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DEVELOPMENT OF A PROPOSED OVERWEIGHT VEHICLE PERMIT FEE STRUCTURE IN ILLINOIS

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A report of the findings of

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Validation and Revision of Fees Charged for Oversize/Overweight Vehicle Permits

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16. Abstract

Permits are one of the most effective and common tools for state agencies to regulate the operation of overweight and oversize vehicles by ensuring the safety of passenger and freight traffic and minimizing damage to pavements and bridges while promoting commerce and the safe movement of goods and services. Although the State of Illinois uses a relatively comprehensive permit system, many of its parts have not been revised for more than 30 years. Therefore, the objective of this study was to revise the current permit system by evaluating up-to-date impacts of overweight vehicles. In this study, impacts of overweight (OW) vehicles were evaluated in three aspects (bridge, pavement, and traffic safety) and individual fees were developed for each. The most recent databases on infrastructure condition and state-of-the-art prediction/classification algorithms were employed to produce the realistic and up-to-date assessment of OW vehicles' impact. Finally, a combined permit fee was recommended as a function of miles to be traveled, as well as axle and weight information, by aggregating the calculated individual fees.

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EXECUTIVE SUMMARY

Highway agencies are responsible for protecting the billions of taxpayer dollars invested in highway infrastructure by enforcing federal and state regulations and policies. Permits are one of the most effective and common tools for state agencies to regulate the operation of oversized and overweight (OS/OW) vehicles. Properly designed OS/OW vehicle permit programs can ensure the safety of passenger and freight traffic and minimize damage to pavements and bridges, while at the same time promote commerce and expedite movement of goods and services. Although the State of Illinois uses a relatively comprehensive permit system, many of its parts have not been revised for more than 30 years. Therefore, the objective of this study was to update the current permit system by evaluating the current impacts of OS/OW vehicles.

In this study, overweight (OW) vehicles were evaluated based on their impacts on bridges, pavements, and traffic safety. These impacts were assessed by employing the most recent databases on infrastructure condition and state-of-the-art prediction/classification algorithms. Because of the lack of traffic data on vehicle size/dimensions, the assessment could be done only for OW vehicles.

Pavement performance models were developed using the Condition Rating Survey (CRS) data extracted from the Illinois Roadway Information System (IRIS) database. The developed models consider the effects of highway type (interstate or non-interstate), pavement surface type (hot-mix asphalt or Portland cement concrete), traffic loading (in terms of equivalent single-axle load [ESAL]), existing pavement condition before rehabilitation, and climate condition. Based on the maintenance strategy and costs used by IDOT, life-cycle cost analyses (LCCA) were performed to estimate pavement damage cost. The estimated pavement damage cost was found to vary significantly depending on truck traffic volume and highway type. The overall OW permit fee was estimated based on the assumptions of representative truck traffic volume, vehicle miles traveled on interstate highways, and the relative percentage of pavement types in the Illinois highway network.

A novel framework, which considers both applied load (i.e., gross weight of vehicles) and bridge load-carrying capacity, was introduced to quantify the impact of OW vehicles on bridges. The information about vehicle weight distribution on each bridge was obtained by developing Gaussian mixture models. These models are capable of estimating the weight distribution on each bridge from WIM data based on their location (i.e., latitude and longitude). Applied load information was later combined with data from the National Bridge Inventory (NBI) to develop prediction models that input bridge characteristics (e.g., age, location, inventory ration) and output bridge condition. These models were employed to calculate expected bridge service life with different loading scenarios to compute bridge life reduction per damaging load, which is defined as load greater than the load level that can safely utilize an existing structure for an indefinite period of time. As a final step, bridge life-cycle cost was conducted to convert calculated service life into a fee.

A set of customized Illinois-specific safety performance functions (SPFs) were developed to quantify the impact of OW trucks on the safety of Illinois roadways. Statistical tools were developed to quantify the relationships among total vehicle traffic, OW truck traffic, and expected crash frequency. A new network traffic flow model was developed to estimate the distribution of OW vehicle traffic on

Illinois’ roadway network, and state-of-art regression analysis was performed to develop the SPFs. After the SPFs were developed, the expected costs related to crashes by severity type (obtained from IDOT) were used to translate these SPFs into marginal safety costs caused by OW truck traffic. Costs vary with respect to crash severity, traffic exposure, and roadway type, while averages are computed across Illinois roadways to yield the overall safety cost of OW trucks.

Individual fees calculated for pavements, bridges, and safety aspects were aggregated, and a combined permit fee was recommended for OW trucks as a function of miles to be traveled as well as axle and weight information.

CONTENTS

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Freight transportation has grown over time with the increase in population and growth in economic activities within the United States. While the population of the United States climbed to 321.4 million in 2014 (United States Census 2015), the corresponding gross domestic product (GDP) increased to $17.95 trillion, and the GDP per capita increased to $55,836.8 (World Bank 2015).

The State of Illinois is a crossroads for the movement of goods to a large part of the country. In addition, Illinois has the third largest bridge inventory in the nation. Nevertheless, Illinois faces major transportation infrastructure needs that significantly impact the state's economy. According to the American Society of Civil Engineers' 2014 report card for Illinois' infrastructure (ASCE 2014), 42% of major roads are in poor or mediocre condition and 16% of bridges are structurally deficient or functionally obsolete. The same report grades bridge and roadway condition as C+ and D+, respectively.

While it is encouraging that the percentage of structurally deficient and functionally obsolete bridges in Illinois is lower than the national average, inevitable increases in the number of trucks and axle loads will degrade roads and bridges more rapidly. Trucks have traditionally been the most flexible shipping option for freight in the United States, and they have become an appealing option for transporting overdimension and overweight loads on state and federal highways.

According to the Federal Highway Administration (FHWA), the number of overweight load permits increased by a range of 30% to 50% in several states between 2000 and 2010. The federal government does not issue permits for overweight and oversize vehicles. Instead, this responsibility is delegated (by law) to the states. The permitting processes varies by state (e.g., some states issue permits for non-divisible overweight and oversize loads with no regard to the axle weight, gross vehicle weight (GVW), or the federal bridge formula (Appendix A).

A non-divisible load is defined as any load or vehicle exceeding the applicable length or weight limit if the carried load cannot be divided into smaller loads or vehicles. In non-divisible loads, dividing the load would compromise the intended use of the vehicle, diminish the value of the load or vehicle, or require more than eight work hours to dismantle using appropriate equipment. Both non-divisible loads, such as wind energy equipment, construction equipment or mobile houses, or traditionally defined divisible loads, such as agricultural goods, present several challenges to the transportation agency—ranging from operational and administrative cost, long-term damage of pavements and bridges, and safety concerns (FHWA 2015).

The higher impacts of OS/OW loads are not captured fully by existing permit fees or policies. To protect the tax dollars invested in highway infrastructure while encouraging commerce and the movement of goods and services, many states issue oversize and overweight (OS/OW) permits to compensate for the higher cost of damage (Adams et al. 2013). Illinois has established a relatively comprehensive overweight permit fee system that takes GVW, axle weight, and distance into account. However, many such fees have not been reviewed in the past 30 years. Therefore, there is a need to review the existing Illinois permit structure by analyzing the current economic impact of OS/OW vehicles on pavement, bridges and safety.

1.2 CURRENT STATUS OF OS/OW TRUCK PERMITS IN ILLINOIS

The Illinois Department of Transportation (IDOT) currently uses a permit fee system that is established by the Illinois Compiled Statutes (625 ILCS 5/15 Article III). This system calculates the fee for an overweight truck by considering the gross vehicle weight, axle weight and distance traveled. The summarized existing permit fee structure in Illinois is presented in Table 1.1 for oversize vehicles and Table 1.2 for overweight vehicles with weights less than practical maximums (Permits 2015). Although the current fee system is relatively comprehensive (as previously mentioned) many of its parts have not been revised for more than 30 years. Thus, there is a need to review the current fee system to make sure it fairly reflects the up-to-date infrastructure damage caused by OS/OW vehicles and their impact on the roadways safety.

Table 1.1: Current Fee Structure in Illinois for Oversize Vehicles

Category	Dimensions	0-90 mi	91-180 mi	181-270 mi	Over 270 mi
A	Maximum 10’ width 14’6” height 70’ length	$12	$15	$18	$21
B	Maximum 12’ width 14’6” height 85’ length	$15	$20	$25	$30
C	Maximum 14’ width 15’ height 100’ length	$25	$30	$35	$40
D	Maximum 18’ width 16’ height 120’ length	$30	$40	$50	$60
E	More than 18’ width 16’ height 120’ length	$50	$75	$100	$125

Table 1.2: Current Fee Structure in Illinois for Overweight Vehicles

Total Axles **GW (kips)** **F. tandem/axles** **R. tandem/axles**	≥ 6 88 34/2 48/3	≥ 6 100 44/2 54/3	≥ 6 110 44/2 54/3	≥ 6 120 48/2 60/3	5 88 44/2 44/2	5 100 44/2 44/2
0-45 mi	$10	$15	$20	$30	$20	$30
46-90 mi	$12.50	$25	$32.50	$55	$32.50	$55
91-135 mi	$15	$35	$45	$80	$45	$80
136-180 mi	$17.50	$45	$57.50	$105	$57.50	$105.0
181-225 mi	$20	$55	$70	$130	$70	$130
226-270 mi	$22.50	$65	$82.50	$155	$82.50	$155
271-315 mi	$25	$75	$95	$180	$95	$180
316-360 mi	$27.50	$85	$107.50	$205	$107.50	$180
361-405 mi	$30	$95	$120	$230	$120	$230
406-450 mi	$32.50	$105	$132.50	$255	$132.50	$255
451-495 mi	$35	$115	$145	$280	$145	–

In addition, the current fee system used in Illinois has inconsistencies with its neighboring states. These inconsistencies could affect the development of freight transportation economics across states. Table 1.3 briefly shows permit fees in Illinois and its neighboring states. These states impose various types of permit fees, and not all the states have an annual permit fee. Moreover, the trucking industry has expressed concern that raising fees would encourage trucking without permits and would jeopardize the effectiveness of weight enforcement.

A detailed explanation for the Illinois fee structure and its comparison with neighboring states' fee structures is given in Appendix A.

Table 1.3: Overweight Permit Fee in Illinois' Neighboring States

State	**Single Permit Fee**	**Annual Permit Fee**
Illinois	$10–$280	—
Iowa	$10	$300
Indiana	$20 + $0.35–$1.0 per mile	—
Kentucky	$60	$500
Missouri	$15 + $20 per every 10 kips	$300–$624
Wisconsin	$20–$105	$200–$850

Data source: J.J. Keller & Associates, Inc. 2011; and state departments of transportation

1.3 OBJECTIVES AND RESEARCH SCOPE

This study aimed at quantifying the impact of overweight vehicles on pavements, bridges, and safety. This allows more reliable assessment of the relative cost shares for damage caused by these vehicles when determining permit fees. This research validates and recommends revisions (where applicable) to current Illinois statues pertaining to OS/OW vehicle permit fees.

1.4 REPORT ORGANIZATION

The methodology followed in this study is to quantify the monetary impact of the OS/OW vehicles on pavements, bridges, and safety and then combine these impacts into a final permit fee system. Data-driven models have been exploited to develop comprehensive and generalized frameworks for analyzing OS/OW vehicles effects.

The report chapters are organized as follows:

Chapter 2 presents the databases used in this study. The databases are grouped into three categories based on their contexts: traffic (permit databases, weigh-in-motion database, traffic database), infrastructure (roadway inventory system, pavement condition database, National Bridge Inventory) and crash database.

Chapter 3 presents a methodology to allocate the pavement damage cost induced by overweight vehicles based on analysis of pavement performance data in Illinois. It aims in developing pavement performance model through considering the incremental change of performance deterioration due to the effects of traffic loading, climate, and condition before rehabilitation.

Chapter 4 introduces a framework to quantify the damage caused by OS/OW vehicles to bridges in the State of Illinois. This framework is built on two different databases: The National Bridge Inventory and Weigh-In-Motion (WIM). The machine-learning algorithms such as support vector machines and Gaussian mixture are used in this framework for conducting regression analysis.

Chapter 5 presents the methodology to assess the impact of OS/OW trucks on roadway safety. It includes developing a set of safety performance functions (SPF) to quantify the relationship between traffic volumes, roadway characteristics, and crash frequency and severity. State-of-the-art techniques are used for both data processing and regression analysis.

Chapter 6 introduces the fee calculator where the fee for an overweight vehicle is automatically calculated considering its impact on pavement, bridges and safety.

Chapter 7 summarizes the main conclusions and recommendations of this study.

CHAPTER 2: PROJECT DATABASE

2.1 INTRODUCTION

This study develops data-driven models that can accurately assess the impact of OS/OW vehicles on safety, pavement, and bridges. This assessment requires an understanding of load-related fatigue damage on bridges and the effects of size and weight of vehicles on crash numbers and characteristics. These problems should be addressed in the most comprehensive yet general way possible to account for the variability of inputs across the State of Illinois and avoid being limited to a few select cases. Therefore, in this study, the research team collected information, employed a variety of databases, and combined the data with state-of-the-art prediction/classification algorithms to accurately and comprehensively investigate the impact of OS/OW vehicles on Illinois highway transportation infrastructure.

The seven databases used in this study were classified into three categories based on their content: traffic data, infrastructure data, and crash data. The following sections summarize the databases and provide an overview of the extracted data.

2.2 TRAFFIC DATA

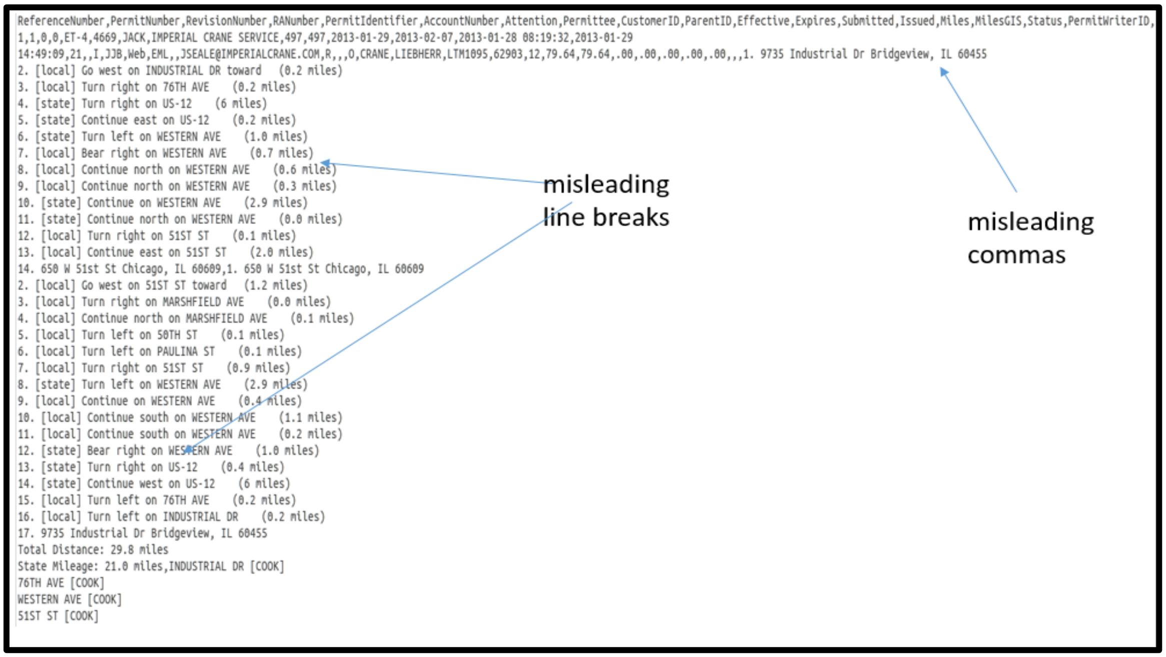
2.2.1 Traffic Database

The traffic database was used primarily to study the safety impacts of OW trucks on the Illinois roadway network. The database consists of three major components: traffic volume, which is one of the most critical component for developing safety impact models; frequency and severity of crashes; and roadway segment classification. Researchers from IDOT and CH2M Hill provided a large dataset containing traffic, roadway, and crash data. Traffic data in this database cover more than 60,000 roadway segments and spans from 1975 to 2012 (95% of the data are in the span from 2008 to 2012). Each roadway segment is associated with an inventory number and its corresponding average annual daily traffic (AADT) and heavy commercial vehicle (HCV) traffic. No data related to overweight trucks (OWT) were available in this dataset. (The methodology developed and used in this report to estimate the OWT traffic is presented in Section 5.2.)

2.2.2 Permit Data

The Illinois Department of Transportation (IDOT) has been using an online permit fee tool since 2013. This tool not only has significantly simplified the procedure for requesting a permit fee, but it also created a database where information about OS/OW vehicles is stored.

IDOT provided the research team with this database, which was in an unformatted text format. In other words, it was not in the format of any commonly known file structure, such as .csv or .xlsx. Therefore, development of a script that structures the data was needed. After visual inspection, it was determined that the provided databases could be converted into a .csv file if two issues were addressed. The first is the presence of *misleading commas*. For example, when origin of a vehicle was entered, a comma was used to separate street name from state name. This comma was flagged as misleading because it falsely separated one cell (i.e., origin column) into two cells (Figure 2.1). The second problem was defined as *misleading line breaks*. For instance, line breaks were used to divide vehicle route into different street names. These line breaks should be identified and removed; otherwise, each street name in route would be considered a different row, although all the street names belong to a cell called “route” for each OS/OW record (Figure 2.1).



```
ReferenceNumber,PermitNumber,RevisionNumber,RANumber,PermitIdentifier,AccountNumber,Attention,Permittee,CustomerID,ParentID,Effective,Expires,Submitted,Issued,Miles,MilesGIS,Status,PermitWriterID,
1,1,0,0,ET-4,4669,JACK,IMPERIAL CRANE SERVICE,497,497,2013-01-29,2013-02-07,2013-01-28 08:19:32,2013-01-29
14:49:09,21,,I,JJB,Web,EML,,JSEALE@IMPERIALCRANE.COM,R,,,0,CRANE,LIEBHERR,LTM1095,62903,12,79.64,79.64,.00,.00,.00,.00,.00,,,1. 9735 Industrial Dr Bridgeview, IL 60455
2. [local] Go west on INDUSTRIAL DR toward   (0.2 miles)
3. [local] Turn right on 76TH AVE    (0.2 miles)
4. [state] Turn right on US-12    (6 miles)
5. [state] Continue east on US-12    (0.2 miles)
6. [state] Turn left on WESTERN AVE    (1.0 miles)
7. [local] Bear right on WESTERN AVE    (0.7 miles)
8. [local] Continue north on WESTERN AVE    (0.6 miles)
9. [local] Continue north on WESTERN AVE    (0.3 miles)
10. [state] Continue on WESTERN AVE    (2.9 miles)
11. [state] Continue north on WESTERN AVE    (0.0 miles)
12. [local] Turn right on 51ST ST    (0.1 miles)
13. [local] Continue east on 51ST ST    (2.0 miles)
14. 650 W 51st St Chicago, IL 60609,1. 650 W 51st St Chicago, IL 60609
2. [local] Go west on 51ST ST toward   (1.2 miles)
3. [local] Turn right on MARSHFIELD AVE    (0.0 miles)
4. [local] Continue north on MARSHFIELD AVE    (0.1 miles)
5. [local] Turn left on 50TH ST    (0.1 miles)
6. [local] Turn left on PAULINA ST    (0.1 miles)
7. [local] Turn right on 51ST ST    (0.9 miles)
8. [state] Turn left on WESTERN AVE    (2.9 miles)
9. [local] Continue on WESTERN AVE    (0.4 miles)
10. [local] Continue south on WESTERN AVE    (1.1 miles)
11. [local] Continue south on WESTERN AVE    (0.2 miles)
12. [state] Bear right on WESTERN AVE    (1.0 miles)
13. [state] Turn right on US-12    (0.4 miles)
14. [state] Continue west on US-12    (6 miles)
15. [local] Turn left on 76TH AVE    (0.2 miles)
16. [local] Turn left on INDUSTRIAL DR    (0.2 miles)
17. 9735 Industrial Dr Bridgeview, IL 60455
Total Distance: 29.8 miles
State Mileage: 21.0 miles,INDUSTRIAL DR [COOK]
76TH AVE [COOK]
WESTERN AVE [COOK]
51ST ST [COOK]
```


Figure 2.1: Examples of misleading commas and line breaks.

