 | -1.59 | -40.5 |
| Off-peak | 4 | 0.7 | 25 | -24.11 | -3.58 | 6.29 | 0.61 | 0.22 | 0.22 | 0.61 | -38.6 |
| Off-peak | 4 | 0.7 | 50 | -20.73 | -7.44 | -10.02 | -3.57 | -2.83 | -2.83 | -3.57 | -28.75 |
| Off-peak | 5 | 0.7 | 25 | -12.07 | 8.72 | -2.31 | -4.01 | -2.83 | -2.83 | -4.01 | -7.86 |
| Off-peak | 5 | 0.7 | 50 | -8.58 | -2.81 | -10.48 | -0.72 | -1.09 | -1.09 | -0.72 | -15.62 |




Figure 21. Percentage change result visualization for Peak and Off-Peak hour.


| Scenario of peak period. platoon site of 2. and intra gaps of 05 |  |  |  |
| :---: | :---: | :---: | :---: |
| TND |  |  | Fisel Consumption |
|  |  | Eminsice PM10 |  |



Figure 22. Scenario Visualization (Peak hour flow).



Figure 23. Scenario Visualization (Off-Peak hour flow).
The results of the truck platooning scenarios were compared with the reference scenario. The result showed that the truck platooning may have a positive impact on operational, environmental, and safety aspects in off-peak hours. For example, if we have a platoon size of 5 with an intra-platoon distance of 0.3 and an mpr of $25 \%$, the TND will be reduced by $36 \%$ and the TIT will be reduced by $45 \%$ compared to the reference. Also, fuel consumption will be reduced by $1 \%$ compared to the reference.

In peak hours, the truck platoons may deteriorate the operational and safety aspects of traffic in the road network. High values of TND and TIT were observed in peak hours due to truck platoons (Fig. 21). For example, if we have a platoon size of 3 with an intra-platoon distance of 0.7 and an mpr of $50 \%$, the TND will be increased by $48.6 \%$ and the TIT will be increased by $39.3 \%$ compared to the reference. But the fuel consumption will be reduced by $2.2 \%$ compared to the reference.

### 5.2. Analysis and Result of Network Level Analysis

### 5.2.1. Data Analysis

Figure 24 presents examples of time-series of speed, acceleration, time headway, and space headway of the probe vehicles using ACC and their three immediate followers in the first run of data collection. According to this figure, the time-series of the vehicles using ACC (3541, 3544, and 3548) are more similar compared to the other three immediate following vehicles. This similarity is more noticeable for the time headway (figure 25.c) and space headway (figure 25.d) series.


Figure 24. Time-space diagram of the vehicles for ten minutes on the Southbound of I-35, Austin, TX, during the morning peak period.

Flow-density plots with aggregation level of 30 seconds for individual lanes as well as their average are presented in Figure 26. According to these plots, the traffic dynamics of the leftmost lanes (Figures 26.a, 26.b) are different from the traffic dynamics of the rightmost lanes (Figures 26.c, 26.d). The flow and density data points for the lanes one and two (leftmost lanes) are more in the congested region compared to the data points for the lanes three and four (rightmost lanes).

These flow-density graphs (and associated speed-density curves) are utilized in the simulation (see the next section) to capture the effects of platooning on traffic flow dynamics. Considering the demand level at the I- 35 corridor (around $6000 \mathrm{veh} / \mathrm{hr}$ in this location), the simulation is setup to utilize the collected data to ensure realistic movement of vehicles across the corridor.

### 5.2.2. Impact of Truck Platooning on I-35

This section presents a simulation effort to capture the effects of truck platooning on I-35 in Austin, TX. I-35 is among the nation's interstate highways with highest truck traffic. The high truck volume on I-35 creates a lot of issues in Austin, TX. Austin, a fast-growing city in Texas, already faces congestion problem. The increasing truck traffic through this city just adds to the complication of managing congestion throughout this city. While truck platooning has been mainly proposed for energy consumption reduction, it is among the technologies that can potentially ease the negative impacts of truck traffic on a corridor. Accordingly, in this section, we will provide an assessment on the impacts of truck platooning on traffic flow dynamics on I-35 in Austin, TX. The simulation setup contains the section that was utilized in the data collection. Such a selection ensures the validity of using the generated speed-density (and flow-density curves) in the simulation platform.

Figure 27 illustrates the impact of platooning on traffic flow dynamics. All the simulations in this figure are conducted with $15 \%$ trucks in the traffic stream and the size of the platoon does not change during the simulation. This figure shows that as the size of platoons decreases, the scatter in fundamental diagram decreases and the traffic flow becomes more stable. This can be considered trivial as longer truck platoons introduce certain complications for other drivers. Moreover, lane-changing becomes increasing difficult as the size of the platoon increases, which can result in shockwaves and scatter in flow-density curves. Note that flow-density curves (and traffic flow dynamics) do not change significantly for platoon sizes over 4-trucks, as any lanechanging for the platoon becomes almost impossible for 4-trucks and more platoons.



Figure 25. Trajectory data example of ACC vehicles and their followers.
Figure 28 illustrates the flow-density curves for various penetration rates of 3-truck platoons. Similar to Figure 27, it is also assumed that the formation of the platoon remains the same throughout the segment. The figure shows that as the number of truck platoons increases, congestion improves. In other words, higher number of truck platoons translates into less scatter in fundamental diagram. This finding contradicts the general assumption that higher number of trucks can result in more congestion. In fact, this finding shows that autonomous truck platooning can help mitigate congestion, although the impact is not as obvious as autonomous passenger vehicles.

(a) Lane 1

(c) Lane 3

(e) Overall

Figure 26. Flow-density plots for each lane with aggregation resolution of 30 seconds.

(b) Lane 2

(d) Lane 4


Figure 27. Flow-density plots for different platoon sizes.



Figure 28. Flow-density plots for different market penetration rates of autonomous trucks (3-truck platoons).

### 5.3. Results and Discussion of Pavement Analysis

Table 12 demonstrates the analysis levels for load magnitude, number of axles, tire speed, pavement temperature, wander control, tire pressure. Load magnitudes are considered in way to the maximum, mode, and allowable load magnitude for single, tandem, and tridem axles. The overloading effect is evaluated using the maximum load magnitude.

As previously stated, the tire wandering follows a normal distribution pattern, and the fixed-path truck platooning is analyzed as the worst-case scenario. Although some other patterns have been suggested (83), yet their implementation feasibility from the safety and technology standpoints are not clearly validated.

In this analysis, the tire speed level is considered equal to 70 mph with reference to speed limit regulations for IH-35 (San Antonio to Austin) (84). Tire inflation pressure levels were selected based on the literature review and common truck tire inflation pressure range $(57,85)$. The MEPDG design considers a tire inflation or contact pressure of 120 psi which represent the tire pressure of heavy trucks during hot days (86). Figure 29 illustrates a tridem axle simulation using ABAQUS software.


Figure 29. Tridem axle simulation using ABAQUS software.
Table 12. Analysis levels for each variable.

| Load Magnitude | No. of <br> Axles | Speed | Temperature | Wander <br> Control | Tire <br> Pressure |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Maximum load for <br> each type of axle | Single | 70 mph | Seasonal <br> average | Fixed pathway | 80 psi |
| Load with the <br> highest frequency | Tandem | 70 mph | Seasonal <br> average | Normal <br> wandering | 100 psi |
| Representative <br> Overload | Tridem | 70 mph | Seasonal <br> average | 120 psi |  |

### 5.3.1. Model Validation

The developed FEM model is validated using the field data obtained using multi-depth deflectometers (MDDs) at the bottom of the asphalt layer, bottom of the base layer, and the depth of 8 inches from the subgrade layer surface. Figure 30 shows the field-measured deflections for a 3S2 truck travelling at a speed of 70 mph . A schematic of 3 S 2 truck is also shown in the bottom of the figure. 3 S 2 truck is a 5 -axle vehicle comprised of a single steering axle and 2 tandem axles (a total of 4 axles). The axle loads for the test truck were assigned to be 10.5 and 34 kips for single and tandem axles, respectively.