Table 2.1 presents statistics about permit fees. As shown in the table, the permit fee ranges between \$1 and \$3501.40, with an average of \$88.57. Maximum and minimum values for distance traveled, on the other hand, are computed as 149.04 miles and 1 mile, respectively. For both fee and distance, data are closer to the mean. This suggests that the data are not spread or clustered around the mean. Figure 2.2.2 shows the relation between permit duration and miles traveled. OS/OW vehicles with a permit of approximately ten days appear to travel the most.

Table 2.1: Statistics on Permit Fees

	Max	Min	Avg.	Std.
Fee ($)	3501.40	1	88.57	138.57
Distance (mi)	918	1	149.04	114.69

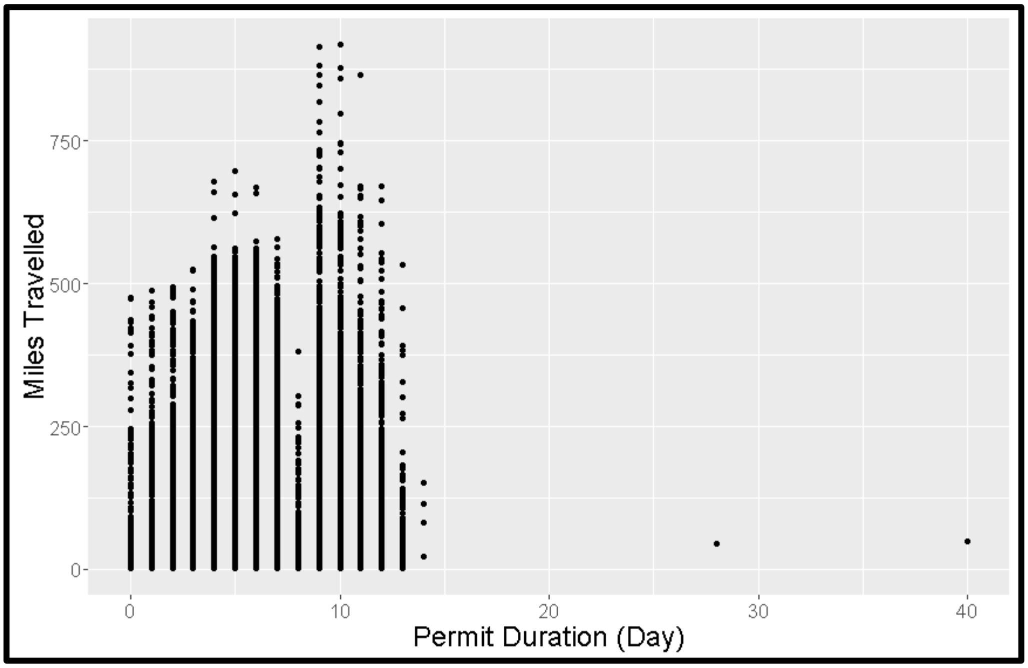


Figure 2.2: Permit duration versus miles traveled.

2.2.3 WIM Data

All the proposed frameworks in this report require information regarding vehicle weight and counts within a highway network. This need was met by weigh-in-motion (WIM) data. Although there are 22 WIM stations in Illinois, data were available from only 13 of the WIM stations. In Figure 2.3, the green crosses show the location of WIM stations whose data were available for this study, while the red dots indicate WIM stations that were not used due to unavailability of date.

The list of the variables provided in the WIM data is as follows:

> year, month, day, hour, minute, second, error, number, status, code, record, type, lane, speed, class, length, GVW, ESAL, W1, S1, W2, S2, W3, S3, W4, S4, W5, S5, W6, S6, W7, S7, W8, S8, W9, S9, W10, S10, W11, S11, W12, S12, W13, S13, W14, temperature

In the variable list, W and S stand for weight and axle spacing, respectively. Note that heavy vehicles in the WIM database are classified according to FHWA standards.

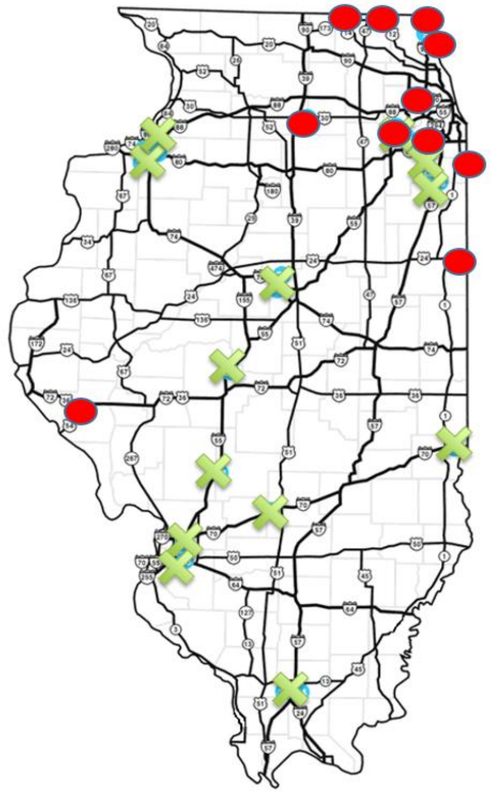


Figure 2.3: WIM stations in Illinois.

Figure 2.4 and Figure 2.5 illustrate the change in monthly overweight (OW) vehicle percentages with respect to time for the Moline EB and Bolingbrook NB stations. Maximum, minimum, and average OW vehicle percentages are also indicated on the plots.

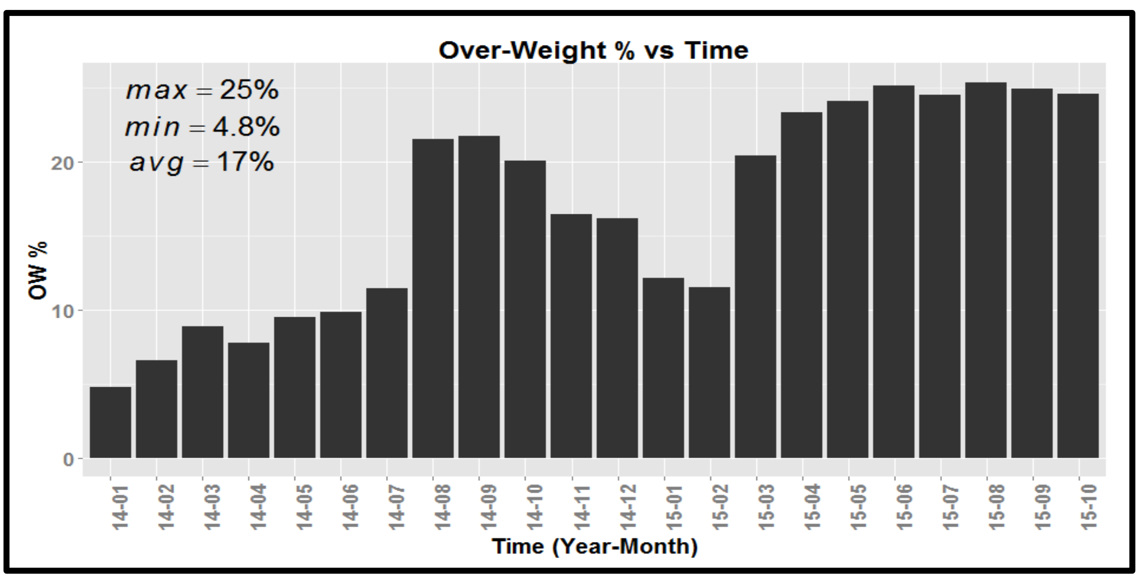


Figure 2.4: Overweight vehicle percentages for the Moline EB WIM station.

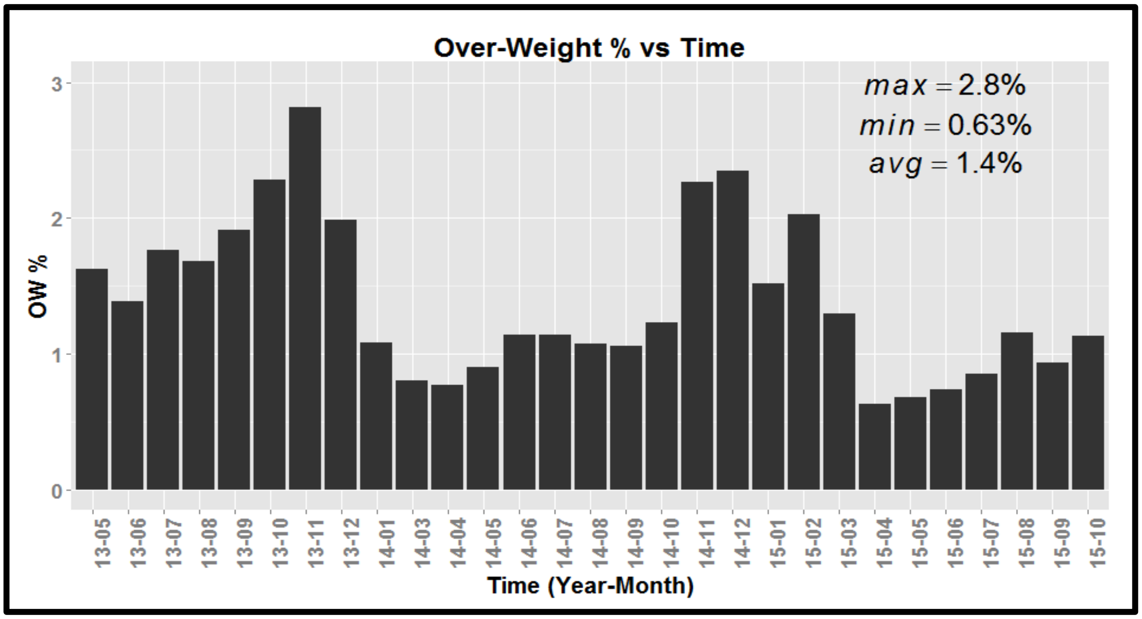


Figure 2.5: Overweight vehicle percentages for the Bolingbrook NB WIM station.

Table 2.2 provides the maximum, minimum, and average OW vehicle percentages for all WIM stations, along with their IDOT district numbers.

Table 2.2: OW Vehicle Percentages for WIM Stations

Name	District	Max OW%	Min OW%	Avg OW%	Start Date	End Date
Bolingbrook (Northbound)	1	2.8	0.63	1.4	05/13	10/15
Bolingbrook (Southbound)	1	0.79	0	0.33	05/13	10/15
Brownstown (Eastbound)	7	22	1.2	5.6	05/13	10/15
Carlock (Eastbound)	3	6.9	0.095	1.8	03/13	10/15
Carlock (Westbound)	3	22	4	16	03/13	10/15
East Moline (Eastbound)	2	5.9	0.95	3.1	03/13	10/15
Frankfort (Eastbound)	1	14	4.2	7	05/13	10/15
Litchfield (Northbound)	6	0.55	0.1	0.3	05/13	10/15
Marshall (Westbound)	5	14	0.0065	4.2	05/13	10/15
Maryville (Westbound)	8	7.2	0.15	1.9	04/13	10/15
Moline (Eastbound)	2	25	4.8	17	01/14	10/15
Moline (Westbound)	2	4.7	0.06	0.91	08/13	10/15
Peotone (Northbound)	1	6.9	0.51	2.3	11/13	10/15
Peotone (Southbound)	1	8.9	0.22	2.5	05/13	10/15
Williamsville (Southbound)	6	29	0.45	13	05/13	10/15

2.3 INFRASTRUCTURE DATA

2.3.1 Roadway Inventory System

IDOT provided the roadway dataset with the traffic data as described in Section 2.2.1. The relevant fields on the roadway characteristics are as follows:

- Inventory Number
- Beginning Station
- Ending Station
- Route Type Description
- Number of Lanes
- Median Type
- Urban Code

The inventory number is a string of numbers that indicates the key route designation assigned to a highway. This information is used by IDOT to uniquely identify each highway. The beginning and ending stations identify the highway segment. Thus, the length of the segment can be easily calculated as the difference between these two numbers.

The dataset provided by IDOT also contained roadway classification. For safety modeling, the roadway segments must be divided into various peer groups based on their design features, and similar segments must be categorized into a single group. By doing this, some categorical variables, such as area type, functionality classification, and median type, can be included in the model without requiring an extra set of dummy variables. In total, there are 12 different peer groups in the dataset provided by IDOT, but to make the data sample size larger for each roadway peer group (to ensure statistical significance for safety modeling of each peer group), it was decided to aggregate those groups into larger peer groups. The following four segment peer groups only are considered hereafter:

- Rural Undivided Highway
- Rural Divided Highway
- Urban Undivided Highway
- Urban Divided Highway

The methodology of dividing roadway characteristics into the above-mentioned peer groups is as follows:

> Area Type—The Urban field of the roadway dataset dictates whether a segment or intersection is in an urban or rural area.
>
> Undivided Highway—The Median Type field dictates what type of median is used. If no median is present, then the segment is classified as being an undivided highway.

Divided Highway—The Median Type field dictates what type of median is used. If any type of median is present, then the segment is classified as being a divided highway.

2.3.2 Pavement Condition Data

Pavement condition data was extracted from the Illinois Roadway Information System (IRIS) database to estimate pavement service life after rehabilitation treatments. In the IRIS database, the Condition Rating Survey (CRS) values for interstate highways and non-interstate highways were determined every 2 years from 2000 to 2014.

The extracted pavement condition data include Condition Rating Survey (CRS) values, International Roughness Index (IRI), rutting, and faulting. CRS indicates the wearing surface condition of a highway and its scale is 1.0 to 9.0. IDOT considers the pavement condition poor when the CRS value is less than or equal to 4.5. The pavement distress code is a set of letters and numbers indicating the pavement distress types and range. Distress codes A to K are concrete surface distresses, and distress codes L to X are asphalt surface distresses.

Physical location of key route, pavement surface type, AADT, average annual daily truck traffic (AADTT) of multiple-unit trucks and heavy commercial trucks (including multiple-unit and single-unit trucks), and pavement condition from the CRS indicate the data used to group segments into one section. Table 2.3 shows the dataset example.

Table 2.3: Dataset Example

Section No.	Inventory No.	Begin Station	End Station	Surface Type	AADT	Multiple Unit Count	Heavy Commercial Count	CRS	Pavement Distress
1	002 10057	0	1.48	740	9800	4300	4700	6.8	E1J1
2	002 10057	1.48	3.71	740	10,300	4350	4750	6.8	E1J2
3	002 10057	3.71	4.29	640	10,300	4350	4750	6.8	S2P2X2
4	010 10072	0	2.96	640	13,000	2600	3000	6.9	S2P3Q2
5	010 10072	2.96	8.03	640	13,300	2600	3000	6.9	S2P3Q2
6	010 10072	8.03	8.39	630	13,300	2600	3000	6.9	S2P3Q2
:	:	:	:	:	:	:	:	:	:

2.3.3 National Bridge Inventory

The National Bridge Inventory (NBI) is an online database that is published and maintained by the Federal Highway Administration (FHWA). The database contains data on more than 475,000 bridges (excluding culverts) nationwide, categorized by state and by year built, from 1992 through 2015. For each bridge, 94 metric data are collected and presented in the following groupings:

- Identification data (facility carried, location, latitude, and longitude)
- Structure type and material type (structure and material of main and approach structures, wearing surface, and deck protective system)

- Age and service (year built, number of lanes, average daily traffic)
- Geometric (length and width of bridge, horizontal and vertical clearances)
- Navigational data (pier protection and navigational clearances)
- Classification data (National Highway System designation, toll road, parallel structure)
- Condition data (deck, superstructure, and substructure)
- Load rating and posting data
- Appraisal items (calculated geometric data evaluations for functional obsolescence, safety features, scour criticality)
- Inspection data (routine inspection date and interval)

Bridges are evaluated on three components: deck, substructure, and superstructure. Bridge components are rated on a scale ranging from 0 to 9 (Table 2.4).

Table 2.4: Description of Condition Rating of Bridge Elements

Condition	Description
N	Not Applicable
9	Excellent
8	Very Good—no problem noted
7	Good
6	Satisfactory
5	Fair
4	Poor
3	Serious
2	Critical
1	"Imminent" Failure
0	Failed

Figure 2.6 on the following page demonstrates the rating distribution in Illinois from the year 2015. As seen, the most common rating is 8, which shows that many of the bridges in Illinois are in very good condition. In other words, they don't exhibit any major distresses. Another important criterion is having a high number of "not applicable (N)" and "blank (B)" data points, which highlights the importance of the data filtering and cleaning (Chapter 4).

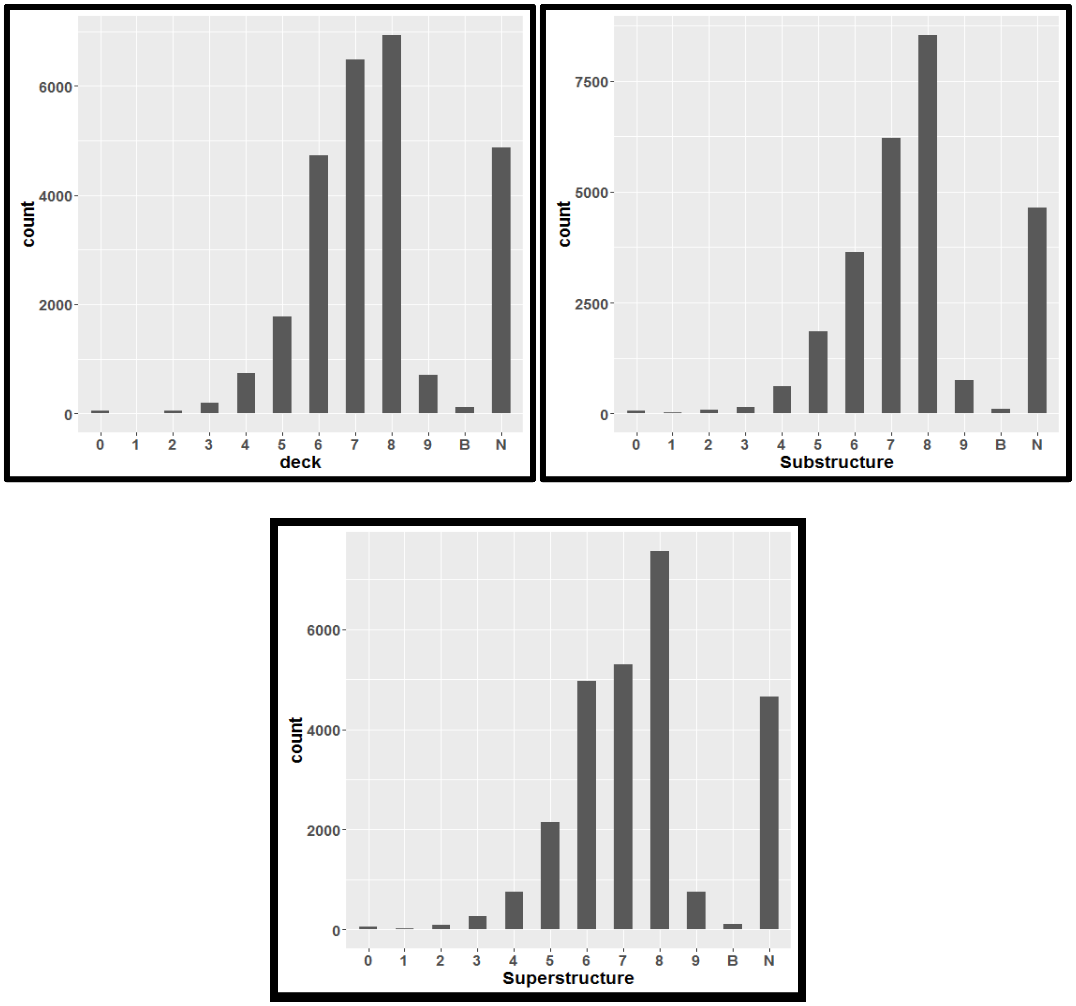


Figure 2.6: Rating distribution for three bridge components from the year 2015.

2.4 CRASH DATA

Similar to the traffic and roadway data, IDOT and CH2M Hill researchers provided the 2009–2013 crash counts by severity for each roadway segment. Crash severities were divided into three categories:

- K-crash: fatal injury caused by the crash
- A-crash: incapacitating injury caused by the crash
- B-crash: non-incapacitating injury caused by the crash

In the case of multiple vehicles or multiple injuries involved in one crash, the most severe injury type is used to describe crash severity (IDOT 2006). Severity type is the only information related to the crash that is included in the database. The database does not include details about the crash, such as the types of vehicles involved in the crash or reasons for the crash.

A simplified example of the dataset provided by IDOT and CH2M Hill is shown in Table 2.5. Only the most relevant information is shown. For each roadway segment, the information includes the number of crashes of each severity that occurred from 2009 to 2013, along with the corresponding peer group (see Section 2.2.1) and the traffic volume. A comprehensive methodology to aggregate those types of data is presented elsewhere (Tegge et al. 2010).

Table 2.5: Sample Dataset

Segment No.	Inventory No.	Beginning Station	Ending Station	AADT	Peer Group	K Crashes	A Crashes	B Crashes
1	014 20805	8.78	9.12	4800	1	0	1	1
2	014 20805	4	4.12	5100	1	0	0	1
3	030 20331	6.62	6.86	5500	1	0	0	1
:	:	:	:	:	:	:	:	:

Table 2.6 displays some statistics related to the counts of different crash severity types per roadway segment in the period of 2009–2013. The B-crashes are more frequent that the A- and K-crashes. Note that the average crash count for a roadway segment for the different severity types is quite low: a large number of segments did not experience any crashes between 2009 and 2013. The largest number of B-crashes occurring on one roadway segment between 2009 and 2013 is 60 crashes, while it was 14 and 4 for A-crashes and K-crashes, respectively.

Table 2.6: Summary of Crash Counts for a Roadway Segment from the Dataset

Crash severity	Average	Standard deviation	Maximum	Minimum
Fatal (K)	0.014	0.127	4	0
Incapacitating injury (A)	0.104	0.454	14	0
Non-incapacitating injury (B)	0.265	1.162	60	0

CHAPTER 3: PAVEMENT DETERIORATION MODEL AND COST ESTIMATION

This chapter contains development of pavement performance deterioration models and life-cycle cost analysis of pavement damage cost.

Pavement performance data were extracted from the Illinois Roadway Information System (IRIS) database to predict pavement service life after rehabilitation. The average annual daily number of single-unit (SU) trucks and multiple-unit (MU) trucks were collected from the IRIS database. The annual number of equivalent single-axle loads (ESALs) was calculated using load-equivalency factors (LEFs). The traffic data from IRIS were compared to the data obtained from weigh-in-motion (WIM) sites located along interstate highways. Pavement deterioration models were developed for hot-mix asphalt (HMA) pavement and Portland cement concrete (PCC) pavement, respectively, considering the effects of traffic loading, existing pavement condition before rehabilitation, and climate condition.

In accordance with the maintenance strategy recommended in Chapter 54 (Pavement Design) of the Illinois Department of Transportation's Bureau of Design and Environment (BDE) Manual (along with maintenance, rehabilitation, and reconstruction [MR&R] costs obtained from IDOT), life-cycle cost analyses (LCCA) were performed to estimate pavement damage costs. After pavement damage cost is determined, a permit fee for specific overweight axles and trucks can be determined based on the axle-load configuration and the excessive load magnitude. Figure 3.1 illustrates the framework for pavement damage cost analysis.

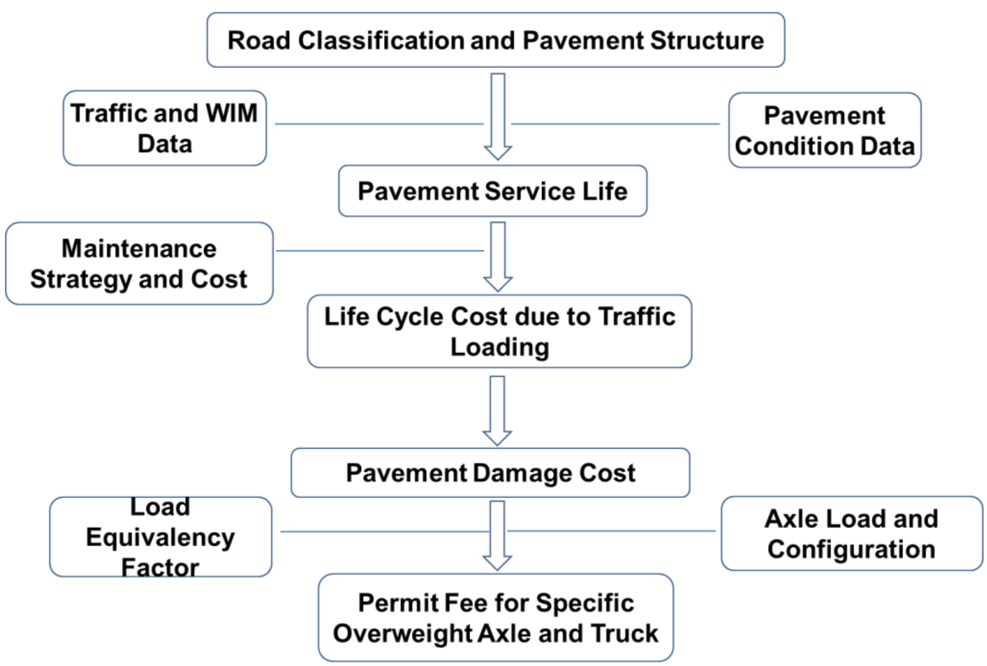


Figure 3.1: Flowchart of pavement damage cost analysis.

3.1 DATA PROCESSING AND ANALYSIS

The records in the IRIS database provided by IDOT contain physical location, traffic, and pavement performance for interstate highways and non-interstate highways. Another traffic data source, the WIM data, was obtained and compared with the traffic data from the IRIS database. In terms of climate factors, the freezing index (FI) was calculated based on the air temperature collected by the State Climatologist Office for Illinois.

3.1.1 Pavement Performance Data

Pavement performance data were extracted from the IRIS database to estimate pavement service life after rehabilitation treatments. For the IRIS database, CRS values for interstate highways and non-interstate highways were determined every 2 years from 2000 to 2014. Although the pavement performance data included Condition Rating Survey (CRS), International Roughness Index (IRI), rutting, and faulting, only CRS data were used in the analysis to determine pavement service life. CRS indicates the wearing surface condition of the highway, and its values range from 1.0 to 9.0. The service life in the study was determined as the time period before the CRS reached 4.5, below which the pavement condition is considered poor.

Four steps were followed to process the pavement condition data.

Step 1: Extract data from the IRIS database. Physical location of key route, pavement surface type, traffic volume of multiple-unit trucks and heavy commercial trucks (including multiple-unit and single-unit trucks), and pavement performance data were recorded for road segments with different starting and ending mileposts. Segments were grouped into a longer segment if they had the same route and surface type, as well as similar traffic.

Step 2: Remove data errors and inconsistencies. For each of the following cases, the data records were excluded from the analysis.

- Concrete surface type with asphalt distresses recorded
- Asphalt surface type with concrete distresses recorded
- CRS equal to zero

Step 3: Organize data in different years. Because the mileposts of road segments varied from 2000 to 2014, the segments were regrouped based on similar CRS and traffic and having the same route and surface type.

Step 4: Delete irrational performance data. The following cases were deleted from the organized dataset.

- The surface type was inconsistent with that in the prior or subsequent year
- CRS increases to 9 when the pavement condition in the previous year was good

After the data extraction, checking/cleansing, and organization, the CRS data and the corresponding traffic at different road segments were used for pavement performance model development.

3.1.2 Traffic Data

The two-way annual average daily truck traffic (AADTT) data were extracted from the IRIS database for single-unit trucks and multiple-unit trucks. The AADTT showed a skewed distribution, with only a few segments having a relatively large traffic volume.

There are 102 pavement segments for interstate highways, categorized by county. For a majority (90%) of pavement segments, the AADTT for single-unit trucks is less than 5000, with a median value of 1214, while the AADTT for multiple-unit trucks is less than 20,000, with a median value of 5670.

There are 2368 road segments for non-interstate highways, categorized by county. For a majority (90%) of pavement segments, the AADTT for single-unit trucks is less than 925, with a median value of 180; the AADTT for multiple-unit trucks is less than 778, with a median value of 105. (As expected, truck traffic on non-interstate highways is much less than truck traffic on interstate highways.)

To consider the equivalent loading effect of different trucks, the number of equivalent single-axle loads (ESAL) of total truck traffic was calculated using Equation 3.1. Table 3.1 shows the load-equivalency factors (LEFs) for flexible and rigid pavements as recommended in Chapter 54 (Pavement Design) of the IDOT BDE Manual.

$$T = S \times SU \times L_S + M \times MU \times L_M \tag{3.1}$$

where

T = average annual daily ESALs

L_S, L_M = lane distribution factor

S, M = 18-kip ESAL application per single-unit vehicle and multiple-unit vehicle (load-equivalency factors)

SU, MU = structural design traffic expressed as the number of single-unit vehicles and multiple-unit vehicles

Table 3.1: Load-Equivalency Factors for 18-kip ESAL Applications per Vehicle (from IDOT Pavement Design Manual)

Facility Class	Flexible Pavements			Rigid Pavements[1]		
	PV[2]	SU	MU	PV	SU	MU
Class I	0.0004	0.363	1.322	0.0004	0.394	1.908
Class II	0.0004	0.307	1.056	0.0004	0.372	1.554
Class III	0.0004	0.299	1.053	0.0004	0.355	1.541

[1] The equivalency factors for rigid pavements are used for HMA/PCC pavements.

[2] Passenger vehicle.

WIM devices can continuously capture and record axle load, gross vehicle weight (GVW), and axle spacing with supplementary data (date, time, speed, lane of travel, vehicle type, etc.) over a measurement site. Although the WIM data can provide more detailed information on axle configurations and loading magnitudes compared to traffic data in the IRIS database, only around 22

WIM stations are installed in Illinois, and the majority of those are on interstate highways. In this study, the WIM data were used to verify the accuracy of traffic data from the IRIS database. Figure 3.2 compares the traffic data (in average annual daily ESALs) collected at the WIM stations on interstate highways with the ones calculated based on traffic data in IRIS. Although some discrepancies were observed, the comparison suggests general consistency between IRIS and WIM traffic data.

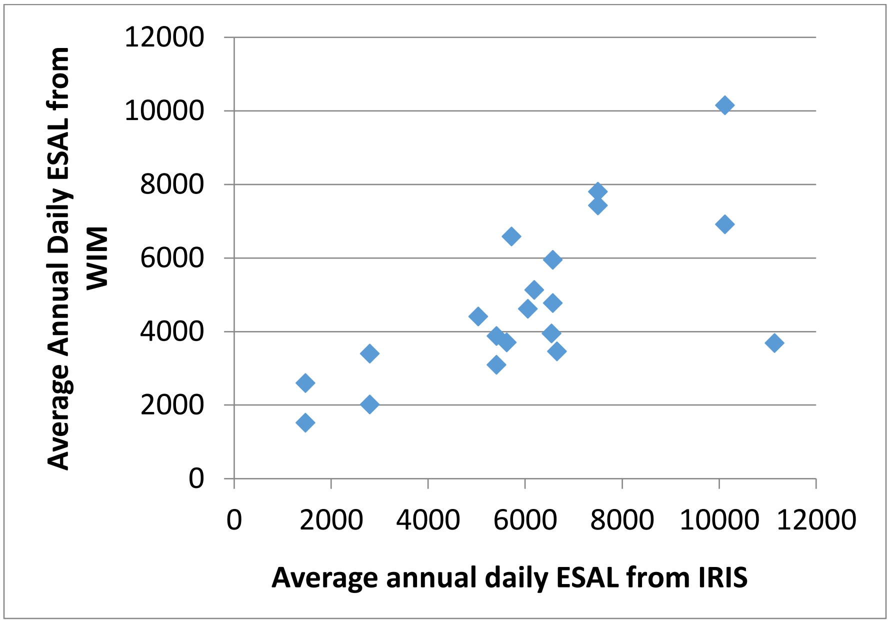


Figure 3.2: Comparison of ESALs obtained from IRIS and WIM in 2014.

3.1.3 Freezing Index

The climate effect on pavement performance is considered through the freezing index (FI). The FI is defined as the absolute value of the sum of all average daily temperatures below 32°F within the freezing season. Air temperature data from 2000 to 2014 were obtained from the State Climatologist Office. The annual FI is computed with the method used in the Long-Term Pavement Performance (LTPP) database, as shown in Equation 3.2.

$$FI = \sum_{i=1}^{n}(32 - T_i) \tag{3.2}$$

where

FI = freezing index, degrees Fahrenheit (°F) degree-days

T_i = average daily air temperature on day i, °F

n = days in one year when average daily temperature is below freezing

i = number of days below freezing

Table 3.2 presents the calculated average FI (°F) from 2000 to 2014 at the 20 stations and their corresponding counties and districts. The FI for each district was then calculated through grouping the stations in the same district and averaging their FI. The average freezing index for each district was calculated, and the median value of freezing index for the nine districts was found being 630 degree-days, which was used in the further analysis of pavement service life.

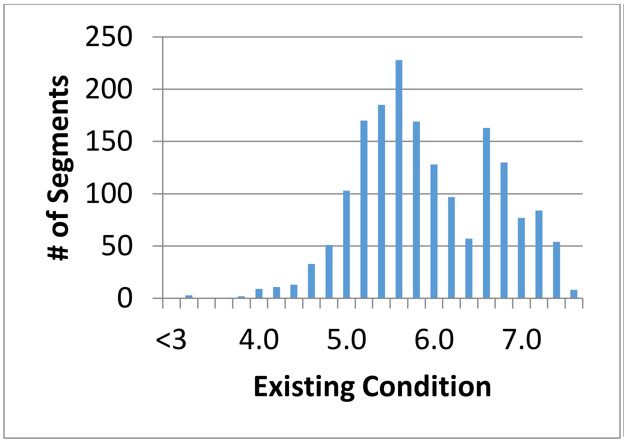
Table 3.2: Estimated Freezing Index from 2000 to 2014

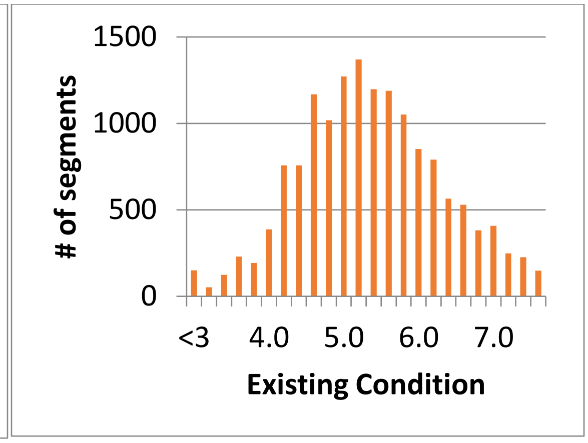
Station	County	District	Average Freezing Index (°F)
Belleville	St. Clair	8	349
Big Bend	Cook	1	958
Bondville	Champaign	5	662
Brownstown	Fayette	7	441
Carbondale	Jackson	9	281
Champaign	Champaign	5	598
DeKalb	DeKalb	3	990
Dixon Springs	Pope	7	266
Fairfield	Wayne	7	335
Freeport	Stephenson	2	1019
Kilbourne	Mason	6	636
Monmouth	Warren	4	797
Olney	Richland	7	384
Peoria	Peoria	4	684
Perry	Pike	6	553
Rend Lake	Franklin/Jefferson	9	308
Springfield	Sangamon	6	504
St. Charles	Kane	1	912
Stelle	Ford	3	831
Wildlife Park	Peoria	4	647

3.1.4 Existing Pavement Condition Before Rehabilitation

It is expected that the existing pavement condition before rehabilitation may affect pavement overlay performance. Figure 3.3 shows the frequency distribution of existing pavement condition (CRS ratings) no more than 2 years before pavement rehabilitation for interstate and non-interstate highways. (Rehabilitation treatments include asphalt overlay on PCC or HMA surface pavements and major patching of PCC pavements.)

The data analysis results show that the CRS ratings before pavement rehabilitation have a wide range of variation. For interstate highways, the median values of CRS ratings on HMA surface pavements and PCC pavements before rehabilitation are 5.6 and 6.0, respectively. For non-interstate highways, the median values of CRS ratings on HMA surface pavements and PCC pavements before rehabilitation are 5.2 and 5.6, respectively.





(a) **(b)**

Figure 3.3: Frequency distributions of existing pavement conditions before rehabilitation for (a) interstate and (b) non-interstate highways.

3.1.4 Vehicle Miles Traveled (VMT)

Because the mileage traveled by an overweight truck will be distributed between interstate and non-interstate highways, the representative percentage of miles traveled on each highway should be known. The permit database from 2013 to 2016 was obtained from IDOT. Because of the large amount of data, only a part of the original permit database was analyzed. The purpose of the analysis was to calculate the percentage of vehicle miles traveled (VMT) on interstate highways and non-interstate highways, which are used to estimate permit fees without knowing the specific route information.

A total of 151,455 permits issued in the year 2013 were extracted from the permit fee database. In the database, the routes traveled by the trucks were recorded and were analyzed by grouping the VMT on interstate highways and non-interstate highways. The percentage of VMT on interstate highways was calculated by dividing VMT on interstate highways by the total mileage traveled. Figure 3.4 shows the cumulative distribution of the percentage of VMT on interstate highways with respect to the total VMT. The mean of the percentage of VMT on interstate highways was 66.26%, and the median was 78.59%.

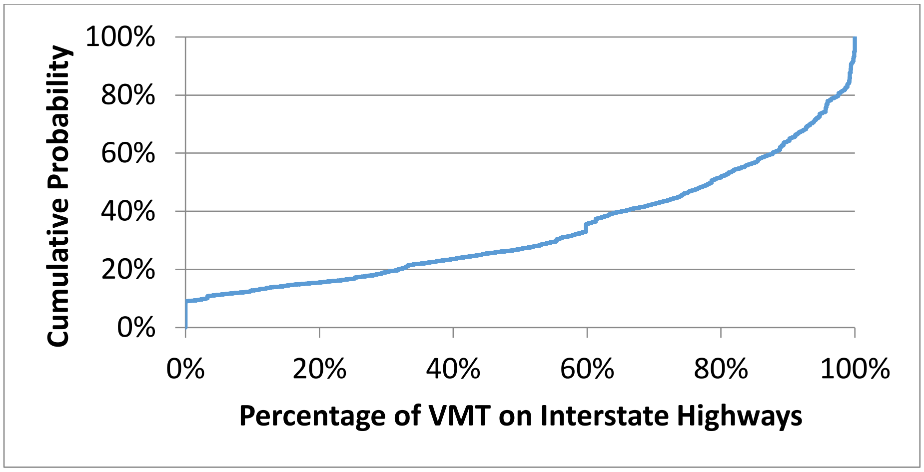
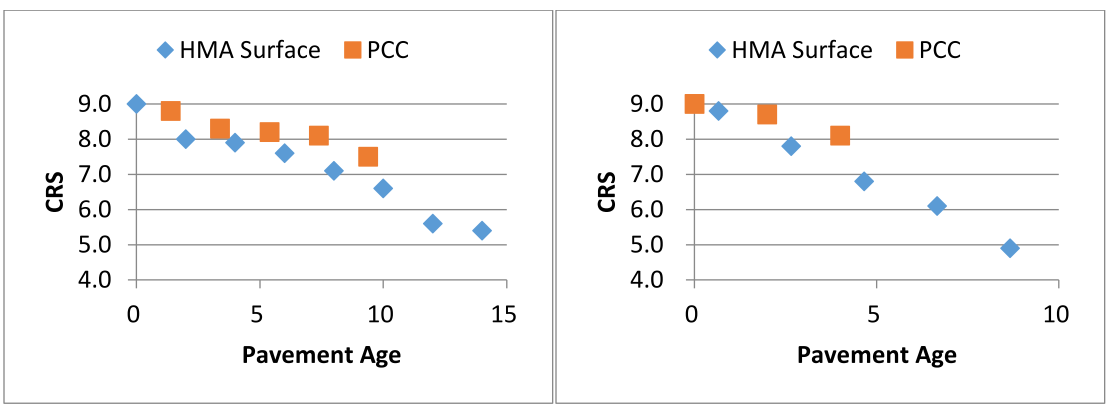


Figure 3.4: Cumulative distribution of the percentage of VMT on interstate highways.

3.2 PAVEMENT PERFORMANCE MODEL DEVELOPMENT

The pavement performance model is the model that describes the deterioration trend of pavement condition over the years. The typical change trend of CRS on interstate highways is shown in Figure 3.5(a). It was found that HMA surface pavements deteriorated faster than PCC surface pavements on interstate highways. Figure 3.5(b) shows the typical change trend of CRS on non-interstate highways. The pavement deteriorates more quickly on non-interstate highways than on interstate highways. In this study, pavement performance models were developed for HMA surface pavements (including full-depth HMA and HMA/PCC pavement) and PCC pavements (including JPCP and CRCP), respectively, on interstate and non-interstate highways.