In order to validate the developed FEM model, a single and tandem axle were separately simulated with same configuration and characteristics of the test truck. For the tandem axle, it is assumed that the 34 kips load is equally distributed among all tires (4.250 kips each).


Figure 30. The field-measured deflections for the test truck.
Figures 31 and 32 show the deflection values obtained for the 34 kips tandem axle on the PFC and subbase layer surface, respectively. Results show that the FEM model deflection value can approximate the field data values with acceptable accuracy. Figure 33 and 34 demonstrate the deflection results for the 10.5 kips steering axle (single axle/single tire) on the PFC and subbase layer surface, respectively. The obtained deflection on the PFC layer from the FEM model agrees with the field measurement value; however, the deflection value on the subbase layer ( 1.7 mils ) is less than the field-measured deflection. With respect to complex nature of AC mixtures and also various influential ambient parameters, it can be concluded that the developed FEM model with the current mesh size is capable of simulating the traffic loads with an acceptable error.


Figure 31. PFC layer surface deflection induced by the 34 kips tandem axle.


Figure 32. Subbase layer surface deflection induced by the 34 kips tandem axle.


Figure 33. PFC layer surface deflection induced by the 10.5 kips single axle (steering).


Figure 34. Subbase layer surface deflection induced by the $\mathbf{1 0 . 5}$ kips single axle (steering).

### 5.3.2. Overloading Effect

As discussed earlier, the MEPDG pavement design procedure accounts for both legal and overweight loads in terms of the loading intervals. Therefore, future overloading predictions are estimated based on the present truck overloading data. Nonetheless, the overloading effect can be still evaluated using the overweight axle loads. As previously noted, the FHWA weight limit for single and tandem axles are 20 and 34 kips , respectively. Due to the fact that neither the FHWA nor the Texas DOT has imposed weigh limits on tridem axle loads (except the 80 kips limit for GVW), the limit was selected based on the recommended magnitudes recommended by other states (87). Most of the states have recommended a range of 42 to 54 kips weight limit; hence, a weight limit of 54 kips is selected for tridem axles to eliminate any possible overestimation of overloading effect (Figure 35).


Figure 35. Tridem axle-loads distribution (the overweight axle-load frequencies are shown in red).
In order to evaluate the effects of overloading, the "weighted average" of overweight axles for single, tandem, and tridem are considered in the analysis. This value is simply obtained by averaging the overweight axle-loads with respect to their frequency. The red columns in Figure 35 show the overweight tridem axle-loads as well as their frequencies. Table 13 shows the representative axle loads and their frequencies.

Table 13. Overloading representative axle loads and their frequencies.

|  | Single Axle | Tandem Axle | Tridem Axle |
| :---: | :---: | :---: | :---: |
| Representative Axle Weight (kips) | 22 | 40 | 63 |
| Frequency (\%) | 1.0 | 2.8 | 3.7 |

### 5.3.3. Wandering Effect

As mentioned earlier, the stress and strain values would decrease with increase in the horizontal distance to loading point. This trend particularly lays emphasis on wheel path wandering effect and the resulting induced damage. By implementing wandering techniques, the structural impact
of a specific wandering loading axle can be equivalent to the impact of the lighter loading axle groups with a fixed-path traffic.

Some research is aimed to evaluate the wandering factors and patterns. Factors such as weather condition, time, truck load, traffic factors and road characteristics are considered to affect lateral wandering length and distribution. Buiter et al. evaluated the impact of the lane width on lateral wandering of vehicles and calculated shift factors to design the pavement thickness (88).

The MEPDG design also categorizes the axle load spectrum into specific intervals based on the axle type. The intervals for single, tandem, and tridem axles are determined to be 1000, 2000, and 3000 lb ., respectively.

Traffic estimation (i.e., number of applied loading cycles during pavement service life) for each axle interval is affected by wandering effect. Surveys demonstrated that the wandering behavior follow a normal distribution pattern $(88,89)$. The MEPDG thickness design method is also based on normally distributed lateral wander and implements this concept to statistically predict a realistic equivalent damage during pavement service life. Based on the Palmgren-Miner hypothesis, the MEPDG averages the fatigue damage (FD) at 11 different points along the lateral direction of moving tire. The accumulated fatigue damage can be calculated using the PalmgrenMiner equation (88):

$$
\begin{equation*}
D=\sum_{1}^{k} \frac{n_{i}}{N_{i}} \tag{25}
\end{equation*}
$$

where:
$\mathrm{D}=$ Fatigue damage;
$n_{i}=$ standard load; and
$N_{i}=$ Service life.
The tire wander is considered to have a normal distribution and the variation of the distribution is defined using a user defined standard deviation. The points are selected in a way to divided the distribution area into 10 equal-area segments which they account for 10 percent of the traffic volume. Figure 36 illustrates this concept using 5 points.
(merge with paragraph above) For the fixed-path truck platooning, the maximum strain (vertically below the tire) would be considered for fatigue or rutting damage. In contrast, for a normally distributed tire-wander, the maximum strain is only applied on $20 \%$ of traffic counts for any specific axle load. Wandering intervals are determined using the standard deviation multiplied by the distribution coefficient for that specified location. In the example provided in the MEPDG manual (Figure 36), the suggested distribution factors are $-1.28155,-0.5244,0,0.5244,1.28155$. selection


Figure 36. Wandering considerations using 5 points (86).
In addition to inability of MEPDG to consider the realistic dynamic effect of the traffic loads, another major drawback of The MEPDG method is the limited number of points to calculate the stress and strain (i.e., the induced damage). Due to the complex nature of pavement analysis and significant variation of pavement structural response within trivial distance, number of intervals and assigned evaluation points are critical for future damage estimation.

In contrast, the finite element model is not only capable of simulating the dynamic loads, but it also can measure the structural impact of lateral wandering for unlimited points (depending on the mesh size) at any depth and any distance perpendicular to moving direction. Hence, implementation of the FEM in pavement analysis can provide a more realistic view of complex nature of AC pavements and a more reliable design over its service life.

In this approach, the evaluation points across the wheel path are not defined based on the equal area concept but they are determined based on the mesh fineness. For the assigned mesh size in this study ( 2 -inch mesh size), the normal area between each consecutive lateral mesh are calculated using a normal distribution table. The MEPDG suggests a 10 -inch wandering standard deviation; therefore, every evaluation would be evenly spaced on the normal distribution at 0.2 (2/10) intervals.

### 5.3.4. Damage Analysis

Fatigue Analysis: Based on the MEPDG manual, the bottom-up fatigue damage is determined using parameters including strain magnitude at the bottom of the AC layer, stiffness (Young's modulus), thickness, and volumetric properties of the AC layer. As mentioned in the previous
chapter, the tensile strain for each loading condition was measured using a dynamic FEM model. Figure 37 shows the FEM simulation of tandem axle.

From a structural point of view, the maximum tensile strain does not necessarily occur along the predefined horizontal axes ( $x$ and $z$ directions shown in Figure 37) and instead, the principal tensile strain should be considered in the analysis to account for the shear strains. Figures 38 and 39 show the lateral and maximum principal tensile strain for a 40 kips tandem axle and Figures 40 and 41 demonstrate the vertical and maximum principal tensile strain for a 63 kips tridem axle.


Figure 37. Dynamic loading and axle configuration for tandem axle.


Figure 38. Lateral strain for the 40 kips tandem axle.


Figure 39. Maximum principal tensile strain for a 40 kips tandem axle.


Figure 40. Vertical strain for the $\mathbf{6 3}$ kips tridem axle.


Figure 41. Maximum principal tensile strain for a 63 kips tridem axle.