(a) **(b)**

Figure 3.5: CRS trends over time on (a) interstate and (b) non-interstate highways.

A number of previous research studies have been conducted to develop pavement performance models to predict new pavement or overlay service life by associating pavement condition indicators (International Roughness Index (IRI), rutting, pavement condition rating (PCR), alligator cracking, et al.) with contributing factors. The cumulative 18-kips ESALs was a common traffic loading indicator used in the past works. The existing pavement condition before rehabilitation (Madanat et al. 2005;

Abaza 2005; Madanat et al. 2005; Zhou et al. 2009; Wang 2013; Wang and Nie 2014), pavement age (George 2000; Li and Sinha 2000; Ong 2010; Wang 2013), and pavement or overlay thickness or structural number (George 2000; Prozzi and Madanat 2002; Abaza 2005; Madanat et al. 2005; Zhou et al. 2009; Wang 2013) were considered critical factors affecting pavement failure. Climate factors, such as temperature and precipitation (Martin 2002; Madanat et al. 2005; Wang 2013), frost gradient (Prozzi and Madanat 2002), freezing index (Ong 2010), freeze-thaw cycles (Madanat et al. 2005; Wang 2013) were also considered in pavement deterioration models.

Several regression models were developed to predict pavement life, as shown in Equations 3.3 through 3.6. The models differ in terms of the influence variables considered and the model format. The first model was developed by relating CRS to average annual daily ESALs, pavement age, and FI. The second model has a format similar to the first model but uses the existing pavement condition as an influencing variable, which makes the performance model more reliable. In the third model, the incremental change of CRS is related to CRS in the prior year, daily ESALs, age difference, FI, and existing pavement condition before rehabilitation. The CRS is determined by subtracting the change of CRS from the initial CRS.

$$\text{Model 1: } CRS = a - c \times Log(T) \times t - d \times Log(FI) \times t \tag{3.3}$$

$$\text{Model 2: } CRS = a - c \times Log(T) \times t - d \times Log(FI) \times t + e \times Log(CRS\_E) \times t \tag{3.4}$$

$$\text{Model 3: } CRS = CRS_0 - \sum\nolimits_0^t \Delta CRS \tag{3.5}$$

$$\text{with } \Delta CRS = c \times Log(T) \times I + d \times Log(FI) \times I - e \times Log(CRS\_E) \times I \tag{3.6}$$

where

- t = pavement age (years) since previous rehabilitation
- T = daily ESALs
- FI = freezing index
- ΔCRS = change of CRS; $\Delta CRS = c \times Log(T) \times I + d \times Log(FI) \times I - e \times Log(CRS\_E) \times I$
- CRS_0= initial CRS
- I = age difference (years)
- CRS_E = existing CRS before rehabilitation
- a, c, d, e = regression coefficients

Regression analyses were conducted for the three models, and the results are shown in Tables 3.3, 3.4, and 3.5. The results show that, in general, the R^2 values are similar for the three models; however, model 3 shows better fitting. It is noted that for some road segments, the available pavement condition data may not cover the year right after rehabilitation. In other words, for the initial CRS rating lower than 9, the rehabilitation could have been applied before 2000. When model 1 or 2 is used, the pavement age at which the first CRS data point was recorded was estimated using the districtwide pavement performance model reported in prior research (Heckel and Ouyang 2007). This estimation may affect the model accuracy. However, no estimation of pavement age is needed for model 3. Thus, the pavement life predicted by the incremental pavement damage model (Equations 3.5 and 3.6) was used in the further analysis.

Table 3.3: Regression Analysis Results for Model 1

Route	Surface Type	# of Points	a	c	d	R^2
Interstate	HMA	9051	8.8266	0.0157	0.0885	84
Highway	PCC	2239	8.9223	0.0050	0.0498	80
Non-Interstate	HMA	129,237	8.963	0.001	0.126	85
Highway	PCC	13,875	8.412	0.001	0.041	73

Table 3.4: Regression Analysis Results for Model 2

Route	Surface Type	# of Points	a	c	d	e	R^2
Interstate	HMA	5064	8.86	0.031	0.073	0.001	84
Highway	PCC	997	8.80	0.0071	0.0518	0.0401	77
Non-Interstate	HMA	44,287	8.640	0.005	0.106	0.001	76
Highway	PCC	4,073	8.338	0.001	0.041	0.002	65

Table 3.5: Regression Analysis Results for Model 3

Route	Surface Type	# of Points	c	d	e	R^2
Interstate	HMA	4587	0.055	0.133	0.258	86
Highway	PCC	828	0.001	0.095	0.001	80
Non-Interstate	HMA	42,086	0.009	0.164	0.074	84
Highway	PCC	3,459	0.049	0.131	0.190	82

3.3 PAVEMENT LIFE-CYCLE COST ANALYSIS

Life-cycle cost analysis (LCCA) is a process for evaluating the overall long-term economic worth of a project segment by calculating initial costs and discounted future costs, such as maintenance, rehabilitation, and user costs. Agency costs and user costs are two major factors in LCCA. Agency costs are defined as the costs related to the owning organizations over the life of the project, such as initial construction costs and maintenance costs. User costs include travel time, vehicle operation, accidents, and inconvenience costs paid by the user of road. In this study, only agency costs were considered in the pavement LCCA (i.e., user costs were not included in the LCCA).

Analysis period and discount rate are the two most important parameters affecting LCCA. According to National Cooperative Highway Research Program (NCHRP) Guide for Pavement-Type Selection, an analysis period of at least 40 years was suggested for new construction or reconstruction of pavements, while an analysis period of at least 30 years was suggested for rehabilitation of pavements. A respectively longer analysis period should be selected for long-life pavements. Discount rate is used to convert future costs to present-year costs. Historically discount rates are in the range of 3% to 5%. The long-term real discount rate values supplied in the lately updated edition of the Office of Management and Budget (OMB) Circular A-94, Appendix C, was suggested to use in life-cycle cost analysis.

In accordance with Chapter 54 (Pavement Design) of the BDE Manual, pavement maintenance and rehabilitations strategies for a 45-year analysis period were established for flexible and rigid pavements. A discount rate of 3% is suggested to convert future costs to present-year costs.

The net present value (NPV) is defined as the sum of the present values of the individual cash flows of the same entity and has wide application in pavement life-cycle cost analysis. The NPV of agency costs during the analysis period is computed using the discounted monetary value of future costs and salvages by transforming costs occurring in different time periods and salvages at the end of the analysis period to a common unit of measurement. The NPV of pavement life-cycle cost can be calculated using Equations 3.7 and 3.8.

$$NPV = C + M_i\left(\frac{1}{1+r}\right)^{n_i} + \ldots + M_j\left(\frac{1}{1+r}\right)^{n_j} - S\left(\frac{1}{1+r}\right)^{N} \quad (3.7)$$

$$S = \left(1 - \frac{L_A}{L_E}\right)C \quad (3.8)$$

where

NPV = net present value or present worth

C = present cost of initial rehabilitation activity

M_i, M_j = cost of the ith and jth maintenance and rehabilitation (M & R) alternative in terms of constant dollars

r = discount rate

n_i, n_j = number of years from the present to the ith and jth M & R activity

N = length of the analysis period in years

S = salvage value at the end of the analysis period

L_A = analysis life of the rehabilitation alternative in years

L_E = expected life of the rehabilitation alternative

C = cost of the rehabilitation alternative

The maintenance and rehabilitation activities during 45 years of service were considered based on the activities recommended in Chapter 54 (Pavement Design) of the BDE Manual. For full-depth HMA pavement, milling and asphalt overlay is applied in the 15th and 30th years after initial construction, and routine maintenance is applied every 5 years after initial construction and rehabilitation. For JPCP and CRCP, patching is applied every 5 years after the first one implemented in the 10th year, and then asphalt overlay is applied in the 30th year after initial construction. It is noted that for PCC pavement, the pavement type is changed to HMA/PCC pavement after asphalt overlay is applied.

Typically, an asphalt overlay is applied when the pavement reaches its service life, which is defined as the age when its CRS becomes equal to or less than 4.5. Because pavement service life depends on many factors—such as traffic loading, climate condition, existing pavement condition and routine maintenance activities—the age at which the asphalt is applied may vary across different pavement types and sections. While it was assumed that routine maintenance activities do not affect pavement performance for full-depth HMA pavements, major patching was assumed to increase the CRS rating and thus extend pavement service life for PCC pavements. Based on the data in IRIS, major patching was applied to extend PCC pavement service life by improving the CRS rating from 6.0 to 8.3 for interstate highways and from 5.8 to 7.6 for non-interstate highways, respectively.

Figures 3.6 and 3.7 illustrate the life-cycle activity flow for a full-depth HMA pavement with a service life of 12 years before overlay, and a JPCP pavement with a service life of 25 years before asphalt overlay (HMA/PCC pavement), respectively.

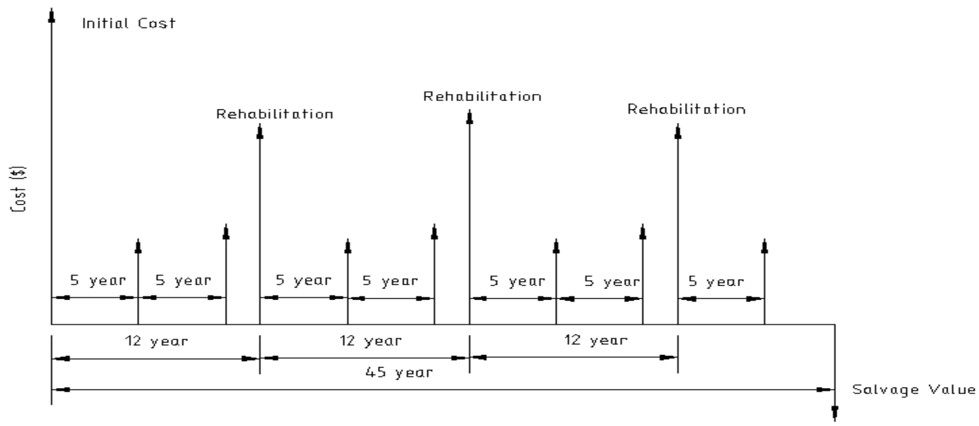


Figure 3.6: Activity flow for full-depth HMA pavement with pavement service life of 12 years before overlay.

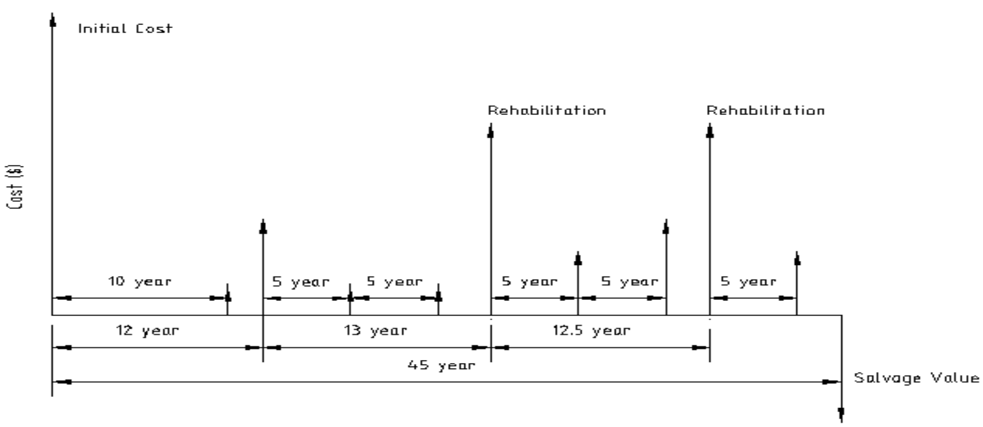


Figure 3.7: Activity flow for JPCP pavement with pavement service life of 25 years before asphalt overlay (HMA/PCC pavement).

Tables 3.6 and 3.7 show the unit cost of initial construction and maintenance and rehabilitation treatments for full-depth HMA pavements and PCC pavements, respectively. The cost data were extracted from the approved life-cycle cost analyses on IDOT's website or provided by IDOT, and were organized on the basis of surface mix, overlay thickness, and treatment type. It should be noted that the unit costs of treatments may decrease as the PCC slab thickness increases, which might be caused by the variance of bid prices, the size of project, and the project location.

Table 3.6: Unit Costs of Treatments for Full-Depth HMA Pavements (FY 2013 $)

	Treatment	Thickness (in)	Cost ($)	Unit
Initial Construction	HMA SURFACE COURSE	2.00	11.39	SQ YD
	HMA TOP BINDER COURSE	2.25	12.10	SQ YD
	HMA LOWER BINDER COURSE	6.5	37.60	SQ YD
	HMA SHOULDER	8	41.7	SQ YD
	IMPROVED SUBGRADE:		3	SQ YD
Maintenance & Rehabilitation	HMA OVERLAY PVMT SURF	2	11.4	SQ YD
	HMA OVERLAY PVMT	3.75/2.25[1]	20.6/12.49[2]	SQ YD
	HMA SURFACE MIX	1.5	8.5	SQ YD
	HMA BINDER MIX	2.25/0.75	12.1/3.96	SQ YD
	HMA OVERLAY SHLD (30 Year)	1.75/2.25	9.1/11.72	SQ YD
	HMA OVERLAY SHLD	2	10.4	SQ YD
	MILLING	2	3	SQ YD
	PARTIAL DEPTH PVMT PATCH (surface mix)	Mill & Fill Surf 2	150	SQ YD
	PARTIAL DEPTH SHLD PATCH (shoulder mix)	Mill & Fill Surf 2	150	SQ YD
	PARTIAL DEPTH PVMT PATCH (leveling binder mix)	Mill & Fill +2	150	SQ YD
	PARTIAL DEPTH SHLD PATCH (shoulder mix)	Mill & Fill +2	150	SQ YD
	LONGITUDINAL SHOULDER JOINT ROUT & SEAL		2	LIN FT
	CENTERLINE JOINT ROUT & SEAL		2	LIN FT
	RANDOM/ HERMAL CRACK ROUT & SEAL		2	LIN FT

[1] The thickness before “/” is for interstate highways, and the thickness after “/” is for non-interstate highways.
[2] The unit cost before “/” is for interstate highways, and the unit cost after “/” is for non-interstate highways.

Table 3.7: Unit Costs of Treatments for PCC Pavements (FY 2013 $)

	Treatment	Thickness (inch)	Cost ($)	Unit
Initial Construction	JPC PAVEMENT 8 inch	8	42.05	SQ YD
	JPC PAVEMENT 9.25 inch	9.25	45.1	SQ YD
	JPC PAVEMENT 9.5 inch	9.5	48.42	SQ YD
	CRC PAVEMENT 9.5 inch	9.5	59.71	SQ YD
	CRC PAVEMENT 9.75 inch	9.75	65.05	SQ YD
	CRC PAVEMENT 10 inch	10	58.41	SQ YD
	CRC PAVEMENT 10.75 inch	10.75	58.52	SQ YD
	STABILIZED SUBBASE (JPCP)	4	15	SQ YD
	STABILIZED SUBBASE (CRCP)	4	22.8	SQ YD
	PCC SHOULDERS	9	35	SQ YD
	SUBBASE GRAN MATL TY C	1.74	25	TON
	IMPROVED SUBGRADE:		3	SQ YD
Maintenance & Rehabilitation	HMA POLICY OVERLAY PVMT	3.75/2.50[1]	20.6/13.81[2]	SQ YD
	HMA SURFACE MIX	1.5	8.5	SQ YD
	HMA BINDER MIX	2.25/1.00	12.1/5.28	SQ YD
	HMA POLICY OVERLAY SHLD	3.75/2.50	19.5/13.02	SQ YD
	CLASS A PAVEMENT PATCHING		178	SQ YD
	CLASS B PAVEMENT PATCHING		225	SQ YD
	CLASS C SHOULDER PATCHING		150	SQ YD
	PARTIAL DEPTH PVMT PATCH (surface mix)	Mill & Fill HMA Surf 2	150	SQ YD
	PARTIAL DEPTH PVMT PATCH (surface mix)	Mill & Fill HMA 2.5	170	SQ YD
	LONGITUDINAL SHOULDER JOINT ROUT & SEAL		2	LIN FT
	CENTERLINE JOINT ROUT & SEAL		2	LIN FT
	REFLECTIVE TRANSVERSE CRACK ROUT & SEAL		2	LIN FT
	RANDOM CRACK ROUT & SEAL		2	LIN FT

[1] The thickness before "/" is for interstate highways, and the thickness after "/" is for non-interstate highways.
[2] The unit cost before "/" is for interstate highways, and the unit cost after "/" is for non-interstate highways.

3.4 PAVEMENT DAMAGE COST

For the same pavement structure, the initial construction cost is unchanged, so pavement life-cycle cost varies depending on pavement service life and maintenance strategy. The unit pavement damage cost (PDC) was obtained by dividing the NPV of life-cycle cost by the number of total ESALs in the analysis period. Using this methodology, pavement damage cost can be calculated as a function of traffic with the inputs of FI and the preferred CRS threshold for pavement rehabilitation.

Figures 3.8 and 3.9 illustrate the estimated pavement damage cost (PDC) for full-depth HMA pavement and HMA/PCC pavement, respectively. The freezing index of 349 degree-days (average freezing index of District 8) and the median values of CRS before rehabilitation were used. The results show that pavement damage cost varies significantly with traffic loading. In general, pavement damage cost decreases with the increase of average annual daily ESAL in a power function. The pavement damage cost for non-interstate highway was found to be much greater than the pavement damage cost for interstate highway. The higher cost of non-interstate highway is primarily related to low traffic volume, although there are similar initial construction and maintenance costs for non-interstate and interstate highways.

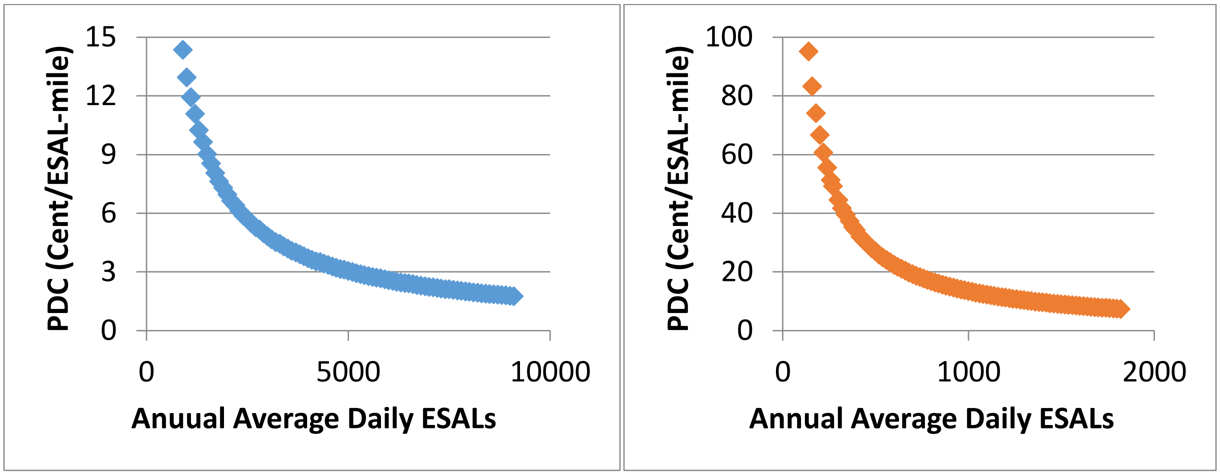


Figure 3.8: Pavement damage cost for full-depth HMA pavement on (a) interstate and (b) non-interstate highways.

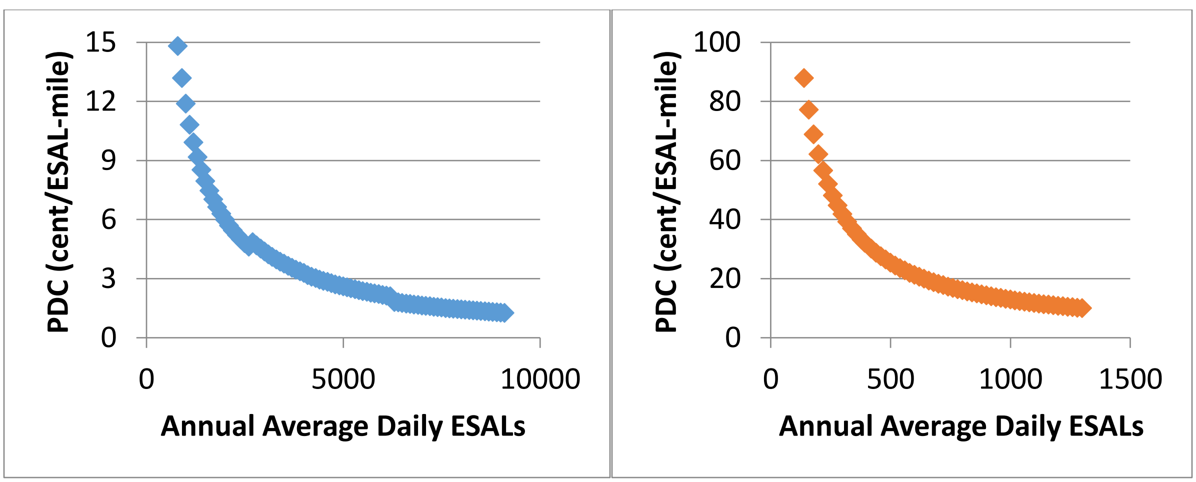


Figure 3.9: Pavement damage cost for HMA/PCC pavement on (a) interstate and (b) non-interstate highways.

Figure 3.10 presents the cumulative distribution of daily ESALs calculated using the equivalency factor for full-depth HMA pavements (flexible) and HMA/PCC pavements (rigid) on interstate and non-interstate highways. The 25th percentile, 50th percentile (median), 75th percentile, and mean of the daily ESALs on interstate highways were 877, 3085, 4769, and 3109, respectively, for full-depth HMA pavements. The 25th percentile, 50th percentile (median), 75th percentile, and mean of daily ESALs on interstate highways calculated were 3306, 5020, 7382, and 5676, respectively, for HMA/PCC pavements. For non-interstate highways, the 25th percentile, 50th percentile (median), 75th percentile and mean of daily ESALs are 35, 105, 248, and 217 for full-depth HMA pavements, respectively. The 25th percentile, 50th percentile (median), 75th percentile, and mean of daily ESALs were 109, 257, 572, and 607, respectively, for HMA/PCC pavements.

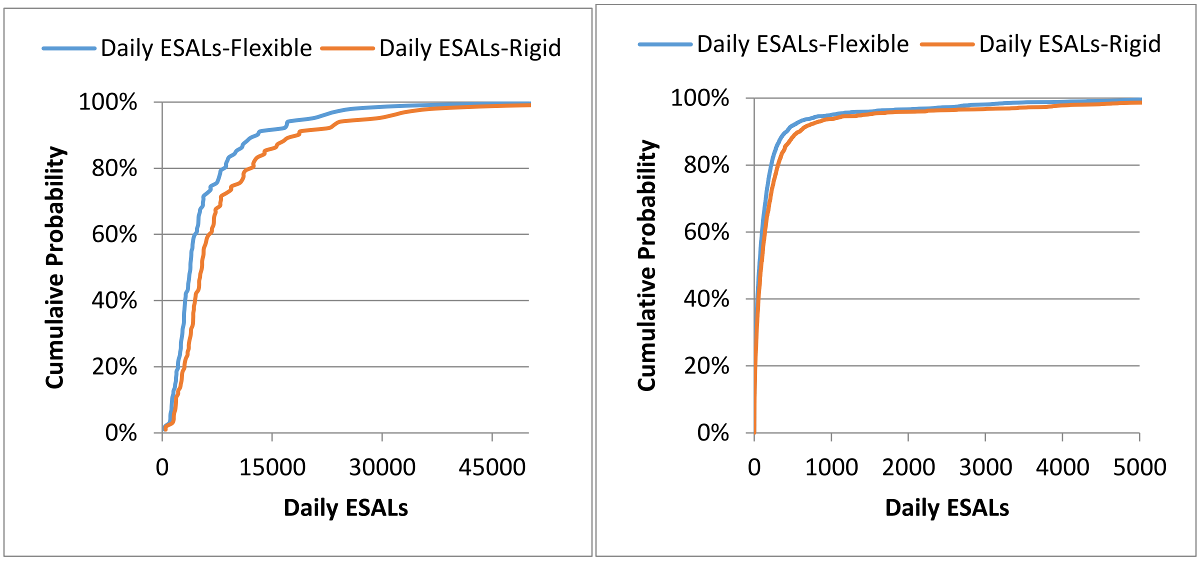


Figure 3.10: Cumulative probability distributions of daily ESALs for (a) interstate and (b) non-interstate highways.

Table 3.8 shows the PDC of different pavement types on interstate and non-interstate highways at different traffic loading levels that were considered the percentile of daily ESALs. These cost values serve as basis to calculate permit fee of overweight trucks.

Table 3.8: PDC at Different Levels of Traffic Loading

		Interstate Highway		Non-Interstate Highway	
Pavement Type	**Percentile of Daily ESAL**	**Daily ESALs**	**PDC (cent/ ESAL-mile)**	**Daily ESALs**	**PDC (cent/ ESAL-mile)**
Full-Depth HMA Pavement	25th	2536	1.96	17	827.23
	50th	3844	1.33	67	207.66
	75th	7189	0.77	181	77.10
	Mean	6292	0.87	256	54.57
HMA/PCC Pavement	25th	3615	3.66	22	606.57
	50th	5430	2.49	91	151.70
	75th	10,249	1.20	252	55.95
	Mean	8922	1.40	366	38.80

CHAPTER 4: BRIDGE DETERIORATION AND LIFE-CYCLE COST ANALYSIS

In the State of Illinois, the overweight limits for a group of two or more consecutive axles are calculated by the following formula (Equation 4.1):

$$W = 500\left(\frac{LN}{N-1} + 12N + 36\right) \tag{4.1}$$

where

W = overall gross weight of any group of two or more consecutive axles, to the nearest 500 lb

L = distance in feet between the extreme of any group of two or more consecutive axles

N = number of axles in the group under consideration

This formula is known as the federal bridge (FB) formula and was enacted by Congress in 1975 and updated in 2006. In addition to Illinois, many U.S. states use the FB formula to legislate and enforce legal truck weight limits. Although FB efficiently meets the need for setting thresholds to identify overweight vehicles, its use becomes questionable when determining the magnitude of permit fees to charge OW vehicles. This is due to the FB formula containing only variables related to the applied load (e.g., gross weight, axle spacing, and number), but no variables related to a bridge's strength.

This study proposes a novel framework where the impact of overweight vehicles is quantified on the basis of a bridge's strength. The strength of the bridges was considered by means of a variable in National Bridge Inventory (NBI) called inventory rating. The inventory rating of a bridge is defined in NBI as "capacity rating [that] will result in a load level which can safely utilize an existing structure for an indefinite period of time." Using an inventory rating allowed the amount of loading that shortens bridge service life to be quantitatively defined. This is referred to as a damaging load (DL), which essentially is the difference between the inventory rating of a bridge and the applied load.

The framework starts with analyzing the NBI and identifying the explanatory variables that affect bridge condition. Next, vehicle weight frequencies (VWF) are computed at each WIM station, which are later interpolated to the entire bridge network by using a kernel-smoothing algorithm (or non-parametric bump regression) modified with regularization techniques. Then, expected service life of bridges is calculated using a prediction engine based on support vector machines under two scenarios: with and without damaging loads. Finally, the calculated expected service life is linked to bridge life-cycle cost assessment to compute the cost of the damaging loads, which cost is later converted to the bridge fee.

4.1 BRIDGE CONDITION PREDICTION DATABASE

Accurately predicting bridge condition with respect to its age is key to developing a permit fee system where OS/OW vehicles are assessed fees in the fairest and most accurate way possible. The NBI, published and maintained by the Federal Highway Administration (FHWA), provides researchers with an extensive database that includes information about material, structure characteristics, traffic, and condition of bridge components. Detailed information about NBI can be found in the second chapter of this report. This study uses NBI for developing regression models for bridge condition prediction. It should be noted that because the deck of bridge deteriorates more rapidly than the superstructure and substructure (Bolukbasi et al. 2004; Hatami et al. 2012), prediction models were developed only for bridge decks.

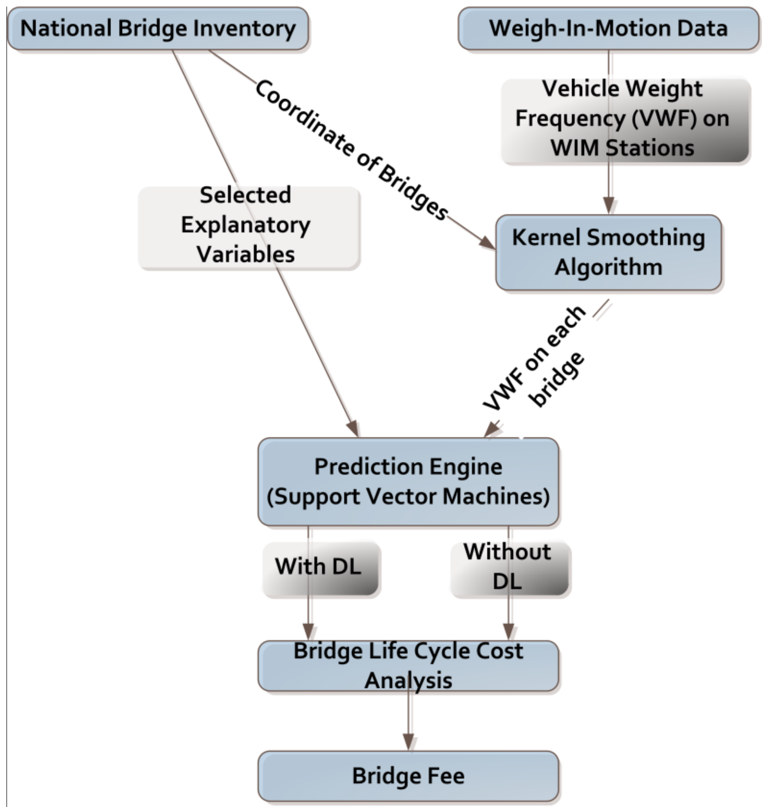


Figure 4.1: Flowchart of developed framework for bridge damage cost analysis

The first step was to clean and filter the data for developing regression models. It should be noted that data filtering and cleaning was not applied on all the variables in NBI. The variables that were preliminarily considered important for this study were extracted beforehand thereby resulting in loss of an entry (i.e., a row) because an insignificant variable was omitted. For example, there is a variable in NBI called traffic safety feature (item 36) that represents information about existing traffic safety features on a bridge, such as approach guardrail, bridge railings, and so on. This variable was omitted from NBI before data filtering and cleaning. Consequently, an entry (i.e., a row) that has blank cell for this variable but has information for other variables (e.g., deck condition, traffic) remained after the data cleaning process.

Preliminarily selected variables are as follows, along with their item code in NBI:

State code (item 1), Highway agency district (item 2), Inventory route (item 5), Structure number (item 8), Location (item 9), Latitude (item 16), Longitude (item 17), Owner (item 22), Functional class of inventory route (item 26), Year built (item 27), Lanes on and under the structure (item 28), Average daily traffic (item 29), Year of average daily traffic (item 30), Design load (item 31), Type of service (item 42), Structure type main (item 43), Structure type approach spans (item 44), Length of maximum span (item 48), Structure length (item 49), Deck (item 58), Superstructure (item 59), Substructure (item 60), Operating rating (item 64), Inventory rating (item 66), Structural evaluation (item 67), Deck geometry (item 68), Bridge posting (item 70), Approach roadway alignment (item 72), Inspection date (item 90), Designated inspection frequency (item 91), Bridge improvement cost (item 94), Year reconstructed (item 106), Wearing surface/protective system (item 108), Average daily truck traffic (item 109).

4.1.1 Data Filtering and Cleaning

Data cleaning and filtering criteria for the NBI are as follows:

The rows that have a blank cell or cells that were deemed non-applicable were omitted. It should be noted that non-applicable entries were removed from the variables in which they generally represent missing data such as deck condition or average daily traffic rather than an observation.

Duplicated rows within each year and between years were detected and removed.

The rows with the same built and reconstruction year, which can be defined as "imaginary bridges" (i.e., there was a plan to build the bridge, but the bridge was not constructed) were removed from the NBI.

The bridges that were owned by state agencies were considered in the analysis because their maintenance and evaluation processes are more reliable (Hatami and Marcous 2012). Culverts were detected and omitted from the NBI.

Unrecorded major maintenance activities are one of the main problems in the NBI. They manifest into a sudden increase in a bridge age versus deck condition curve in which there is no reconstruction or major maintenance activity reported. Detection and removal of the entries that were contaminated by unrecorded major activities are explained below.

A machine-learning algorithm was used in the analysis. Machine-learning algorithms allow developing a regression (or classification) model by solving an optimization problem (Domingos 2012), which minimizes bias (e.g., least-squares error) and variance of the developed model. In the presence of outliers, this optimization process may result in overfitting because the bias (error) from outliers may dominate the minimization. Therefore, it is a crucial yet challenging task to detect and remove outliers from data to significantly improve the accuracy and reliability of the model.

In the NBI, the main source of outliers is unrecorded maintenance and repair activities that may result in classifying bridge components in an unrealistically good condition. As seen in Figure 4.2, although there are two jumps in the data (at years 2000 and 2004) there is no maintenance activity recorded for this particular bridge.

Detection of these outliers starts with calculation of bridge age, which is the minimum of two numbers: inspection year minus built year and inspection year minus reconstruction year [i.e., min (inspection year – built year, inspection year – reconstruction year)]. For example, say that 2014 data are to be analyzed (i.e., 2014 is the inspection year). Moreover, the bridge being analyzed was constructed in 1970. In this case, the age of the bridge would be 44 years (2014 minus 1970). However, it is reported in the database that the same bridge was reconstructed (major maintenance) in 2000. Therefore, bridge age would drop to 14 years, min (44, 14).

Figure 4.2: Inspection year versus deck condition.

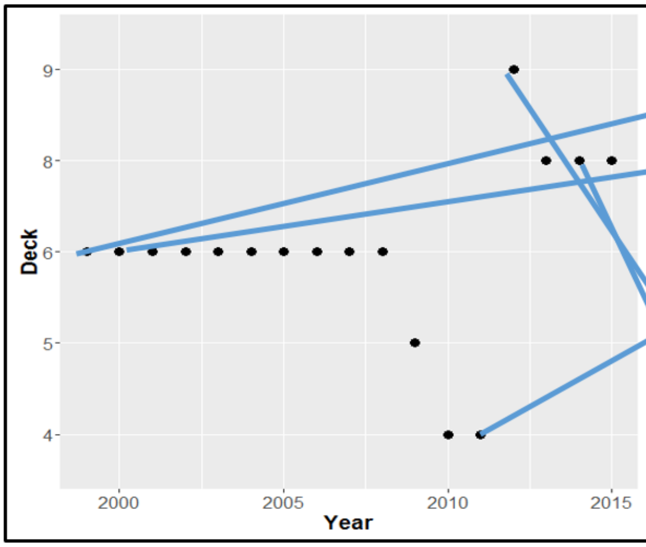
After the age of the bridge was calculated for each entry, the criteria shown in Table 4.1 (originally developed by Morcous and Hatami [2011] and extended in this study) was applied to filter out the outliers.

Table 4.1: Filtering Criteria Applied to Remove Outliers

Conditional Rating	Earliest Possible Age	Latest Possible Age
9	0	30
8	0	40
7	0	50
6	0	60
5	10	70
4	20	80
3	30	90
2	40	100

4.1.2 Building Up the Database for Model Development

After the NBI was cleaned and filtered, it was reorganized to serve as a database for data-driven model development. This process is demonstrated in Figure 4.3. As shown, each entry in the NBI was transferred to the database as a row, along with the calculated bridge age explained in the previous section and other variables. In this database, although the variable “deck” is a dependent or target variable, the other variables (material, age, and so on) are independent or explanatory variables.



Deck	Mat	Age	AADT	...
6	Steel	22	250	...
6	Steel	23	274	...
...				
4	Steel	34	300	...
9	Steel	1	324	...
8	Steel	2	324	...

Figure 4.3: Database preparation for model development.

4.2 APPLIED LOAD ESTIMATION ON BRIDGE NETWORK USING WIM DATA

Predicting bridge condition with respect to its age using the NBI has been studied extensively in the past decade (Li et al. 2010; Tabatabai et al. 2010; Hatami et al. 2012). However, because the NBI does not include any information about the applied load, deterministic approaches (e.g., finite element analysis) are preferred for quantifying the overweight load effects on bridges (Chotickai et al. 2006; Dey et al. 2014). In these studies, because of high computational cost or limited availability of the data, a few bridges out of thousands had to be selected and examined, which raised concerns about introducing bias into evaluating the bridge damage caused by OW vehicles (TRB 2016).

The framework presented in this report is one of the first attempts in the literature that tries to quantify the effect of overweight vehicles on bridges using data-driven models. Because the NBI does not have load information, it was combined with WIM to incorporate load information into data-driven models. The following section provides the steps of developing a model for estimating vehicle weight frequency within the bridge network.

4.2.1 Vehicle Weight Frequency Calculation for WIM Station

The first step in estimating the applied load on each bridge was to calculate vehicle weight frequency (VWF) for each WIM station. To calculate the frequency of vehicle gross weight (VGW), daily truck traffic was categorized into five groups (Categories A through E) based on their gross weights, and a weight frequency histogram was created (Figure 4.4a). For example, vehicles that carry weight between 41 and 60 kips are mapped into category C as 60 kips. It should be noted that vehicles heavier than 100 kips are counted proportionally to their weight. For instance, if a vehicle carries 110 kips, it is counted as 1.1 vehicle (110/100). The histogram was then normalized by total number of vehicles to compute the frequency of each weight category (Figure 4.4b).

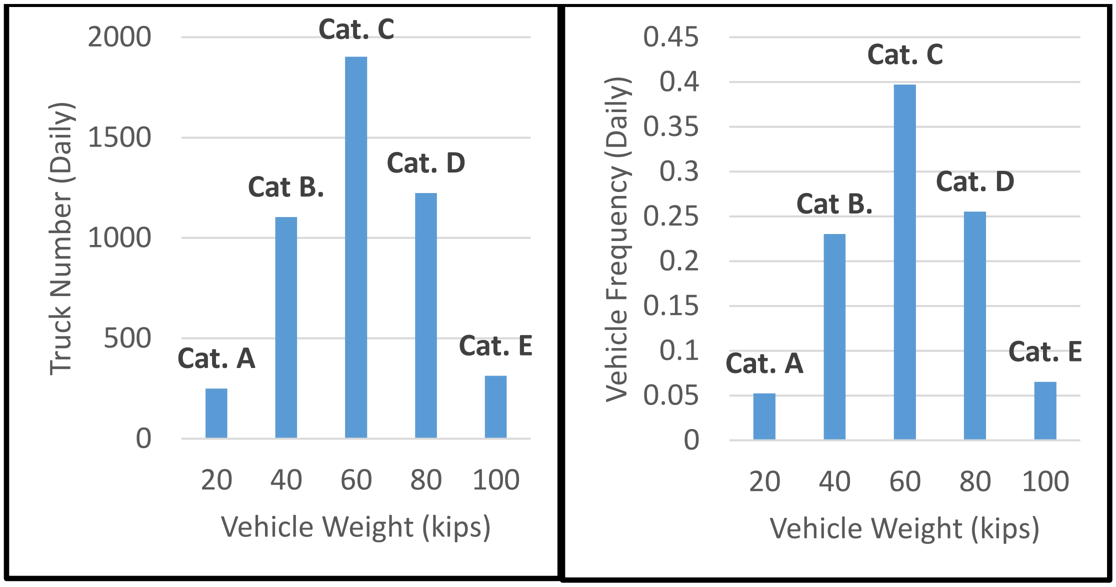


Figure 4.4: (a) VWF based on daily traffic; (b) normalized VWF.

4.2.2 Estimation of Vehicle Weight Frequency Within Bridge Network

Knowledge about the load applied on each bridge is key to quantifying the effects of overweight vehicles on bridges. WIM data is the only source available that provides information about traffic and loads. However, vehicles are weighed at the stations, which are few compared with the number of bridges in the road network. Therefore, the load applied on each bridge has been estimated, resulting in unrealistic simplifications such as assuming every point on a highway has the same traffic and load distribution as a WIM station on that particular highway. This study overcomes this limitation and introduces a novel approach that calculates the applied load on each bridge by estimating vehicle weight distribution on each bridge.