Figure 42. Maximum principal strain at the bottom of SFHMA1' layer.
As shown in Figure 42, the principal tensile strain increases as the probe node gets laterally closer to tire position then it decreases to a minimum between double tires. Higher inflation pressure induces higher maximum tensile strain at the wheel-path and lower magnitudes at around 20 inches away from the wheel center. Based on the fatigue damage analysis, tire inflation does not affect the fatigue damage significantly. For tire inflation pressure 120, 100, and 80 psi the ratio of fatigue life (FatigueLife fixed-path/ $^{\text {/FatigueLife }}$ normal-distribution, 20 -year) ) are $14.6,14.7$, and $15 \%$.

Based on the measured maximum tensile principal strain values (Table 14) at the bottom of the AC critical layer (SFHMA1' layer), it can be concluded that wandering can have influential effect on the fatigue life for single, tandem and, tridem can be decreased in a range of $14 \%$ to $35 \%$, in terms of ratio of fatigue life (FatigueLife fixed-path $/$ FatigueLife $_{\text {normal-distribution, }}$ 20-year).

Table 14. Fatigue life ratio (FatigueLifefixed-path/FatigueLifenormal-distribution, 20-year) for single, tandem, and tridem.

| Axle Weight | Maximum load for each <br> type of axle | Load with the highest <br> frequency | Representative <br> Overload |
| :---: | :---: | :---: | :---: |
| Single | $32.3 \%$ | $29.0 \%$ | $34.5 \%$ |
| Tandem | $15.1 \%$ | $14.7 \%$ | $14.9 \%$ |
| Tridem | $14.7 \%$ | $13.9 \%$ | $15.6 \%$ |

Permanent Deformation: Similar to fatigue damage, the MEPDG permanent deformation evaluation is affected by induced strain levels and mechanical properties of the AC layer. As mentioned before, in this study, a static-loading method is used to measure the elastic vertical strain at mid-depth all AC layers. Figure 43 shows the static-loading FEM model for the tridem axle configuration. Figures 44,45 , and 46 demonstrate the vertical compressive strain values in
the AC layers and Figures 47 and 48 demonstrate the vertical strain values on the subbase and subgrade layers, respectively.


Figure 43. Static loading and axle configuration for tridem axle.


Figure 44. Vertical strain (LE22) on SFHMA1/2' for tridem axle.


Figure 45. Vertical strain at the SFHMA1' layer surface for tridem axle.


Figure 46. Vertical strain at the RBL layer surface for tridem axle.


Figure 47. Vertical strain (LE22) at Subbase surface for tridem axle.


Figure 48. Vertical strain (LE22) at subgrade surface for tridem axle.


Figure 49. Maximum vertical strain at mid-depth of PFC, SMA, SFHMA1/2', SFHMA1', and RBL.
As shown in Figure 49, the maximum vertical strain at mid-depth of the AC layers descends for layers located at larger depth. This can be explained by fact that with increase in depth, the force magnitude is distributed over a larger area and the induced stress magnitude would be decreased.

Figure 50 illustrates the relative permanent deformation damage based on the normal distribution (respecting to the distance from center of highest strain). As it can be seen, the relative damage for the left peak is higher since the it has higher frequency with regards to the normal distribution.


Figure 50. The relative permanent deformation damage based on the normal distribution wandering for a tandem axle.

Table 15 demonstrates the calculated ranges of permanent deformation damage-ratio for the AC layers. On average, fixed-path wandering can increase the permanent deformation damage up to 2.47 times. Moreover, results show a decreasing range for the layers located at lower positions in the pavement structure. This increase can be explained by the flatter shape of strain distribution shown in Figure 49. As the difference between the maximum vertical strain and adjacent points decreases (flatter bell-shape), the effect of wandering would be reduced.

Table 15. Permanent deformation damage-ratio range for AC layers.

| Permanent Deformation <br> Damage Ratio for layers | PFC | SMA | SFHMA <br> $1^{\prime}$ | SFHMA <br> $1 / 2^{\prime}$ | RBL |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Normal Distribution/Fixed Path <br> Platooning | $2.6-2.9$ | $2.50-2.75$ | $2.35-2.4$ | $1.80-2.25$ | $1.2-1.9$ |

Based on the permanent deformation results, on average, the resulting rutting depth can be increased by a factor of 2.47 and can significantly decrease the pavement service life. Using the MEPDG equation for permanent deformation, Table 16 shows the anticipated service life assuming that the permanent deformation damage is the critical design criterion for this pavement section. By changing the design service life (years), the number of allowed load axles is adjusted to compensate for the increased number of maximum-strain loadings. Hence, the fixed-path platooning design life is defined as the year that permanent deformation depth for fixed-path is equal to permanent deformation depth for the normal distribution wandering over 20 years. Hence, Ratio of number of axle to reach the same failure criteria (PDFixed-path/PDnormal-distribution, 20-year) can be compared to assess the effect of wandering on pavement performance. As demonstrated in Table 16, fixed-path truck platooning can significantly accelerate the permanent deformation damage; on average, for the fixed-path platooning, the number of axles is reduced to $41 \%$ and pavement damage can reach to "PDnormal-distribution, 20-year" only during 4.6 years.

Table 16. Anticipated service life for fixed-path truck platooning.

| Pavement Age | PD $_{\text {Fixed-path }} / \mathbf{P D}_{\text {normal-distribution, 20-year }}$ |
| :---: | :---: |
| 4.5 | 0.9914 |
| 4.55 | 0.9973 |
| 4.6 | 1.0031 |

### 5.4. Economic Study

### 5.4.1. Economic Analysis of Operational, Environmental, and Safety impacts

A literature review was conducted to figure out the methods to convert the impacts of truck platooning and its performance indicators into monetary values. A report describing the road user costs, published by FHWA, provided very useful and direct measures of cost (90). The methodology to estimate the cost of the performance indicators are described below:

Operation (Network delay): The network delay cost was calculated by multiplying the daily vehicle hours delay of the study area during the study period with occupancy and value of person travel time. The formula is presented in the 2019 urban mobility report (91).

$$
\begin{equation*}
A P V D=D P V D \times V P T \times O \times A F \tag{26}
\end{equation*}
$$

where:
APVD = Annual Passenger Vehicle Delay Cost;
DPVD = Daily Passenger Vehicle Hours of Delay;
VPT $=$ Value of Person Time;
$\mathrm{O}=$ Vehicle Occupancy; and
$\mathrm{AF}=$ Annual Factor.

$$
\begin{equation*}
A C V D=D C V D \times V C T \times A F \tag{27}
\end{equation*}
$$

where:
ACVD = Annual Commercial Vehicle Delay Cost;
DCVD = Daily Commercial Vehicle Hours of Delay;
VCT $=$ Value of Commercial Vehicle Time; and
$\mathrm{AF}=$ Annual Factor.
According to the 2019 urban mobility report, the occupancy is 1.50 persons/vehicle, the value of person time is $\$ 18.12 /$ hour, and the value of commercial vehicle is $\$ 54.94 /$ hour (91).
So, the annual delay cost for our study will be (from Equation 26 and 27):

$$
\begin{aligned}
& =\mathrm{APVD}+\mathrm{ACVD} \\
& =\mathrm{TND} *[(18.12 * 1.5 * 0.96)+(54.94 * 0.04)] * 24 * 52 * 7 \\
& =247,152 * \mathrm{TND}
\end{aligned}
$$

Fuel: The formula for estimating Fuel estimate cost is provided in the 2019 urban mobility report (91).

$$
\begin{equation*}
A P V F=D F C \times P P V \times G \times A F \tag{28}
\end{equation*}
$$

where:
APVD = Annual Passenger Vehicle Fuel Cost;
DFC = Daily Fuel Consumption;
PPV = Percentage of Passenger Vehicle;
$\mathrm{G}=$ Gasoline Cost; and
$\mathrm{AF}=$ Annual Factor.

$$
\begin{equation*}
A C V F=D F C \times P C V \times D \times A F \tag{29}
\end{equation*}
$$

where:
ACVD $=$ Annual Commercial Vehicle Fuel Cost;

DFC = Daily Fuel Consumption;
PCV $=$ Percentage of Commercial Vehicle;
D = Diesel Cost; and
$\mathrm{AF}=$ Annual Factor.
From, U.S. Energy Information Administration, the avg. gasoline and diesel cost for 2019 was $\$ 2.691$ and $\$ 3.056$ respectively $(92,93)$.