The kernel-smoothing technique (KST) was used to calculate the vehicle weight distribution at any arbitrary point within the bridge network. KST places a kernel function (or a “bump” function) to each training point, which is called a base point (e.g., WIM stations for this study). The main feature of kernel functions is that their values reach their maximum at origin (i.e., at the base point) and then fall off reasonably with the distance from origin. The weighted sum of these kernels is then used to compute the value of the target variable (e.g., vehicle categories in this study) at any arbitrary point. The formula for KST is given below:

$$y = \sum_{j=1}^{N} w_j K\left(\frac{\| x - b_j \|}{h}\right) \tag{4.2}$$

where

K = kernel function
b = base point
N = number of base points
x = an arbitrary point whose value will be estimated
y = prediction
h = width or scale of the kernel function
w_j = parameter that is computed as result of regression

Equation 4.2 can be rewritten by defining the Gram matrix (G), vector Y, and vector W as shown below:

$$G_{ij} = K\left(\frac{\| x_i - b_j \|}{h}\right) \tag{4.3}$$

$$Y_i = y_i \tag{4.4}$$

$$W_i = w_i \tag{4.5}$$

Afterward, assuming base points are chosen as training points, Equation 4.3 becomes the following:

$$Y = GW \tag{4.6}$$

In this equation, the variable h, the scale parameter of a kernel function, is chosen by cross-validation. Forsyth et al. (n.d.) define cross-validation for the KST as process of "selecting a set of scales; holding out some training points, and interpolating the values of others; then computing the error of the predictions for these held-out points."

After selecting scale parameters (h) for kernel functions, W is calculated using the least-squares minimization method. There are two main advantages of the KST over traditional interpolation techniques. First, calculated scale parameters enable the resultant model to capture the local effects in the area. Second, the rank deficient problem caused by having similar rows in the matrix (e.g., two close base points) is inherently eliminated by non-linear kernel functions. The problem with KST, however, is having too many variables, which causes high variance that consequently leads to low testing accuracy.

The variance and error of a model are negatively correlated to each other (Figure 4.5). Therefore, an optimum point should be found that minimizes the summation of variance and bias. Because Equation 4.5 has a linear algebraic form, regularization techniques used for a linear regression model to reduce variance can also be applied on KST. There are two different regularizations methods: L1 regularization (LASSO) and L2 regularization (ridge). The equation that combines error with the both regularization methods is as follows:

$$\left(\frac{1}{N}\right)\left(\hat{Y}-GW\right)^{T}\left(\hat{Y}-GW\right)+\lambda\left(\frac{(1-\alpha)}{2}\parallel w\parallel^{2}{}_{2}+\alpha\parallel w\parallel_{1}\right) \tag{4.7}$$

The first part of this equation is the least-squares error of the model, where predicted and actual values are shown by GW and $\hat{Y}$, respectively. The second and third parts present the LASSO and ridge regularization, respectively. The lambda (λ), which is calculated using cross-validation, is the trade-off parameter between error and variance. The alpha (α) is a weight coefficient between these two different regularization methods and is a user input. In this study, this equation was solved using a package known as "glmnet" (Friedman 2009) in the programming language of R.

A total of 22 WIM stations were used as base points to predict the applied load on each bridge. Latitude and longitude of the WIM stations, which are used to compute the distance between base points, were imported from IDOT (n.d.). The errors for estimating vehicle weight frequency for each category are given in Figures 4.6 through 4.8. In those plots, vertical bars show error distribution on each cross-validation to find the best lambda, and red dots show the mean of these errors. The number of explanatory variables used at each cross-validation step are given at the top of the plots. The plots show that vehicle categories can be estimated with good accuracy, with mean square errors changing from 0.000534 to 0.00337.

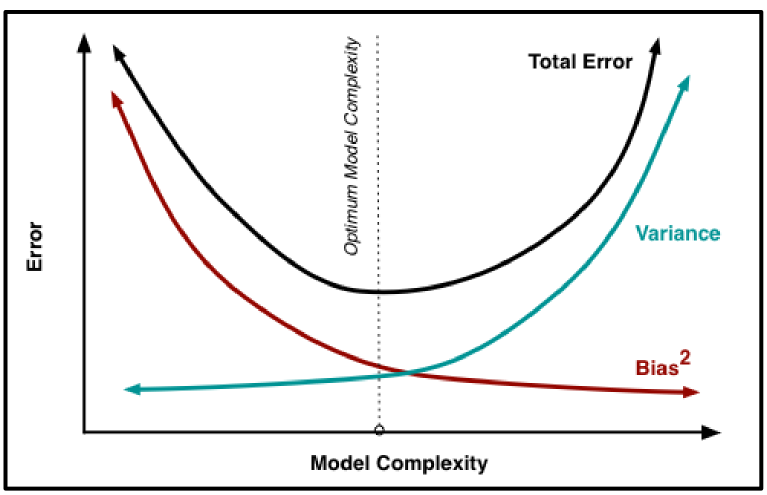


Figure 4.5: Bias–variance trade-off (Fortmann-Roe 2012).

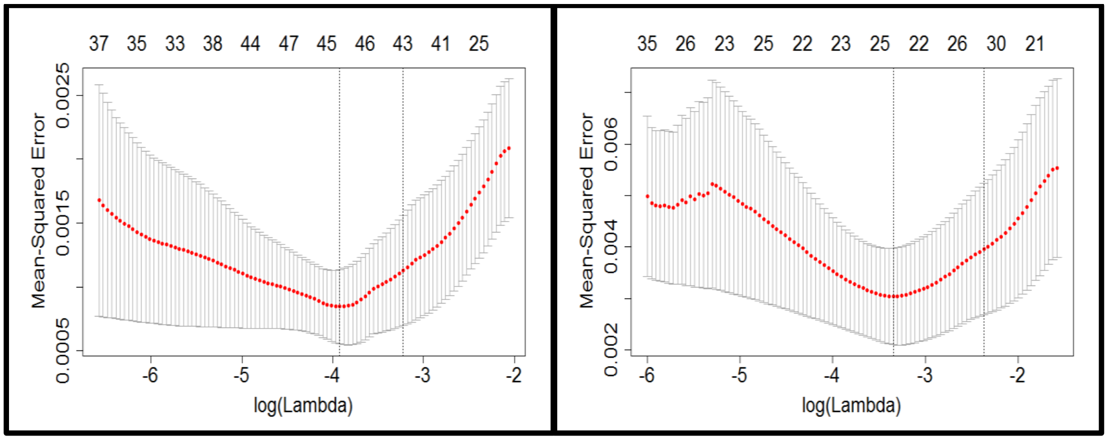


Figure 4.6: Estimation error for vehicle categories A and B.

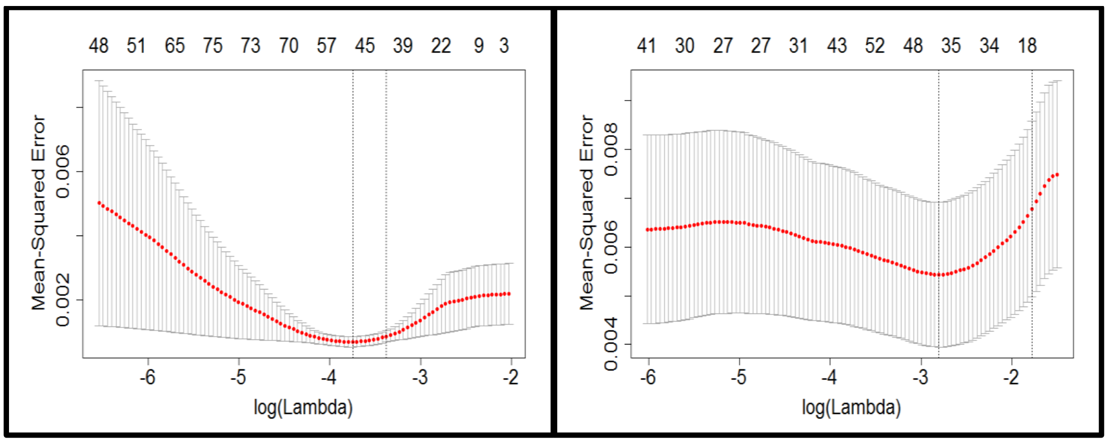


Figure 4.7: Estimation error for vehicle categories C and D.

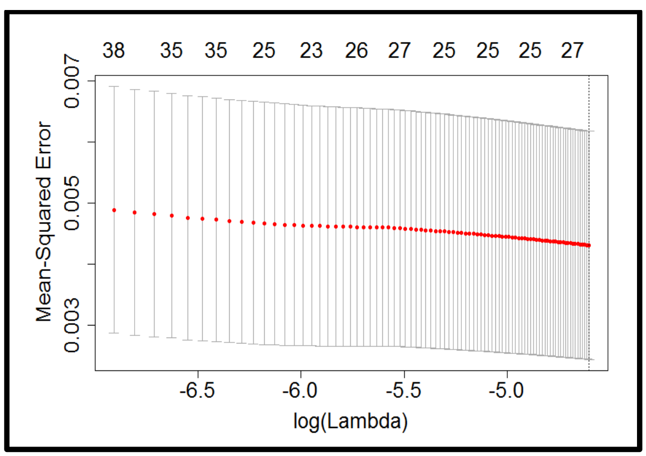


Figure 4.8: Estimation error for vehicle category E.

4.3 EXPLANATORY VARIABLE ANALYSIS AND REGRESSION MODEL DEVELOPMENT

As briefly aforementioned, there are two performance measures for data-driven models: bias and variance. Bias is simply the error of the model on training data, which is generally formulated by the least-squares error method. Variance, on the other hand, is how likely the coefficients of a model are inaccurately estimated. As was shown in Figure 4.5, when the complexity of a model increases (i.e., adding more explanatory variables) bias decreases but variance increases. Therefore, there is a trade-off between bias and variance that should be optimized using similar approaches given in Equation 4.7.

Regularization methods (explained in a previous section) significantly reduce the variation for the case of having similar type of explanatory variable such as KST. In the case of KST, all the explanatory variables were derived from kernel functions with different scale parameters and base points. However, the NBI has mixed types of variables. For example, while material is categorical, variable daily truck traffic is numerical. Therefore, explanatory variable analysis, which is explained in the following section, was conducted to eliminate insignificant explanatory variables to avoid high variance problem.

4.3.1 Explanatory Variable Analysis and Expected Applied Load Calculation

Explanatory variable analysis was conducted by combining three rationales: knowledge-based evaluation, statistical analyses, and trial and error. Knowledge-based evaluation was explained in Section 4.1. The number of variables in NBI was reduced to 38 from 123 by selecting variables that were considered important for predicting bridge condition. After many different combinations were tried, the variables used for developing the predicting models were reduced to *calculated bridge age, daily truck traffic, latitude, longitude,* and *expected applied load*. While the first four variables can be directly imported from the NBI, the expected applied load variable is required to be computed based on the estimated vehicle weight categories using the algorithm presented in Section 4.2. The sample calculation for expected applied load is given below.

Figure 4.9 presents an example of estimated vehicle weight frequency on a bridge whose latitude and longitude are 41.305° and –90.3766°, respectively. The expected applied load is calculated by multiplying frequencies by load categories. For this example, the expected applied load would be calculated as 61.1 kips (0.087 * 20 + 0.236 * 40 + 0.253 * 60 + 0.343 * 80 + 0.073 * 100).

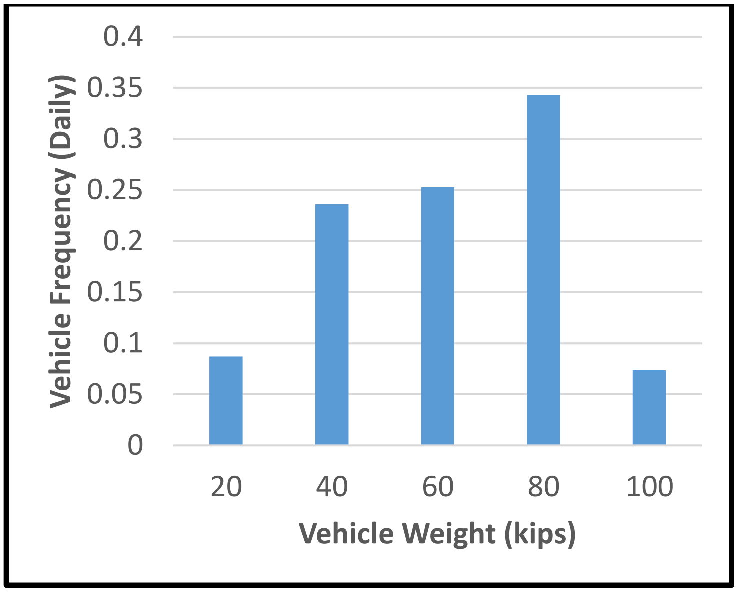


Figure 4.9: Example of estimated vehicle weight frequency on a bridge.

4.3.2 Regression Model Development

4.3.2.1 Support Vector Machine

Development of a model that uses the aforementioned selected explanatory variables as inputs to predict deck conditions required a robust prediction engine. The machine-learning algorithm known as the support vector machine (SVM) was selected because of its ability to handle non-linearities in data. It is important to note that because bridge decks deteriorate faster than other components (i.e., substructure and superstructure), the prediction models were developed for deck condition only.

SVM is a supervised machine-learning algorithm developed by Cortes and Vapnik (1995). SVM constructs a hyperplane by solving a constrained optimization problem given in Equation 4.8. The constraints (Equation 4.9) ensure that the resultant hyperplane will have the highest margin, which is the distance between the hyperplane and closest data point.

$$\frac{1}{2} \parallel w \parallel^2 + C \sum_{i=1}^{N} \xi_i \tag{4.8}$$

$$y_i(w^T x_i + b) \geq 1 - \xi_i, \xi_i \geq 0, i = 1 \ldots N \tag{4.9}$$

where

- w and b = model parameters
- C = tuning parameter for margin and training error
- ξ_i = slack variable for defining soft margin
- N = the number of data points

SVM is capable of separating the data without error if the data are linearly separable. However, for most real-life problems, including predicting bridge condition, the relationship between independent and dependent variables is highly non-linear. For linearly inseparable data, SVM uses the "kernel trick," which maps the training data into a higher dimension space where the data become more linearly separable (Figure 4.10). There are several kernel functions used in SVM. The most common ones are radial-based kernel, polynomial kernel, Gaussian kernel, and exponential kernels. The selection of the kernel function is crucial to build an accurate model (Jebara 2004). Through evaluation process, the radial-based kernel was selected for this study (Equation 4.10).

$$e^{-\gamma\|x-x\prime\|^2} \tag{4.10}$$

where

γ = scaling parameter

x and x′ = data samples

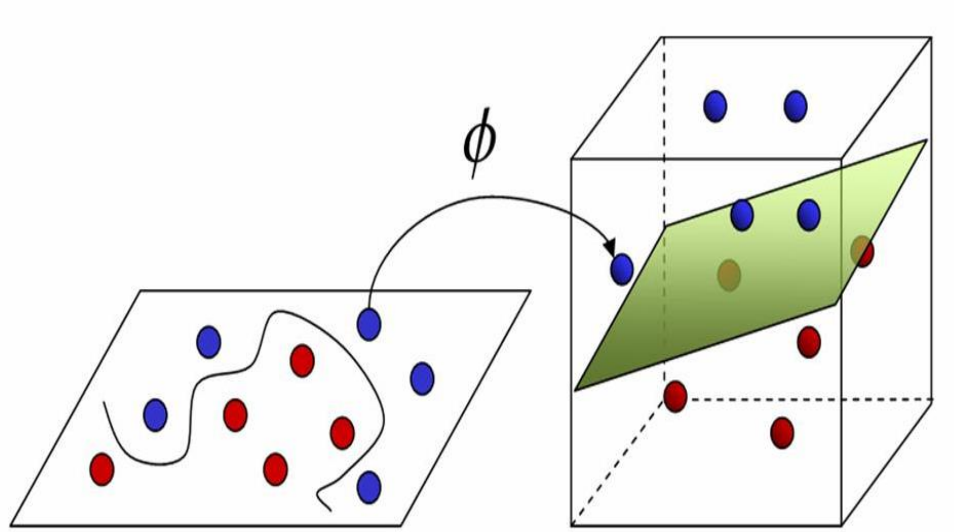


Figure 4.10: Kernel trick (Imtech).

4.3.2.2 Data Preprocessing and Model Results

The deck conditions in the NBI come from manual inspections, which are highly subjective. For example, an inspector can grade a recently constructed bridge either 8 or 9, which both correspond to perfect condition (i.e., a deck without any distress). Therefore, to reduce possible errors in the model caused by the subjectivity, all condition levels were mapped to three categories based on their definitions given in Chapter 2: good (9 and 8), medium (5–7), and poor (0–4). The mapping, however, resulted in an imbalance data problem. Figure 4.11 shows the resulting imbalanced dataset after mapping. As shown in the figure, while only 6% (5387 data points) of the dataset represent decks that are in poor condition, that number jumps to 63% (56,638 data points) for medium condition. The imbalance data problem is an important issue in model development that needs to be addressed because it compromises the performance of models (He et al. 2009). In this study reported herein, a technique known as random undersampling was used to balance the dataset. This technique randomly selects the data from each category as much as the size of the class that has the minimum number of points.

The size of the dataset was reduced to 16,160 from 89,666 after random undersampling. This dataset was later randomly split into training (80%) and testing datasets (20%). The accuracy of the model on training and testing datasets was computed as 75% and 73%, respectively.

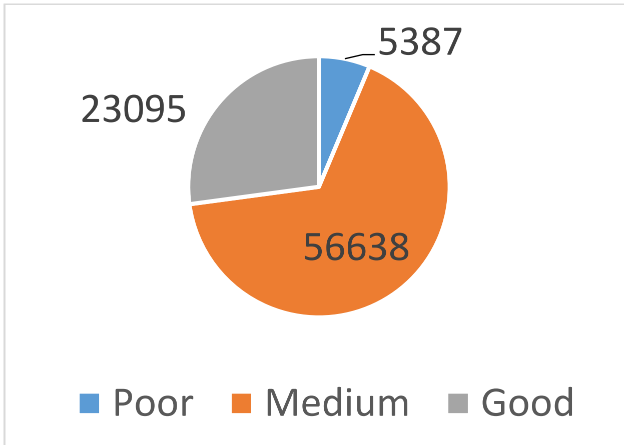


Figure 4.11: Deck condition distribution.

4.3.3 Inventory Rating and Expected Bridge Life Calculation

4.3.3.1 Inventory Rating

As mentioned before, one of the main novelties of this study is combining WIM and the NBI to account for bridge strength in assessing penalties for overweight vehicles. Accomplishing this goal required identifying the variables in the NBI that represent bridge strength. Two variables were found: operation rating and inventory rating.

The NBI defines operation rating as a "capacity rating [that] will result in the absolute maximum permissible load level." By definition, bridges are not supposed to be used by vehicles that carry weight higher than the bridges' operation rating. Therefore, this variable was not considered representative of bridge strength because it is a conservatively calculated failure limit. The second variable, inventory rating, is defined in the NBI as follows: "This capacity rating will result in a load level which can safely utilize an existing structure for an indefinite period of time." Moreover, Hearn (2014) states that inventory rating considers the current condition of the bridge. Therefore, by definition, the inventory rating represents a realistic and current bridge strength and sets a weight limit for gross vehicle weight. Vehicles that carry weight more than this limit are the ones that damage the bridges and shorten their service life. This statement, of course, considers load-related deterioration only and neglects chemical impacts on bridge performance. The inventory rating connects the NBI with WIM data and can play a key role in quantifying the overweight vehicle effect.

4.3.3.2 Expected Bridge Life Calculation Considering Overweight Impact

The prediction engine that is introduced in Section 4.3.2 is used for estimating the service life of bridges. The procedure for calculating expected service life starts with estimating the bridge condition by setting the bridge age as zero (0) and keeping the other variables the same. Then, bridge age is increased by one (1) until the p deck condition becomes 4 or lower, which is assumed to be the failure limit for decks.

The inventory rating is integrated into the model at this step. After service life is estimated for a bridge, its predicted vehicle weight frequency (Section 4.2.2) is normalized by its inventory rating. For example, Figure 4.12a demonstrates the normalized version of a histogram presented in Figure 4.4b. The inventory rating for this particular bridge is 73.4 kips. As a result of the normalization, the x-axis becomes the damage ratio from the vehicle weight. In Figure 4.12a, the vehicle categories (categories D and E in this case) with a damage ratio greater than 1 are the ones that cause load-related damage to this bridge and actually shorten the bridge service life, based on the definition of inventory rating.

The purpose is to quantify the excessive load applied on the bridge (i.e., to assess the damage caused by trucks in categories D and E). To perform this assessment, a hypothetical scenario was developed. In this scenario, damaging trucks (categories D and E) are converted into non-damaging loads, taking into account the fact that the amount of goods (i.e., amount of load) carried over a bridge should stay the same. Additionally, because the number of trucks is desired to be minimized, categories D and E are converted into category C (i.e., the non-damaging truck category with the highest capacity). This conversion results in a new daily truck number and expected load whose calculations are explained below.