So, the annual fuel cost for our study will be (from Equation 28 and 29):
$=$ Fuel consumption * $[(2.691 * 0.96)+(3.056 * 0.04)] * 24 * 52 * 7$
$=23,640 *$ Fuel consumption
Emission: The FHWA report provided a table to get the transportation emission cost per ton in 2010 dollars for different pollutants (90). According to table 32, per tonnage cost of $\mathrm{CO}_{2}$ is $\$ 37$, $\mathrm{NO}_{\mathrm{x}}$ is $\$ 16300$, and $\mathrm{PM}_{10}$ is $\$ 131800$.

The price deflator is calculated using implicit price deflators for 2019 and 2010 (94).
Price deflator $=$ Implicit price deflators for 2019 vs $2010=112.032 / 96.068=1.166$
So, the emission cost for our study will be:
$=\left(\mathrm{CO}_{2} * 37+\mathrm{NO}_{\mathrm{x}} * 16,300+\mathrm{PM}_{10} * 131,800\right) * 1.166 * 24 * 52 * 7$
$=376,889 * \mathrm{CO}_{2}+166,034,669 * \mathrm{NO}_{\mathrm{x}}+1,342,537,997 * \mathrm{PM}_{10}$, (emission unit: ton)
$=0.41545 * \mathrm{CO}_{2}+183.022 * \mathrm{NO}_{\mathrm{x}}+1479.8944 * \mathrm{PM}_{10}$, (emission unit: grams)
Safety: According to the FHWA report, one can estimate the cost of crashes if the crash rate is known (90). Tarko showed in his book that there exists a crash-conflict relationship (95). The crash-conflict relationship may be defined as:

$$
\begin{equation*}
\mathrm{Qc}=\Pi * \mathrm{Qn} \tag{30}
\end{equation*}
$$

where:
$\mathrm{Qc}=$ Number of crashes;
$\mathrm{Qn}=$ Number of observed traffic conflicts; and
$\Pi=$ Crash-conflict ratio.
Many researchers provided the crash-conflict ratio to be used in Equation 30 (96-98). So, if the observed conflict is known, one can convert the conflict to crashes using the above equation. In this study, we used the crash-conflict ration for unsignalized intersection mentioned in Table 5.2 of Hyden (97).

For our study, we used Time-integrated Time to Collision as a performance indicator. Mahmud et al. mentioned TIT in their paper as a relative probability of conflict, as it integrates the time to collision profile of drivers (99). So, we used TIT indicator as an alternate of observed conflict. So using the TIT indicator, we can estimate the number of crash events for our study. But we still do
not know the severity of the crashes. We investigated the historic crash data from 2014 to 2018 to get the relative percentage of fatal, injury, and PDO crashes. We used this percentage to convert the three crash cost into a single crash cost.

Table 17. Estimation of crash cost per event by severity.

| Severity | AdjustedCPI <br> $[\mathbf{C P I}(2019) / \mathbf{C P I}(2001)]$ | AdjustedECI <br> $[\mathbf{E C I}(2019) / \mathbf{C I}(2001)]$ | Cost/event (2019 dollars) |
| :--- | :--- | :--- | :--- |
| Fatalities | $255.657 / 177.1=1.4436$ | $137.475 / 85.95=1.6$ | $(1,277,640 * 1.4436)+(4,106,620$ <br> $-1,277,640) * 1.6=6,370,769$ |
| Injuries | 1.4436 | 1.6 | $(52,569 * 1.4436)+(98,752-$ <br> $52,569) * 1.6=149,781$ |
|  |  | 1.6 | $(6497 * 1.4436)+(7,800-6497) *$ <br> $1.6=11,464$ |
| PDO | 1.4436 |  |  |

We need to adjust the crash cost to 2019 dollars using CPI and ECI values (100,101). We estimated the crash cost per event using the following formula (90):

$$
\begin{equation*}
\mathrm{AC}=\mathrm{HC} * \mathrm{CPI}(2019) / \mathrm{CPI}(2001)+(\mathrm{CC}-\mathrm{HC}) * \mathrm{ECI}(2019) / \mathrm{ECI}(2001) \tag{31}
\end{equation*}
$$

where:
AC = Adjusted cost;
CPI $=$ Consumer Price Index;
ECI = Employment Cost Index;
HC = Human capital cost; and
$\mathrm{CC}=$ Comprehensive cost.
We got the human capital cost and comprehensive cost in 2001 dollars from the FHWA report (90). The calculation is shown in Table 17.

Now we can estimate the unit crash cost per event by multiplying cost/event with the associated severity percentage that we got from crash data (Table 18).

Table 18. The relative percentage of crash severities.

| Severity | Frequency | Percent |
| :--- | :--- | :--- |
| Fatalities | 42 | 0.42 |
| Injuries | 2699 | 26.86 |
| PDO | 7307 | 72.72 |

Unit crash cost/event $=$ Severity crash cost * relative percentage
$=(0.42 * 6,370,769+26.86 * 149,781+72.72 * 11,464) / 100$
$=75,325$
So, the safety cost for our study will be:
$=$ Crash events * Cost per event
$=\mathrm{TIT} *(\Pi * 24 * 52 * 7) * 75,325$
$=(37 \mathrm{E}-06 * 8736) * 75,325 * \mathrm{TIT}$
$=24,347 *$ TIT
Economic analysis results: The estimated costs were used to calculate the total cost of truck platooning impacts (e.g., safety, environmental and operational impacts) on highways for all the scenarios developed at the corridor level analysis.

First, Table 19 illustrates the total costs of truck platooning impacts for peak scenarios at corridor level analysis (e.g., impacts on pavement is not included). As shown in Table 19 that the total cost varies between 391 million and 1134 million USD. Note the cost of the reference scenario (e.g., no truck platooning) is about $\$ 620$ million. It was found that the worst-case occurred in the extreme scenario with $100 \%$ market penetration rate. The best recommended scenario during peak hours (total cost was $\$ 391$ million) is a combination of a platoon size of 2 trucks, market penetration ratio (MPR) of $50 \%$, and an intra-platoon distance of 0.7 s . By applying this scenario during peak hours, the total cost can be reduced from $\$ 619.88$ million to $\$ 391.34$ million (about $58 \%$ decrease). It is worth mentioning that the safety cost plays an important role in this optimum scenario. Figure 51 also shows the total cost comparison of the peak hour scenarios.

Table 19. Total cost of truck platooning impacts for peak scenarios at corridor level analysis.

| Time <br> period | MPR | Platoon <br> size | Intra platoon <br> distance | Network <br> Delay Cost | Fuel <br> Cost | Emission <br> Cost | Safety <br> Cost | Total <br> Cost |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| reference | 0 | 0 | 0 | 492.35 | 93.67 | 26.98 | 6.87 | 619.88 |
| peak | 25 | 2 | 0.3 | 848.35 | 94.37 | 27.18 | 12.14 | 982.04 |
| peak | 50 | 2 | 0.3 | 487.58 | 93.45 | 26.92 | 6.57 | 614.52 |
| peak | 100 | 2 | 0.3 | 577.27 | 93.42 | 26.91 | 9.09 | 706.68 |
| peak | 25 | 3 | 0.3 | 645.86 | 94.13 | 27.11 | 8.36 | 775.46 |
| peak | 50 | 3 | 0.3 | 488.35 | 93.82 | 27.02 | 6.47 | 615.66 |
| peak | 100 | 3 | 0.3 | 998.3 | 94.9 | 27.34 | 13.46 | 1134 |
| peak | 25 | 2 | 0.5 | 478.19 | 92.63 | 26.68 | 6.98 | 604.49 |
| peak | 50 | 2 | 0.5 | 578.09 | 92.74 | 26.71 | 8.46 | 706.01 |
| peak | 25 | 3 | 0.5 | 453.77 | 93.62 | 26.97 | 6.26 | 580.62 |
| peak | 50 | 3 | 0.5 | 628.61 | 92.57 | 26.67 | 9.54 | 757.39 |
| peak | 25 | 2 | 0.7 | 538.05 | 92.09 | 26.53 | 7.63 | 664.3 |
| peak | 50 | 2 | 0.7 | 270.51 | 91.15 | 26.26 | 3.43 | 391.34 |
| peak | 25 | 3 | 0.7 | 573.47 | 92.14 | 26.54 | 6.99 | 699.14 |
| peak | 50 | 3 | 0.7 | 731.72 | 91.62 | 26.39 | 9.57 | 859.3 |



Figure 51. Cost comparison of peak scenarios (corridor level analysis).
Second, Table 20 illustrates the total costs of truck platooning impacts for off-peak scenarios at corridor level analysis (e.g., impacts on pavement is not included). As shown in Table 20, the total cost varies between 293 million and 447 million USD. The best recommend scenario during offpeak hours, with a total cost of 293 million, is a combination of a platoon size of 4 rucks, market penetration ratio (MPR) of $25 \%$, and an intra-platoon distance of 0.5 s . The second-best scenario, with a total cost of 300 million, is a combination of a platoon size of 4 , an mpr of 50 , and an intraplatoon distance of 0.3 s .

In Figure 52, the total cost comparison of the off-peak hour is shown. It can be observed that most of the scenarios are better than the reference scenarios for the off-peak hour. So, truck platooning can improve traffic safety, traffic operation, reduce vehicular emissions and fuel consumptions during off-peak hours. To have positive impacts from truck platooning during peak hours, it is recommended to minimize the size of truck platoon to two vehicles.

Table 20. Total cost of truck platooning impacts for off-peak scenarios at corridor level analysis.

| Time period | MPR | Platoon <br> size | Intra <br> platoon <br> distance | Network <br> Delay <br> Cost | Fuel <br> Cost | Emission <br> Cost | Safety <br> Cost | Total <br> Cost |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| reference | 0 | 0 | 0 | 311.86 | 76.66 | 22.08 | 4.38 | 414.97 |
| Off-peak | 25 | 4 | 0.3 | 343.66 | 77.12 | 22.21 | 4.12 | 447.12 |
| Off-peak | 50 | 4 | 0.3 | 254.71 | 75.95 | 21.88 | 2.98 | 355.52 |
| Off-peak | 100 | 4 | 0.3 | 260.3 | 76.48 | 22.03 | 3.39 | 362.21 |
| Off-peak | 25 | 5 | 0.3 | 199.55 | 75.95 | 21.88 | 2.41 | 299.79 |
| Off-peak | 50 | 5 | 0.3 | 269.37 | 77.53 | 22.33 | 2.83 | 372.07 |
| Off-peak | 100 | 5 | 0.3 | 338.6 | 77.11 | 22.21 | 4.62 | 442.54 |
| Off-peak | 25 | 4 | 0.7 | 236.67 | 77.12 | 22.22 | 2.69 | 338.7 |
| Off-peak | 50 | 4 | 0.7 | 247.2 | 73.92 | 21.29 | 3.12 | 345.54 |
| Off-peak | 25 | 5 | 0.7 | 274.22 | 73.58 | 21.2 | 4.03 | 373.03 |
| Off-peak | 50 | 5 | 0.7 | 285.09 | 76.11 | 21.92 | 3.69 | 386.81 |
| Off-peak | 50 | 3 | 0.7 | 193.67 | 75.44 | 21.73 | 2.6 | 293.44 |
| Off-peak | 25 | 3 | 0.7 | 262.03 | 74.77 | 21.54 | 3.31 | 361.65 |
| Off-peak | 50 | 5 | 0.5 | 288.23 | 76.97 | 22.17 | 3.95 | 391.33 |
| Off-peak | 25 | 4 | 0.5 | 262.77 | 76.82 | 22.13 | 3.33 | 365.05 |
| Off-peak | 50 | 4 | 0.5 | 213.44 | 74.88 | 21.57 | 2.81 | 312.7 |
| Off-peak | 25 | 5 | 0.5 | 263.32 | 75.61 | 21.78 | 3.29 | 363.99 |
| Off-peak | 50 | 3 | 0.5 | 288.8 | 76.03 | 21.9 | 4.29 | 391.01 |
| Off-peak | 25 | 3 | 0.5 | 244.8 | 74.97 | 21.6 | 3.11 | 344.48 |
| Off-peak | 50 | 3 | 0.3 | 277.33 | 76.84 | 22.13 | 3.51 | 379.81 |
| Off-peak | 25 | 3 | 0.3 | 316.97 | 76.1 | 21.92 | 4.42 | 419.41 |



Figure 52. Cost comparison of off-peak scenarios.

### 5.4.2. Economic Analysis of pavement damage due to wandering

Based on a study by Lee et al., the perpetual pavement (PP) economic analysis is compared against traditional pavements that are constructed in Texas (102). They implemented the FPS 21 and TxME software to design the PP located in 10 different districts. PPs are defined as a full-depth AC layers designed for a service of at least 50 years, which are not expected to experience notable full-depth rutting or bottom-up cracking. This concept is based on limiting the vertical and horizontal strain value on subgrade and at bottom of AC layer, respectively. However, the AASHTOWare design (as the MEPDG design software) is based on the predicted pavement distresses in future.

Two approaches are considered to analyze the effects of tire wandering pattern on the pavement: strain-value-based (TxME) and distress-based (MEPDG) (86). In the strain-value based approach it is assumed that the pavement does not require any major maintenance or reconstruction and only a 2 " AC overlay is constructed on the existing pavement every 12 years to improve ride quality, surface friction, or top-down cracking resistance (102). The distress approach depends on the critical type of pavement distress.


Figure 53. Maintenance strategies for the normal wandering (top) and fixed-path wandering (bottom).
In order to conduct the strain-value-based economic analysis, 2 cases are considered: the normal wandering and fixed-path wandering. For the normal wandering, it is assumed that a 2 " AC overlay is constructed on the existing pavement every 12 years to improve ride quality and surface friction.

For fixed-path wandering, due to the limited lateral movement of tires it is assumed that polishing occurs faster and the interval for overlay would be shorter. The 5.7-years reduced time for overlay interval is determined with respect to number of passing axles (Figure 53).

In the distress-based approach and based on the results previously obtained in permanent deformation and fatigue analysis, distress-based for fatigue and distress-based for rutting are considered for cases that the fatigue and permanent deformation (rutting) are the critical distress, respectively (Figure 54).

Table 21 demonstrates the "present" cost for strain-value-based (TxME) and distress-based (MEPDG) approaches for a 50 -year life cycle costs for the PP sections located in San Antonio. The provided costs include initial cost, recurring maintenance, and reconstruction. The strain-value-based approach with normal wandering shows the benchmark cost that is already considered for PP maintenance strategy. Based on the results of the fixed-path wandering on the studied pavement section (provided in previous sections), the present cost for strain-value-based (fixedpath wandering), distress-based for fatigue, and distress-based for rutting are $\$ 3,375,000$, $\$ 3,997,000$, and $\$ 3,076,000$ per mile, respectively. These values for the 1.3 miles section (constructed in 2006) are $\$ 3,245,000, \$ 3,764,000$, and $\$ 2,897,000$ per mile, respectively. On average, the calculated costs for the strain-value-based (fixed-path wandering), distress-based for fatigue, and distress-based for rutting values show an increase of $25 \%, 46 \%$, and $13 \%$.


Figure 54. Reconstruction strategy based on pavement condition for A) Fatigue distress and B) Permanent deformation.