For the bridge used as an example in this chapter, the daily truck number is given as 250. Moreover, the number of trucks in categories D and E is calculated as 104 using VWF, as shown in Figure 4.12a. Additionally, the expected load for this bridge was computed as 61.1 kips (Section 4.3.1). The 104 trucks (categories D and E) in this hypothetical scenario are essentially converted to category C, keeping the amount of load the same. This results in a new truck distribution (Figure 4.12b). This new distribution has the expected load of 51.32 kips. Moreover, the new truck number is calculated as 298 (250 * 61.1/51.32). Feeding this new set of inputs into the prediction engine and following the same procedure introduced in Section 4.3.3.2, the service life for the hypothetical scenario is calculated as 45 years. In summary, the expected bridge service life is reduced by 5 years as a result of damaging trucks. Quantification of this reduction regarding cost is presented in the next section.

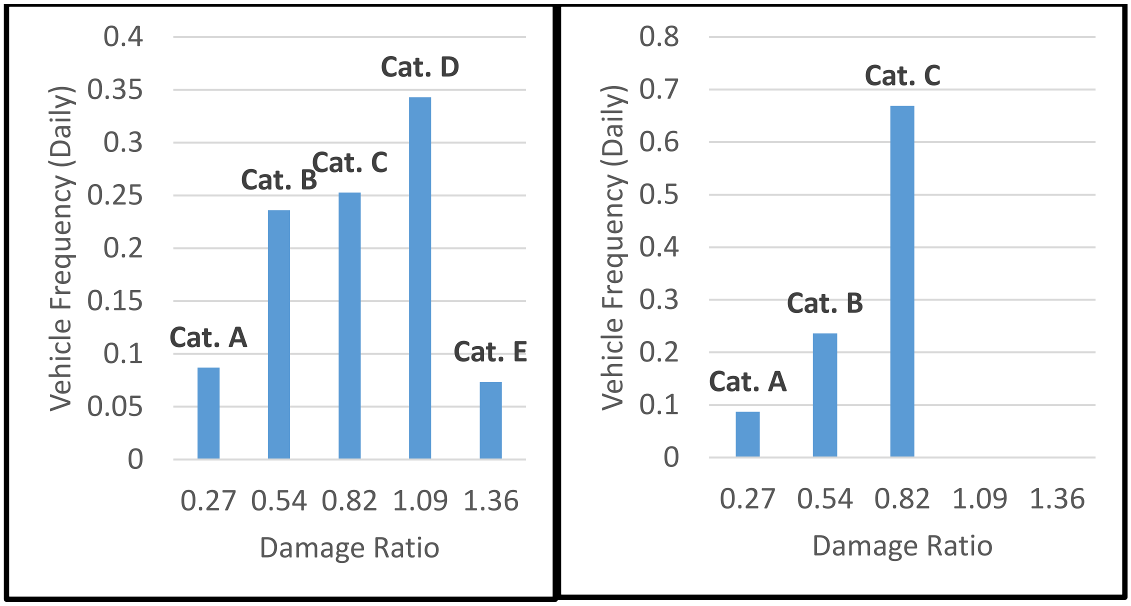


Figure 4.12: (a) normalized vehicle weight frequencies; (b) frequencies after omitting overloads.

4.4 BRIDGE LIFE-CYCLE COST ASSESSMENT AND OVERWEIGHT VEHICLE DAMAGE QUANTIFICATION

The bridge life-cycle cost assessment used in this study was adapted from Lou et al. (2016). First net present value (NPV) was calculated using Equation 4.11.

$$NPV = \sum_{t=0}^{T} \frac{C_t}{(1+r)^t} \tag{4.11}$$

where

T = lifetime of the project, which was assumed to be 75 years

C_t = The agency cost at the time (t) of construction activity; the cost for new construction (t = 0) and deck replacement were adapted as \$162.4/ft^2 and \$112.5/ft^2 (these values are computed based on the data provided by IDOT; see Appendix G)

r = discount rate, which was assumed to be as 3.00%

NPV was later converted to equivalent annual uniform cost (EUAC) using Equation 4.12.

$$EUAC = NPV\left[\frac{r(1+r)^T}{(1+r)^T - 1}\right] \tag{4.12}$$

Following this formula, EUAC can be calculated based on any given bridge service life. For the aforementioned example (5 years), the difference in EUAC would be \$0.21/ft^2 for 45 and 40 years of expected bridge service life. The damaging load, which is estimated based on daily traffic, was calculated as 9.78 kips. Therefore, the EUAC per damaging truck for this bridge would be \$5.66-7/Δkip * ft^2 (0.18/(9.78 * 365 * 104)). It should be noted that the load is multiplied by 365 to convert daily load to yearly to make the units consistent with EUAC. For a typical deck area, 7300 ft^2, the resultant cost would be computed as \$0.41E-3/Δkip, where Δkip is the difference between the applied load and the inventory rating of the bridge.

This damage quantification approach that is explained using a bridge as an example was applied on all state bridges in the database that survived the data filtering and cleaning process. The results are given in Table 4.2. As seen, the maximum reduction in bridge service life was found to be 12.26 years, while the maximum EUAC as determined is \$6.5E-04/Δkip * ft^2.

Table 4.2: Cost Quantification Results

	Mean	**Std**	**Max**	**Min**
ESL (years)	12.26	8.43	47	1
EUAC (\$/Δkip * ft^2)	3.93E-6	2.29-E5	6.5E-04	5.49E-09

CHAPTER 5: SAFETY COST QUANTIFICATION

According to statistics generated by the National Highway Traffic Safety Administration (NHTSA), there were 1,677,989 fatal and injury crashes in the United States in 2014 (NHTSA 2016a), including 61,929 in Illinois. A total of nearly 92,000 large trucks were involved in such crashes (NHTSA 2016b), including 20,363 in Illinois (IDOT 2014), or about 22% of the nationwide total. Traffic crashes related to trucks and heavy vehicles is a particular issue for the State of Illinois. The cost of one fatality in a crash, for planning purposes, is estimated to be about $4,000,000 (IDOT 2006), which imposes a significant economic impact to the society.

Transportation agencies such as the Illinois Department of Transportation (IDOT) have made various efforts to improve roadway safety, focusing specifically on reducing traffic-related fatalities and severe injuries. Given the high percentage of crashes related to trucks, a key issue for IDOT is to assess the economic impact of overweight (OW) trucks on roadway safety. In addition to establishing adequate truck permit fees, the analysis in this chapter of the impact of OW truck safety could serve multiple purposes, such as identifying exposure of roadways to truck-related crashes and quantifying the safety cost of heavy commercial vehicles.

Safety analysis has become an intensively studied topic, with the particular goal of identifying engineering, environmental, and human factors that influence crashes (e.g., roadway characteristics, weather, and driver). One of the most common methods of defining the safety of a roadway segment, intersection, or ramp is to determine the crash frequency for a given site. Although controlled experiments would help investigate the subject, they are too complex and costly to undertake in the field of roadway safety. Therefore, observational studies, which provide data-based analysis to investigate correlation between variables, have been extensively used in safety analysis (Hauer 1997; Woodrooffe 2001; Abdel-Rahim et al. 2006; Persaud et al. 2001).

Thus, one important step toward understanding safety performance of roadways and the impacts of OW trucks is to quantify the relationships among traffic volumes, other risk exposure variables, crash density (number of crashes per unit of time per unit roadway length), and crash severity. Generally, such relationships can be predicted with statistical models called safety performance functions (SPFs), which can be calibrated for various roadway types (Tegge 2008). Developing SPFs will provide realistic and accurate predictions of crash frequencies over a roadway network (segments or intersections) with respect to OW truck traffic.

This chapter presents the development of a set of Illinois-specific SPFs based on a negative binomial regression using RStudio software. The cost associated with expected crashes is also quantified. The subsequent sections describe the data processing, the development of a traffic estimation model to make up for the lack of available data, the development of SPFs through regression analysis, and finally the quantification of the economic impacts of OW trucks on roadway safety.

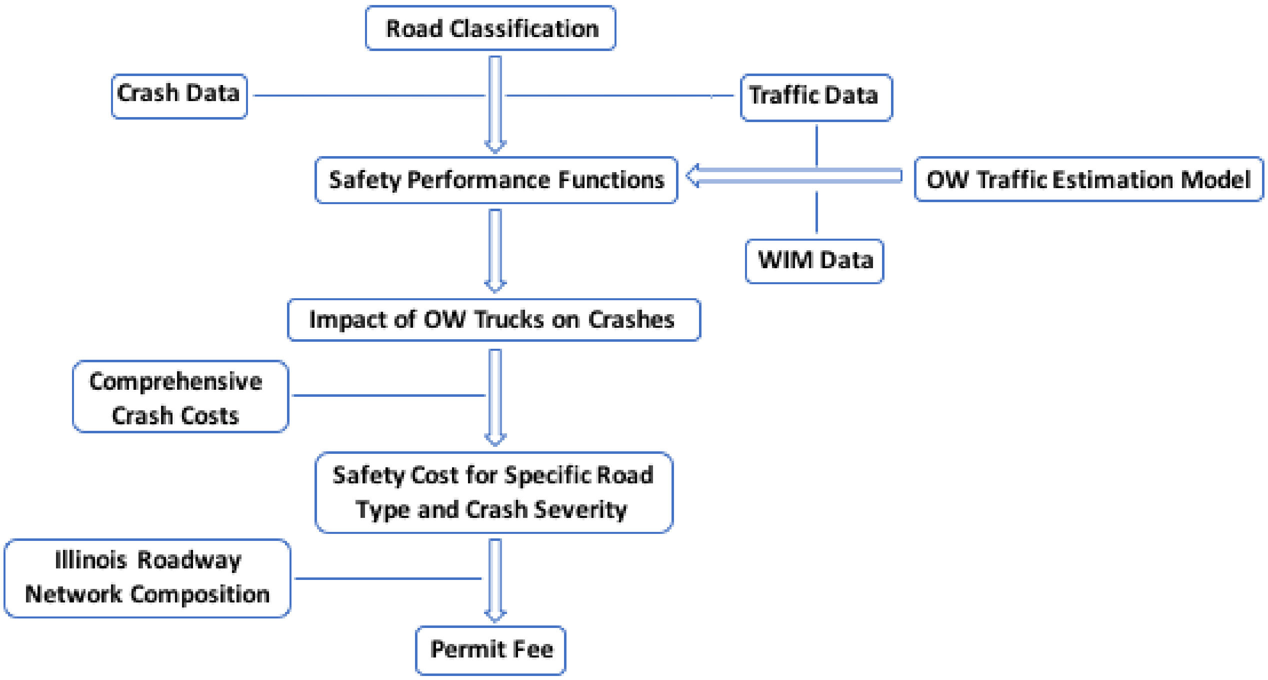


Figure 5.1: Flowchart of safety cost analysis.

5.1 DATA PROCESSING

The data available for safety analysis are presented in Chapter 2 (Sections 2.2, 2.3.1, and 2.4). Essentially, traffic data, roadway data, and crash data are the main components necessary to develop SPFs. Crash data provide the number of crashes for different severity types, while roadway data include detailed information on roadway design. Traffic data provide traffic volumes for all vehicles and heavy commercial vehicles (HCV) only.

The crash database was cleaned and merged with the traffic and roadway databases to obtain a single comprehensive dataset that can be used to develop SPFs. In the crash database, each crash was identified with its crash ID and was located with its geographic coordinates. The route was identified with its route number and milepost. In the roadway database, routes were identified with an inventory number. IDOT and CH2M Hill researchers processed the databases to obtain a comprehensive dataset that identifies each roadway segment by its inventory number and counts the number of crashes that occurred on each segment similar to the work by Tegge et al. (2010). Peer groups were also included in the roadway database.

One major limitation of the research on the impact of OW trucks on roadway safety is the poor quality or availability of crash and traffic exposure data. Most crash data collections are inadequate for studying heavy vehicles (FHWA 2013). Even though several sources are available for crash data related to truck traffic—the Fatal Accident Reporting System (FARS) from NHTSA, the Office of Motor Carrier Safety file (OMCS), the National Accident Sampling System (NASS), and the Truck Involvement in Fatal Accidents (TIFA) files from the University of Michigan Transportation Research Institute—they may not reliable. Therefore, most of the former studies collected data from police reports (Campbell et al. 1988; Fancher and Campbell 1995; Blower and Matteson 2010). Thus, data collection for HCV traffic, and more specifically for OW trucks, remains a challenge in assessing the impact of OW trucks on roadway safety performance.

The only OW truck traffic data available for the project were raw data from the WIM stations. As explained in Section 2.2.3, WIM data was processed and cleaned in this project. Weight of the truck was the only truck characteristic taken into account in this study because data on truck size were lacking. Because each entry in the dataset represented a single truck, data had to be aggregated to obtain the average annual daily OW truck traffic. The process was automated with RStudio software to obtain the OW truck volumes for each WIM station for each year available. Thus, OW truck volumes were known at some discrete locations in the Illinois roadway network. However, the data needed covered all roadway segments included in the roadway dataset.

Therefore, a traffic estimation model was developed to estimate the OW truck traffic volumes in the entire Illinois roadway network. This is introduced in the next section.

5.2 TRAFFIC ESTIMATION MODEL FOR SAFETY

5.2.1 Background

While total HCV traffic volume was available for each roadway link, the necessary OW truck traffic data were not available in the Illinois roadway network, except at a few WIM stations. Therefore, OW truck traffic volumes had to be estimated by extrapolating the existing WIM data from the discrete locations (WIM stations) to the entire network. Traffic estimation models have been extensively studied in the literature. Most studies on traffic estimation aimed at determining the origin-destination (OD) matrix; please refer to Abrahamsson (1998) and Bera and Krishna Rao (2011) for comprehensive overviews of different methodological approaches to estimating the OD matrix. However, most of the algorithm presented in Bera and Krishna Rao (2011) relied on a large set of data, such as an outdated target OD matrix or a matrix from a sample survey. In the present study, traffic counting locations are very limited with respect to the size of the Illinois roadway network, and none of those techniques adequately serves the purpose of the study.

Another approach to estimating OW truck traffic volumes is to use linear programming (LP) models of network flow. Network-based problems constitute one of the widest classes to which LP have been applied (Ahuja 1988; Revelle and Whitlatch 1996). Network-based problems include typical transportation scenarios such as the shortest-path problem (Ahuja et al. 1990), the transshipment problem (Herer et al. 2006), and the maximum flow problem (Goldberg and Tarjan 1988). The transshipment problem consists of optimizing the supply of demand nodes from source nodes through a given network at the least total distribution cost (e.g., supply chain). Typically, source nodes represent the sites of manufacturing plants and demand nodes represent warehouses that supply goods to local customers. Other nodes, which are neither demand nor source nodes, have links (roads) entering or exiting with no goods being produced or demanded. Transportation costs associated with each link or road in the network is also set. Finally, the problem comes down to determining the flows in the network to provide goods from the source nodes to the demand nodes, at the lower cost of distribution.

To clarify the objective of this section, Figure 5.2 shows a small-size problem example. It depicts a small roadway network with two WIM stations and two OW truck routes traveling through some links of the network. Based on the data available, it is known that two OW truck routes went through WIM station 1 and one went through WIM station 2. Nothing else is known. The truck traffic on all other network links need to be guessed. Because available data are very limited, multiple estimates may exist, and each could match the observed truck information at the two WIM stations. Among possible traffic guesses, the developed model will select the most reasonable one.

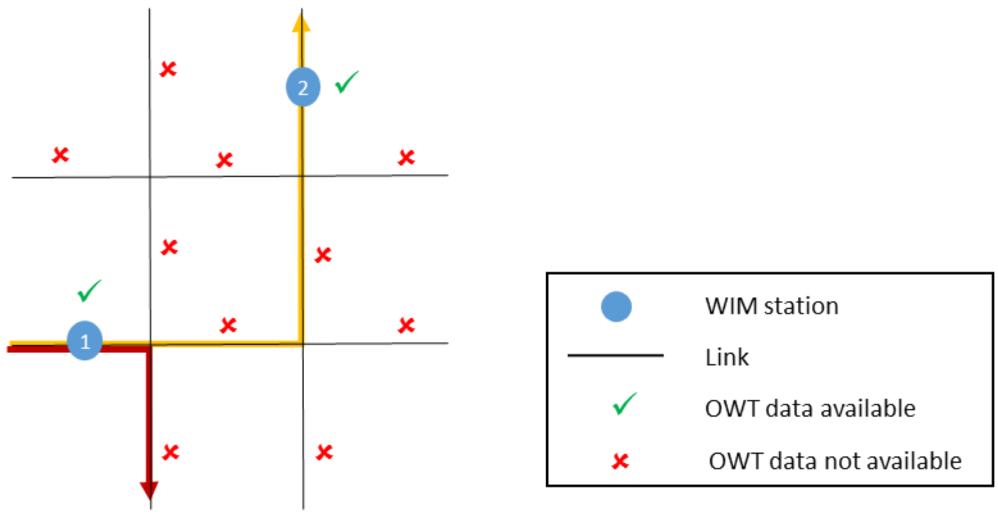


Figure 5.2: Small-size example of a roadway network with two WIM stations.

The network considered in this study is presented in Figure 5.3. It includes about 250 links, representing interstate highways, U.S. highways, and state routes in Illinois, along with 70 nodes representing major conjunctions. Among the 22 traffic WIM stations in Illinois, 13 provided useable data. Of them, seven locations had two WIM stations counting traffic in both directions. Data from those stations were presented in Section 2.2.3. Traffic count data were retrieved for each station, as explained in Section 5.1.

5.2.2 Traffic Estimation Model

It is assumed that truck travel within the Illinois network should more or less follow least-cost routes. The model discussed in this section is similar to the multi-commodity transshipment problem. WIM stations, where traffic information is known, are considered both demand and source nodes. Truck traffic for different classes represents the different commodities being transported in the network. As mentioned in Section 2.2.3, heavy vehicles in the WIM database are classified according to FHWA standards. OW trucks are considered an additional class. Thus, WIM stations send different types of goods through the network with the condition that they receive the same amount of the goods. This ensures that the traffic in a link that crosses a WIM station matches the WIM observations.

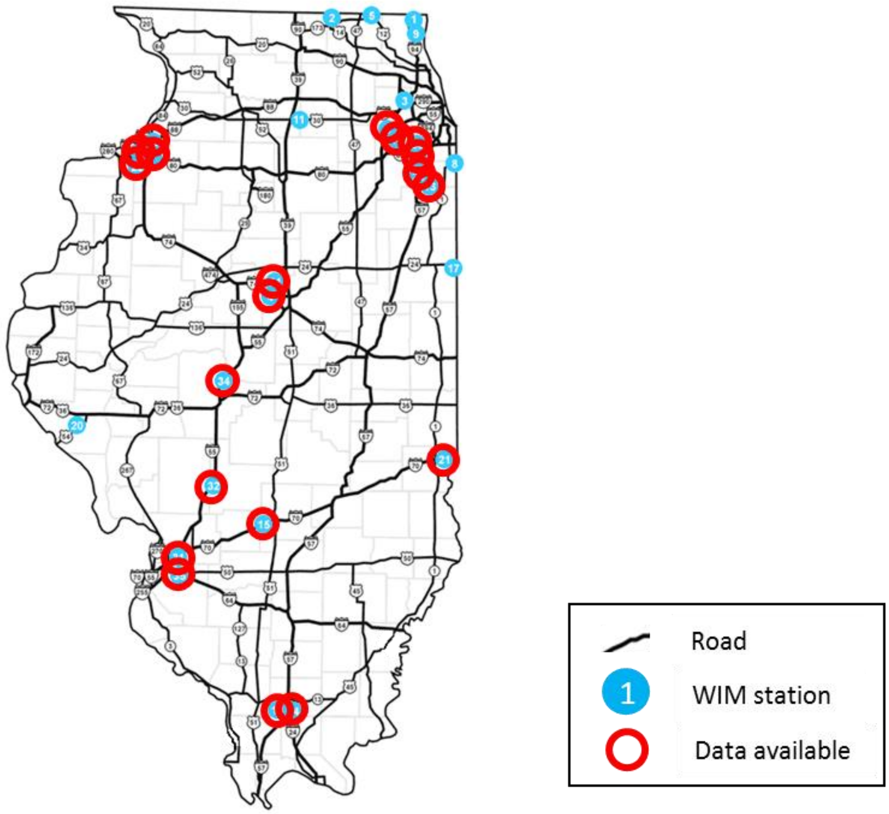


Figure 5.3: Roadway network and WIM stations.