Table 21. Present cost for strain-value-based and distress-based approaches per mile over a 50 -year life cycle costs.

| PP |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Section | Length $\left.$| Strain-Value |
| :---: |
| based (TxME) - |
| Normal |
| Wandering |$\quad$| Strain-Value |
| :---: |
| based |
| (TxME) - |
| Fixed Path |$\quad$| Distress based |
| :---: |
| (MEPDG) - |
| Fatigue Damage | | Distress |
| :---: |
| based |
| (MEPDG) - |
| Rutting | \right\rvert\,

## 6. CONCLUSIONS

Truck platooning is one of the most important applications of connected and autonomous vehicle technology. Truck platooning has great potential in addressing current challenges facing freight movements like reducing highway traffic congestions, energy savings, and increasing safety. This study investigated the impacts of truck platooning in the South-central region of the US. Both corridor and network-level simulation analyses were conducted to estimate the operational, environmental, and safety impact of truck platooning on highways. Finite element modeling was performed also to model the impact of truck platooning on highways pavement. An economic feasibility analysis was conducted then to quantify the impacts of truck platooning in monetary terms.

The corridor-level analysis result showed that the truck platooning deteriorates traffic operation, and safety during peak-hours period. A high TND and TIT value was observed in different scenarios. Therefore, it is recommended to minimize the size of platoon to two trucks only during peak hours considering the results of the economic analysis.

On the other hand, truck platooning performed very well in the off-peak hour period where most of the scenarios were observed to improve traffic operation, environment, and safety. The scenario presented the highest safety and second-highest operational improvement was comprised of a platoon size of 5 , an mpr of $25 \%$, and an intra-platoon distance of 0.3 s . The second-best scenario that showed the highest traffic operational improvement and second-highest safety improvement was comprised of a platoon size of 3 , an mpr of $50 \%$, and an intra-platoon distance of 0.7 s . It can be seen that if the mpr value is increased, the platoon size can be reduced to maintain the same operational and safety impacts of truck platooning.

The economic feasibility analysis was done for all the performance indicators to convert them into monetary value. The total cost for each scenario was estimated by summing up all the costs of performance indicators. From the results of the economic study, it was observed that in peak-hours, most of the scenarios performed poorly with high total cost compared to the reference scenario (human-driven trucks). On the contrary, in the off-peak hour, most of the scenarios showed improved performance with a lower total cost than the reference scenario. The two scenarios that cost the lower, consists of a platoon size of 4 trucks. So, in the off-peak hour, a platoon size of 4 may be suitable, as all the scenarios including a platoon size of 4 showed lower total cost than the reference.

In addition to the microscopic analysis, a large-scale analysis of the impacts of truck platooning on congestion and traffic flow dynamics is conducted. Accordingly, a simulation model of I-35 is developed. The model was calibrated based on the trajectory data collected from a segment of I35 near Austin, TX. The data contained the trajectory of multiple 3-vehicle platoons to better capture the impacts of platooning on traffic flow dynamics. The impacts of various market penetration rates of truck platooning as well as the size of the platoon on traffic flow dynamics were explored. The findings show that as the size of platoon decreases, the scatter in fundamental diagram decreases and the traffic flow becomes more stable. Moreover, with a fixed platoon size, higher penetration rate of autonomous trucks results in smoother traffic and less scatter in fundamental diagram.

The impact of truck platooning on pavement are addressed using the elastic and dynamicviscoelastic finite element method (FEM) models. The mechanical response obtained from the
simulations are implemented to predict the effects of platooning due to limited wandering (lateral movement of truck tires). Based on the calculated mechanical response values (strain) in AC layer and the economic analysis, the followings can be concluded:

- FEM models show promising capability of simulating AC layer responses under different types of loads. The implemented viscoelastic model was able to simulate the field performance of pavement within acceptable accuracy.
- Based on the results, it can be concluded that wandering can have influential effect on the fatigue life and permanent deformation damage, which failure to take account of it can impose significant maintenance costs on national and state road network.
- Fatigue life can be decreased in a range of $14 \%$ to $35 \%$, in terms of numbers of cycles over 20-year design life (FDFixed-path/FDnormal-distribution, 20-year).
- The ratio of number of axle to reach the same permanent deformation failure criteria (PDFixed-path/PDnormal-distribution, 20-year) are determined for all AC layers. Compared to normal distribution, for zero-wandering scenario, pavement design life can be decreased from 20 years to 4.6 years.
- Based on the economic analysis of pavement damage, a range of $13 \%$ to $46 \%$ increase in present construction-maintenance cost can be anticipated when fixed-path wandering is implemented in the truck platooning.


## REFERENCES

1. Federal Highway Administrator. Freight Analysis Framework Inter-Regional Commodity Flow Forecast Study - FHWA Freight Management and Operations. 2016.
2. Maks Inc. FAF4 Freight Traffic Assignment Report. 2016.
3. Calvert SC, Schakel WJ, van Lint JWC. Will Automated Vehicles Negatively Impact Traffic Flow? Journal of Advanced Transportation. 2017; 2017:3082781.
4. Rios-Torres J, Malikopoulos AA, editors. Impact of connected and automated vehicles on traffic flow. 2017 IEEE 20th International Conference on Intelligent Transportation Systems (ITSC); 2017 16-19 Oct. 2017.
5. Martin-Gasulla M, Sukennik P, Lohmiller J. Investigation of the Impact on Throughput of Connected Autonomous Vehicles with Headway Based on the Leading Vehicle Type. Transportation Research Record. 2019; 2673(5):617-26.
6. Moridpour S, Mazloumi E, Mesbah M. Impact of heavy vehicles on surrounding traffic characteristics. Journal of Advanced Transportation. 2015;49(4):535-52.
7. Schakel WJ, Arem Bv, Netten BD, editors. Effects of Cooperative Adaptive Cruise Control on traffic flow stability. 13th International IEEE Conference on Intelligent Transportation Systems; 2010 19-22 Sept. 2010.
8. Bergenhem C, Hedin E, Skarin D. Vehicle-to-Vehicle Communication for a Platooning System. Procedia - Social and Behavioral Sciences. 2012;48:1222-33.
9. Segata M, Bloessl B, Joerer S, Sommer C, Gerla M, Cigno RL, et al., editors. Towards intervehicle communication strategies for platooning support. 2014 7th International Workshop on Communication Technologies for Vehicles (Nets4Cars-Fall); 2014 6-8 Oct. 2014.
10. Gao S, Lim A, Bevly D. An empirical study of DSRC V2V performance in truck platooning scenarios. Digital Communications and Networks. 2016;2(4):233-44.
11. Xiao L, Wang M, Arem Bv. Traffic Flow Impacts of Converting an HOV Lane Into a Dedicated CACC Lane on a Freeway Corridor. IEEE Intelligent Transportation Systems Magazine. 2020;12(1):60-73.
12. Alam AA, Gattami A, Johansson KH, editors. An experimental study on the fuel reduction potential of heavy duty vehicle platooning. 13th International IEEE Conference on Intelligent Transportation Systems; 2010 19-22 Sept. 2010.
13. Liang K-Y, Mårtensson J, Johansson KH. When is it Fuel Efficient for a Heavy Duty Vehicle to Catch Up With a Platoon? IFAC Proceedings Volumes. 2013;46(21):738-43.
14. Sugimachi T, Fukao T, Suzuki Y, Kawashima H. Development of Autonomous Platooning System for Heavy-duty Trucks. IFAC Proceedings Volumes. 2013;46(21):52-7.
15. Liang K, Deng Q, Mårtensson J, Ma X, Johansson KH, editors. The influence of traffic on heavy-duty vehicle platoon formation. 2015 IEEE Intelligent Vehicles Symposium (IV); 201528 June-1 July 2015.
16. Lioris J, Pedarsani R, Tascikaraoglu FY, Varaiya P. Doubling throughput in urban roads by platooning**This research was supported by the California Department of Transportation and TUBITAK-2219 program. We thank Rene Sanchez of Sensys Networks for the data in Table 1, and Alex A. Kurzhanskiy, Gabriel Gomes, Roberto Horowitz and Sam Coogan and others in the Berkeley Friday Arterial seminar for stimulating discussions. IFAC-PapersOnLine. 2016;49(3):49-54.
17. Browand F, McArthur J, Radovich CJPrr. Fuel Saving Achieved in the Field Test of Two Tandem Trucks. 2004.
18. Tsugawa S. An Overview on an Automated Truck Platoon within the Energy ITS Project. IFAC Proceedings Volumes. 2013;46(21):41-6.
19. Lu X-Y, Shladover SE. Automated Truck Platoon Control and Field Test. In: Meyer G, Beiker S, editors. Road Vehicle Automation. Cham: Springer International Publishing; 2014. p. 247-61.
20. Yeo H, Skabardonis A, Halkias J, Colyar J, Alexiadis V. Oversaturated Freeway Flow Algorithm for Use in Next Generation Simulation. Transportation Research Record. 2008;2088(1):68-79.
21. Schakel WJ, Knoop VL, van Arem B. Integrated Lane Change Model with Relaxation and Synchronization. Transportation Research Record. 2012;2316(1):47-57.
22. Zhao L, Sun J. Simulation Framework for Vehicle Platooning and Car-following Behaviors Under Connected-vehicle Environment. Procedia - Social and Behavioral Sciences. 2013;96:91424.
23. Lu X, Lee J, Chen D, Bared J, Dailey D, Shladover S, editors. Freeway Micro-simulation Calibration: Case Study Using Aimsun and VISSIM with Detailed Field Data2014.
24. Lu X-Y, Kan X, Liu H, Lu X, Shladover S, Ferlis R. An Enhanced Microscopic Traffic Simulation Model for Application to Connected Automated Vehicles2018.
25. Xiao L, Wang M, van Arem B. Realistic Car-Following Models for Microscopic Simulation of Adaptive and Cooperative Adaptive Cruise Control Vehicles. Transportation Research Record. 2017;2623(1):1-9.
26. Ramezani H, Shladover SE, Lu X-Y, Altan OD. Micro-Simulation of Truck Platooning with Cooperative Adaptive Cruise Control: Model Development and a Case Study. Transportation Research Record. 2018;2672(19):55-65.
27. Kan X, Xiao L, Liu H, Wang M, Schakel WJ, Lu X-Y, et al. Cross-Comparison and Calibration of Two Microscopic Traffic Simulation Models for Complex Freeway Corridors with Dedicated Lanes. Journal of Advanced Transportation. 2019;2019:8618476.
28. Jo Y, Kim J, Oh C, Kim I, Lee G. Benefits of travel time savings by truck platooning in Korean freeway networks. Transport Policy. 2019;83:37-45.
29. Yang D, Kuijpers A, Dane G, der Sande Tv. Impacts of large-scale truck platooning on Dutch highways. Transportation Research Procedia. 2019;37:425-32.
30. Calvert SC, Schakel WJ, van Arem B. Evaluation and modelling of the traffic flow effects of truck platooning. Transportation Research Part C: Emerging Technologies. 2019;105:1-22.
31. Wang M, van Maarseveen S, Happee R, Tool O, van Arem B. Benefits and Risks of Truck Platooning on Freeway Operations Near Entrance Ramp. Transportation Research Record. 2019;2673(8):588-602.
32. Li Y, Wang H, Wang W, Xing L, Liu S, Wei X. Evaluation of the impacts of cooperative adaptive cruise control on reducing rear-end collision risks on freeways. Accident Analysis \& Prevention. 2017;98:87-95.
33. Zhao P, Lee C. Assessing rear-end collision risk of cars and heavy vehicles on freeways using a surrogate safety measure. Accident Analysis \& Prevention. 2018;113:149-58.
34. Tu Y, Wang W, Li Y, Xu C, Xu T, Li X. Longitudinal safety impacts of cooperative adaptive cruise control vehicle's degradation. Journal of Safety Research. 2019;69:177-92.
35. Elliott D, Keen W, Miao L. Recent advances in connected and automated vehicles. Journal of Traffic and Transportation Engineering (English Edition). 2019;6(2):109-31.
36. Kuijpers AGJ. Truck platooning: a framework to optimize traffic management near the port area of Rotterdam: Eindhoven University of Technology; 2017.
37. Hyden C. Traffic Conflicts Technique: State-of-the-art. Traffic Safety Work with Video Processing. HH Topp. Kaiserslauten, Germany, University Kaiserslautern. Transportation Department. 1996.
38. Derpanis KG. The harris corner detector. York University. 2004:1-2.
39. Lowe DG. Distinctive image features from scale-invariant keypoints. International journal of computer vision. 2004;60(2):91-110.
40. Bay H, Tuytelaars T, Van Gool L, editors. Surf: Speeded up robust features. European conference on computer vision; 2006: Springer.
41. Rublee E, Rabaud V, Konolige K, Bradski G, editors. ORB: An efficient alternative to SIFT or SURF. 2011 International conference on computer vision; 2011: Ieee.
42. Muja M, Lowe D. Fast library for approximate nearest neighbors (FLANN). git://github com/mariusmuja/flann git url: http://www cs ubc ca/research/flann. 2013.
43. Derpanis KG. Overview of the RANSAC Algorithm. Image Rochester NY. 2010;4(1):2-3.
44. Girshick R, Donahue J, Darrell T, Malik J, editors. Rich feature hierarchies for accurate object detection and semantic segmentation. Proceedings of the IEEE conference on computer vision and pattern recognition; 2014.
45. Lin T-Y, Goyal P, Girshick R, He K, Dollár P, editors. Focal loss for dense object detection. Proceedings of the IEEE international conference on computer vision; 2017.
46. Khajeh-Hosseini M, Devunuri S, Talebpour A, Hamdar SH, editors. Vehicle Trajectory Data Collection Using Aerial Videography. The 99th Annual Meeting of the Transportation Research Board of National Academies; 2020; Washington D.C.
47. Gillespie TD, Karamihas SM, Sayers MW, Hansen W. Effects of heavy-vehicle characteristics on pavement response and performance. Washington, D.C: Transportation Research Board; 1993 1993. 126 p .
48. Sarkar A. Numerical comparison of flexible pavement dynamic response under different axles. International Journal of Pavement Engineering. 2016;17(5):377-87.
49. Pais JC, Amorim SIR, Minhoto MJC. Impact of Traffic Overload on Road Pavement Performance. J Transp Eng. 2013;139(9):873-9.
50. Rys D, Judycki J, Jaskula P. Analysis of effect of overloaded vehicles on fatigue life of flexible pavements based on weigh in motion (WIM) data. International Journal of Pavement Engineering. 2016;17(8):716-26.
51. Gungor OE, Petit AMA, Qiu J, Zhao J, Meidani H, Wang H, et al. Development of an overweight vehicle permit fee structure for Illinois. Transport Policy. 2019;82:26-35.
52. Zaghloul S, White T. Guidelines for Permitting Overloads; Part 1: Effect of Overloaded Vehicles on the Indiana Highway Network. West Lafayette, IN: Purdue University; 19941994. Report No.: FHWA/IN/JHRP-93/05, 2042.
53. Noorvand H, Karnati G, Underwood BS. Autonomous Vehicles: Assessment of the Implications of Truck Positioning on Flexible Pavement Performance and Design. Transportation Research Record: Journal of the Transportation Research Board. 2017;2640(1):21-8.
54. Sampei M, Tamura T, Kobayashi T, Shibui N. Arbitrary path tracking control of articulated vehicles using nonlinear control theory. IEEE Trans Contr Syst Technol. 1995;3(1):125-31.
55. Erlingsson S, Said S, McGarvey T, editors. Influence of heavy traffic lateral wander on pavement deterioration2012.
56. Mshali MRS, Steyn WJ. Effect of truck speed on the response of flexible pavement systems to traffic loading. International Journal of Pavement Engineering. 2020:1-13.
57. Al-Qadi IL, Wang H, Yoo PJ, Dessouky SH. Dynamic Analysis and in Situ Validation of Perpetual Pavement Response to Vehicular Loading. Transportation Research Record. 2008;2087(1):29-39.
58. Todd KB, Kulakowski BTJTrr. Simple computer models for predicting ride quality and pavement loading for heavy trucks. 1989;1215:137-50.
59. Airey GD, Rahimzadeh B. Combined bituminous binder and mixture linear rheological properties. Construction and Building Materials. 2004;18(7):535-48.
60. Al-Qadi IL, Loulizi A, Elseifi M, Lahouar S. The Virginia Smart Road: The Impact of Pavement Instrumentation on Understanding Pavement Performance. Asphalt Paving Technology: Association of Asphalt Paving Technologists-Proceedings of the Technical Sessions. 2004;73:427-65.
61. Assogba OC, Tan Y, Sun Z, Lushinga N, Bin Z. Effect of vehicle speed and overload on dynamic response of semi-rigid base asphalt pavement. Road Materials and Pavement Design. 2019:1-31.
62. Forough SA, Nejad FM, Khodaii A. A comparative study of temperature shifting techniques for construction of relaxation modulus master curve of asphalt mixes. Construction and Building Materials. 2014;53:74-82.
63. Pellinen TK, Witczak MW, Bonaquist RF. Asphalt Mix Master Curve Construction Using Sigmoidal Fitting Function with Non-Linear Least Squares Optimization. Recent Advances in Materials Characterization and Modeling of Pavement Systems2003. p. 83-101.
64. Williams ML, Landel RF, Ferry JD. The Temperature Dependence of Relaxation Mechanisms in Amorphous Polymers and Other Glass-forming Liquids. Journal of the American Chemical Society. 1955;77(14):3701-7.
65. García G, Thompson MJCES, Illinois Center for Transportation Series. Hma Dynamic Modulus Predictive Models (a Review). 2007.
66. Glover C, Davison R, Domke CH, Ruan Y, Juristyarini P, Knorr D, editors. Development of a new method for assessing asphalt binder performance durability2001.
67. Walubita LF, Alvarez AE, Simate GS. Evaluating and comparing different methods and models for generating relaxation modulus master-curves for asphalt mixes. Construction and Building Materials. 2011;25(5):2619-26.
68. Walubita LF, Liu W, Scullion T. Texas perpetual pavements : experience overview and the way forward. 2010.
69. Walubita LF, Scullion T. Texas perpetual pavements : new design guidelines. 2010.
70. Graziani A, Cardone F, Virgili A, Canestrari F. Linear viscoelastic characterisation of bituminous mixtures using random stress excitations. Road Materials and Pavement Design. 2019;20(sup1):S390-S408.
71. Yang S-f, Yang X-h, Chen C-y. Simulation of rheological behavior of asphalt mixture with lattice model. Journal of Central South University of Technology. 2008;15(1):155-7.
72. Papagiannakis AT, Abbas A, Masad E. Micromechanical Analysis of Viscoelastic Properties of Asphalt Concretes. Transportation Research Record. 2002;1789(1):113-20.
73. Mazurek G, Iwański M. Modelling of Asphalt Concrete Stiffness in the Linear Viscoelastic Region. IOP Conference Series: Materials Science and Engineering. 2017;245:032029.
74. Gloeckle WG, Nonnenmacher TF. Fractional integral operators and Fox functions in the theory of viscoelasticity. Macromolecules. 1991;24(24):6426-34.
75. Williamson MJ. Finite element analysis of hot-mix asphalt layer interface bonding: Kansas State University; 2015.
76. Yoo J, Al-Qadi IL. The truth and myth of fatigue cracking potential in hot-mix asphalt: Numerical analysis and validation. Asphalt Paving Technology: Association of Asphalt Paving Technologists-Proceedings of the Technical Sessions. 2008;77:549-90.
77. Bodhinayake BC, Hadi MN. Analysis of flexible pavements by finite element modelling: a comparative study of numerical analysis with field data. Road Engineering Association of Asia and Australasia (REAAA) Conference; Tokyo, Japan2000.
78. Fitch JW. Motor truck engineering handbook. Anacortes, Washington: James W. Fitch; 1984.
79. Papagiannakis AT, Masad E. Pavement design and materials. Hoboken, N.J: John Wiley; 2008 2008. 542 p.
80. May D, Gordon A, Segletes D. The Application of the Norton-Bailey Law for Creep Prediction Through Power Law Regression2013.
81. Board TR. Regulation of Weights, Lengths, and Widths of Commercial Motor Vehicles: Special Report 267. Washington, DC: The National Academies Press; 2002. 285 p.
82. Federal Highway Administration. Bridge formula weights. 2019 Aug 2019.
83. Chen F, Song M, Ma X, Zhu X. Assess the impacts of different autonomous trucks' lateral control modes on asphalt pavement performance. Transportation Research Part C: Emerging Technologies. 2019;103:17-29.
84. Texas Department of Transportation. Speed Limits - 75 and 80 Mile Per Hour [Available from: https://www.txdot.gov/government/enforcement/speed-limits/approved.html.
85. Barriera M, Pouget S, Lebental B, Van Rompu J. In Situ Pavement Monitoring: A Review. 2020;5:18.
86. American Association of State H, Transportation O. Mechanistic-Empirical pavement design guide : a manual of practice. [Washington D.C.]: American Association of State Highway and Transportation Officials; 2020.
87. Compilation of Existing State Truck Size and Weight Limit Laws - Appendix A: State Truck Size and Weight Laws - FHWA Freight Management and Operations. 2015.
88. Buiter R, Cortenraad W, Eck AV, Rij HVJTRR. Effects of transverse distribution of heavy vehicles on thickness design of full-depth asphalt pavements. 1989.
89. Timm D, Priest A. Wheel Wander at the NCAT Test Track. 2005.
90. Mallela J, Sadavisam S. Work zone road user costs: Concepts and applications. United States. Federal Highway Administration; 2011.
91. Lasley P. 2019 URBAN MOBILITY REPORT. 2019.
92. U.S. Energy Information Administration. U.S. All Grades All Formulations Retail Gasoline Prices (Dollars per Gallon) [Available from: https://www.eia.gov/dnav/pet/hist/LeafHandler.ashx?n=PET\&s=EMM_EPM0_PTE_NUS_DPG $\& \mathrm{f}=\mathrm{M}$.
93. U.S. Energy Information Administration. U.S. No 2 Diesel Retail Prices (Dollars per Gallon) [Available from: https://www.eia.gov/dnav/pet/hist/LeafHandler.ashx?n=PET\&s=EMD_EPD2D_PTE_NUS_DP G\&f=M.
94. Bureau of Economic Analysis. National Data [Available from: https://apps.bea.gov/iTable/iTable.cfm?reqid=19\&step=3\&isuri=1\&nipa_table_list=13.
95. Tarko A. Measuring Road Safety with Surrogate Events: Elsevier; 2019.
96. Migletz D, Glauz W, Bauer K. Relationships between traffic conflicts and accidents volume IExecutive Summary. 1985.
97. Hydén C. The development of a method for traffic safety evaluation: The Swedish Traffic Conflicts Technique. 1987(70).
98. El-Basyouny K, Sayed TJSs. Safety performance functions using traffic conflicts. 2013;51(1):160-4.
99. Mahmud SM, Ferreira L, Hoque M, Hojati A. Application of proximal surrogate indicators for safety evaluation: A review of recent developments and research needs. IATSS Research. 2017;41.
100. U.S. Bureau of Labor Statistics. Consumer Price Index Historical Tables for U.S. City Average : Mid-Atlantic Information Office : U.S. Bureau of Labor Statistics.
101. U.S. Bureau of Labor Statistics. Employment Cost Index News Release. 2019.
102. Lee SI, Walubita LF, Hu S, Scullion T. Sustainable Perpetual Asphalt Pavements and Comparative Analysis of Lifecycle Cost to Traditional 20-Year Pavement Design. 2018.