The study focuses on the network displayed in Figure 5.2, represented as (E, V), E being the set of edges and V the set of vertices, and $L = |E|$. The length of each link is denoted as $c(u, w)$. WIM stations, $i \in I$, are considered both sources and sinks, s_i and t_i, respectively. Heavy-vehicle class is denoted by $k \in K$. The class k truck traffic at the WIM station i is represented as d_i^k. From the traffic dataset, the HCV traffic is known for each link and is denoted as $D(u, w)$, where $(u, w) \in E$. Finally, m^k is the average ratio of class k trucks among HCV, which is estimated from the 13 WIM stations. $x_k(u, w)$ and $y_k(u, w)$ are the slack variables that represent the deficit and surplus, respectively, of the estimated class k truck flow with respect to the estimated target flow $m^k D(u, w)$ for each link $(u, w) \in E$.

The decision variables, the OW truck traffic flows, are denoted by $f_k(u, w), \forall (u, w) \in E,$ and $\forall k \in K$. The first term in the objective function below captures the gap between the estimated OW truck traffic and the target expected traffic, along with the weighted sum of traffic flow and link length. The latter term of the objective function represents the total vehicular distance across the roadway network. Because trucks are expected to take the shortest path to their destination, the objective minimizes the total vehicle distance across the network.

$$\min \sum_{k\in K}\left[\left(\sum_{(u,w)\in E}\left(x_k(u,w)+y_k(u,w)\right)\right)\times max_{(u,w)}\left(c(u,w)\right)\times L + \sum_{(u,w)\in E}\left(c(u,w)\times f_k(u,w)\right)\right] \tag{5.1}$$

subject to

$$\sum_{w\in V} f_k(u,w)=0, \qquad \forall u\neq u_i,t_i, \forall i\in I, \forall k\in K \tag{5.2}$$

$$\sum_{(s_i,w)\in E} f_k(s_i,w)=\sum_{(u,t_i)\in E} f_k(u,t_i)=d_i^k, \qquad \forall i\in I,\forall k\in K \tag{5.3}$$

$$f_k(u,w)+x_k(u,w)-y_k(u,w)=m^kD(u,w), \qquad \forall (u,w)\in E,\forall k\in K \tag{5.4}$$

$$\sum_{k\in K} f_k(u,w)=D(u,w), \qquad \forall (u,w)\in E \tag{5.5}$$

$$f_k(u,w), \qquad x_k(u,w), \qquad y_k(u,w)\geq 0, \qquad \forall (u,w)\in E,\forall k\in K \tag{5.6}$$

Again, the objective function 5.1 includes the gap between the estimated traffic flow and the average flow derived from the HCV traffic, and it also includes the total vehicle distance. Equations 5.2 and 5.3 ensure flow conservation and demand satisfaction, respectively. Equation 5.4 defines the estimated target flow from the traffic composition at the WIM stations. Equation 5.5 ensures that estimated total traffic volumes (including both OW trucks and other vehicles) match exactly with the observed total HCV volumes on other roadway links. Equation 5.6 is a non-negativity constraint, to ensure non-negative traffic flows, and slack variables.

5.3 SAFETY PERFORMANCE FUNCTIONS

5.3.1 Multivariate Models

Regression models that relate crash frequency to traffic and other location characteristics are often referred to as multivariate models (Persaud et al. 2001). Accident prediction models are intended to estimate the relative safety of a location by determining an SPF. According to the FHWA (2014), SPFs are “equations used to predict the average number of crashes per year at a location as a function of exposure and, in some cases, roadway or intersection characteristics (e.g., number of lanes, traffic control, or median type).”

Thus, such functions help to quantify the expected number of crashes for various design variables and condition settings (Tegge 2008; Hauer 2004; Bauer and Harwood 2000; Miaou 1994; Lord 2000; Vogt and Bared 1998).

The most commonly used form for SPFs (Tegge 2008; Bauer and Harwood 2000; Persaud et al. 2001; Hauer and Persaud 1996; Hauer 2004) is the following:

$$\lambda_i = (Segment\ length)_i \cdot (e^{\beta_0}) \cdot (AADT_i^{\beta_1}) \cdot exp(\beta_2 X_{2i} + \cdots + \beta_q X_{qi}) \quad (5.7)$$

where λ_i is the mean of the variable Y_i, the crash frequency of roadway segment *i*.

To model the SPF, Hauer (2004) split it into two parts: a multiplicative portion and an additive portion. The former accounts for factors that apply to stretches of road (e.g., traffic flow, lane width, shoulder type), and the latter accounts for factors that apply locally (e.g., number of driveways, short bridges). Tegge (2008) split SPFs into two categories. Level 1 SPFs are models based only on traffic volumes for both segments and intersection. Past studies found that average annual daily traffic (AADT) has the largest impact on the variability of the accident data (Tegge 2008; Bauer and Harwood 2000). Level 2 SPFs also incorporate other variables to model the crash frequency, under the form of Equation 5.7 Thus, to account for the OW truck traffic impact on crash frequency, the following SPF is used in the remainder of the report.

$$\lambda_i = (SL)_i \cdot e^a \cdot \left(\frac{AADT_i}{10{,}000}\right)^b \cdot e^{c*OWT_i} \quad (5.8)$$

where λ_i is the predicted number of crashes on a given segment i, SL_i is the length of segment i in miles, $AADT_i$ is the average annual daily traffic of segment i, OWT_i is the average annual daily OW truck traffic of segment i, and a, b, and c are the regression coefficients to be determined.

It is probably a reasonable assumption that the distribution of OW and OS traffic in the Illinois roadway network is highly correlated. Due to the lack of information on OS traffic, the model assumes, for the sake of simplicity, that the mixture ratio of OW and OS vehicles is homogeneous across the network, such that the spatial distribution of OS vehicles is proportional to that of OW vehicles. The contribution of OS vehicles to safety can therefore be captured indirectly by the included OW traffic variable (as a proxy) in the developed SPFs – the coefficient of the OW traffic variable actually now includes the impact of OS vehicles. Of course, the validity of this assumption can be tested in the future once OS data become available.

5.3.2 Log-Linear Analysis

One commonly used regression model is the negative binomial (NB) model. The NB distribution describes the occurrence of discrete events and deals with the overdispersion problem (Dean and Lawless 1989; Miaou et al. 1996, as cited in Persaud et al. 2001) by allowing the variation to be larger than the mean. Essentially, the NB model allows for variation based on variables that were not included in the model (Tegge 2008). Equation 5.9 expresses the probability of a random variable that follows an NB distribution to take value y_i, as follows:

$$P(y_i) = \frac{\Gamma(y_i + \frac{1}{k})}{y_i!\Gamma(\frac{1}{k})} \left(\frac{k\lambda_i}{1+k\lambda_i}\right)^{y_i} \left(\frac{1}{1+k\lambda_i}\right)^{1/k} \quad (5.9)$$

The variance can be expressed as $\lambda_i + k(\lambda_i)^2$, where λ_i is the mean defined in Equation 5.8. Hence, this regression model assumes that the crash frequency follows an NB distribution with parameters λ_i and k.

5.3.3 Implementation and Results

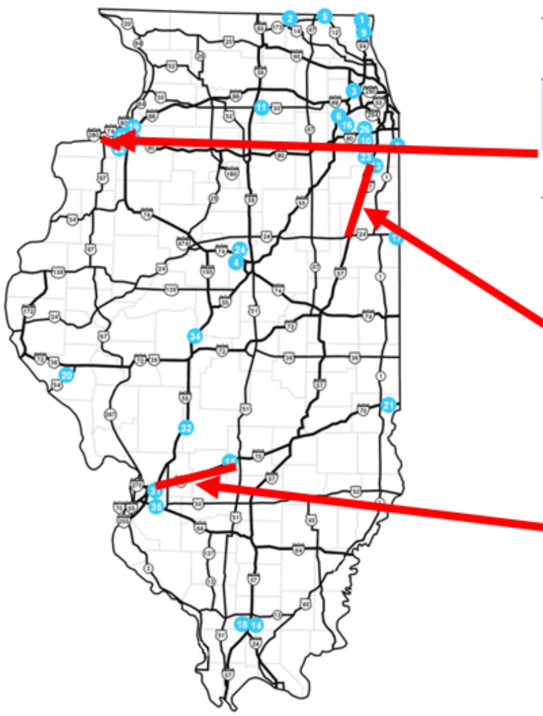
First, the traffic estimation model was implemented using AMPL software (Fourer et al. 2003). The problem for Illinois has about 760 variables and 340 constraints. The LP is solved successfully by the commercial solver CPLEX (CPLEX 2008). Some demonstrative results are shown in Figure 5.4.

The Illinois-specific SPFs were developed based on NB regressions using R regression analysis tools. As previously stated, the NB regression is an appropriate regression method for data that have low occurrence frequency and where the variance exceeds the mean. The SPFs were developed for a 5-year period; therefore, the number of crashes was produced for a time unit of 5 years instead of on a year-by-year basis (similar to Tegge et al. 2010).

Safety performance functions for roadway segments, using Equation 5.8, were developed for each peer group of homogeneous (variable length) segments. They included two elements to estimate the predicted number of crashes: segment length and traffic volumes.

Using the *glm.nb* function in the RStudio software, estimates of the regression parameters were found for various SPF peer groups. The software estimated the three regression coefficients and the overdispersion parameter using the maximum likelihood (MLE) method. A summary of the Illinois segment SPFs is provided in Table 5.1. It displays the regression coefficients, dispersion parameter, number of segments, and total miles with each peer group.

Overall, total vehicular traffic has a significant impact on crashes, represented by the value of b. For some peer groups, the value of the coefficient c that pertains to OW trucks was not statistically significant; thus, it was dropped from the analysis. In that case, only a and b were determined through regression analysis. The value of c for K-crashes was significant for most of the peer groups. The peer group urban undivided highway yields the largest value of c, which means that OW truck traffic's greatest impact occurs on this type of road. Note that the value of a is negative for K-crashes; because this type of crash has the lowest occurrence, the value of e^a will scale down the rest of the multiplicative portion of Equation 5.8.



Category	Class 6-7	Class 8-10	Class 11-13	OWT
Annual count of trucks	12,439	351,489	28,363	79,935
Percentage (%)	2.63	74.43	6.00	**16.94**

Category	Class 6-7	Class 8-10	Class 11-13	OWT
Annual count of trucks	34,910	752,036	46,154	44,895
Percentage (%)	3.98	85.65	5.26	**5.11**

Category	Class 6-7	Class 8-10	Class 11-13	OWT
Annual count of trucks	13,547	992,374	100,773	56,940
Percentage (%)	1.16	85.28	8.67	**4.89**

Figure 5.4: Sample from the traffic estimation results.

Table 5.1: SPF Parameters and Characteristics for Peer Groups

Peer group	K-crashes ($/mi)	A- and B-crashes ($/mi)	Total miles in Illinois
Rural undivided highway	0.02	0	9,290
Rural divided highway	0.10	0	1,638
Urban undivided highway	0.46	0.04	2,963
Urban divided highway	0.02	0	2,195

5.4 COST ESTIMATION

Many researchers have estimated human capital costs and comprehensive costs associated with human fatalities and injuries. The NHTSA, the National Safety Council (NSC), and IDOT provide economic costs of motor vehicle crashes. Tables 5.2 and 5.3 display the 2006 cost estimates used by IDOT for planning purposes. The 2006 values were adjusted using a GDP deflator (U.S. Bureau of Labor Statistics 2015). Other sources found in the literature include the U.S. Department of Transportation (US DOT), the U.S. Environmental Protection Agency (US EPA), and the U.S. Department of Agriculture (USDA) (Viscusi 1995; Miller 2000; Dionne and Lanoie 2002; Mrozek and Taylor 2002).

Table 5.2: Comprehensive Costs of Motor Vehicle Crashes on Urban Highways (IDOT 2006)

Crash type	2006 dollars	2015 dollars
Fatal (K)	\$4,239,438	\$5,197,156
Incapacitating injury (A)	\$251,633	\$295,839
Non-incapacitating injury (B)	\$63,738	\$74,935

Table 5.3: Comprehensive Costs of Motor Vehicle Crashes on Rural Highways (IDOT 2006)

Crash type	2006 dollars	2015 dollars
Fatal (K)	\$4,374,698	\$5,143,243
Incapacitating injury (A)	\$258,578	\$304,004
Non-incapacitating injury (B)	\$63,041	\$74,116

Using the SPFs from the previous section, the safety impacts of OW trucks can be directly quantified. SPFs capture the impact of OW truck traffic and provide a parametric relationship to expected crashes. Total vehicle traffic and truck traffic are related to the expected number of crashes at each roadway site. Then, each expected crash can be associated with a predetermined economic loss value based on IDOT guidelines. These costs can be used to estimate overall OW truck-related safety costs.

For each roadway peer group, the parameters *a*, *b*, *c*, and *k* have been determined (Table 5.1); therefore, once AADT is fixed, the expected number of crashes is simply a function of a single variable: OW truck traffic. The derivative of this function can be seen as the marginal increase, $\Delta\lambda$, of the expected number of crashes if the OW truck traffic increases by ΔOWT (Figure 5.5).

For this analysis, the relationship between the variations of AADT and OW truck traffic was considered to compute the derivative of the SPFs. A linear relationship was assumed among the changes in AADT, $\partial AADT$, and the changes in OW truck traffic, ∂OWT. A reasonable assumption would be that the goods transported by one OW truck would be loaded onto an 80-kip truck, which is the maximum gross vehicle weight (GVW) allowed in Illinois. OW trucks are estimated to carry, on average, 100-kip of goods (determined from the WIM database); therefore, 1.25 80-kip trucks would be needed to replace a single OW truck.

$$\partial AADT = -1.25 \cdot \partial OWT \tag{5.10}$$

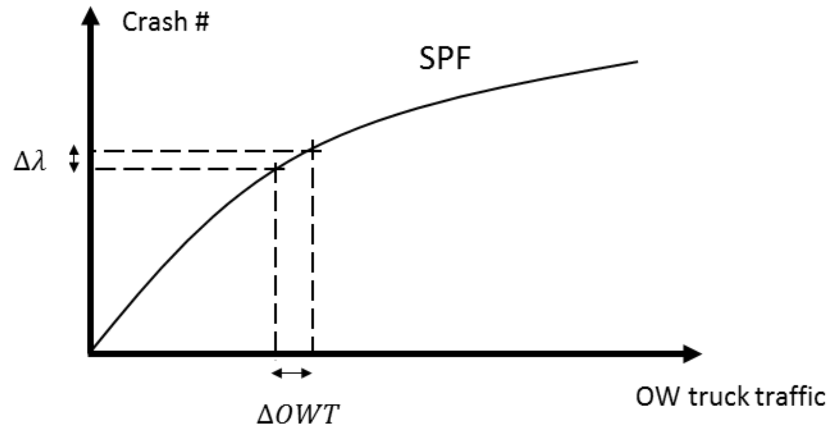


Figure 5.5: Marginal increase of crashes for a given AADT.

From the derivative of Equation 5.8, with respect to OW trucks, the impact of variations of the OW truck traffic on the safety performance of a particular roadway segment can be estimated.

$$\frac{\partial \lambda_i}{\partial OWT} = (Segment\ length)_i \frac{\mathrm{e}^a \cdot \mathrm{e}^{cOWT_i}}{10,000}\left(\frac{AADT_i}{10,000}\right)^{b-1}\left[-1.25\cdot \mathrm{b} + \mathrm{c}\cdot \mathrm{AADT}_i\right] \qquad (5.11)$$

Then, the safety performance of a roadway segment can be approximated per unit length as follows:

$$\frac{1}{(Segment\ length)_i}\frac{\Delta \lambda_i}{\Delta HCV} = \frac{\mathrm{e}^a \cdot \mathrm{e}^{cOWT_i}}{10,000}\left(\frac{AADT_i}{10,000}\right)^{b-1}\left[-1.25\cdot \mathrm{b} + \mathrm{c}\cdot \mathrm{AADT}_i\right] \qquad (5.12)$$

The unit of this indicator will be in number of crashes per mile for a given severity type.

The safety performance of a peer group can be taken as the average safety performance of all its roadway segments. Then, the safety performance of a peer group $p \in P$ can be translated into costs using the comprehensive costs presented in this section.

$$Cost_p = \left(C_\$\right)\frac{1}{N_p}\left[\sum_{i\in P}\frac{\mathrm{e}^a \cdot \mathrm{e}^{cOWT_i}}{10,000}\left(\frac{AADT_i}{10,000}\right)^{b-1}\left[-1.25\cdot \mathrm{b} + \mathrm{c}\cdot \mathrm{AADT}_i\right]\right] \qquad (5.13)$$

where N_p is the number of segments in peer group p and $C_\$$ is the comprehensive cost corresponding to the severity type and road type.

As shown in Table 5.1, A-injury and B-injury crashes were aggregated into a single category. The cost corresponding to a crash from this category was estimated as the weighted sum of A-crash and B-crash costs (Tables 5.2 and 5.3) and their average count (see Section 2.4). Thus, for A+B crashes, the unit cost of a crash was taken as $137,195 for urban highways and $138,908 for rural highways.

Table 5.4 summarizes the different costs per unit length associated with fatal and non-fatal crashes. The larger costs pertain to K-crashes and the peer group urban undivided highway. On such roadways, the traffic is usually denser. Owing to the absence of a shoulder dividing the highway, vehicles are more likely to off track or to swerve into the opposite lane. Because OW trucks are also less efficient in unusual maneuvers (e.g., obstacle avoidance) and heavier than most other vehicles, they are more likely to cause fatalities. Another interesting insight from the results is the absence of the impact of OW trucks on the safety of urban divided highways. Overall, OW truck traffic has a non-negligible impact on fatal crashes, compared with its impact on non-fatal crashes.

Table 5.4: Summary of Crash Costs per Unit Length for Each Crash Severity

Segment group	K-crashes ($/mi)	A+B crashes ($/mi)
Rural undivided highway	0.02	0
Rural divided highway	0.10	0
Urban undivided highway	0.46	0.04
Urban divided highway	0.02	0

5.5 SUMMARY

Traffic, roadway, and crash data were provided by IDOT, in addition to data from the WIM stations. The dataset included traffic data, roadway data, and crash data. However, OW truck traffic data were partially missing. Thus, the study first developed a traffic estimation model to assess OW truck traffic on the Illinois roadway network. A multi-commodity flow model for OW truck traffic was developed and run with a commercial optimization software.

The development of SPFs is a critical element of safety analysis. They predict the expected number of crashes per year based on given roadway's characteristics. The study developed a set of SPFs for four roadway peer groups, and for fatal and combined A- and B-injury crashes. The SPFs took OW truck traffic into account to assess the impact of such trucks. SPFs coefficients were estimated using regression analysis. Note that this study did not focus on crashes in which OW trucks were directly involved; rather, it focused on the direct and indirect impacts of OW truck traffic on expected roadway safety.

After developing the SPFs, the study analyzed the costs related to crashes by severity type, and developed a simple methodology to translate the SPFs into dollar costs caused by OW truck traffic per unit of travel length. Costs will vary with respect to crash severity and road type. Therefore, once the route of an OW truck has been determined, it is possible to quantify the cost associated with this route based on the different segments (type and length) that compose the route.

CHAPTER 6: RECOMMENDATION OF PERMIT FEE

6.1 PERMIT FEE DUE TO PAVEMENT DAMAGE

As shown in the analysis in Chapter 3, PDC varies significantly depending on pavement type and traffic level. The following assumptions were made to determine the permit fee across Illinois. The traffic level was assumed at the 50th percentile (median) of the accumulated ESAL distribution in Illinois. The median values of freezing index and CRS before rehabilitation in Illinois were selected to estimate the effect of climate condition and exiting pavement condition on pavement damage cost. For interstate highways, this results in PDC of full-depth HMA and HMA/PCC pavements equal to 4.93 cents/ESAL-mile and 2.70 cents/ESAL-mile, respectively. For non-interstate highways, the PDC of full-depth HMA and HMA/PCC pavements equals 132.80 cents/ESAL-mile and 54.83 cents/ESAL-mile.

The median of the percentage of VMT on interstate highways (78.59%) was used to estimate the mileage traveled in interstate highway by the overweight truck. The relative percentage of road segments with different pavement types in the routes traveled by the overweight truck was assumed based on the IRIS database. According to the pavement-type records in the IRIS database, 5.25% of interstate highways are full-depth HMA pavements, and 10.68% of the non-interstate highways are full-depth HMA pavements. The remaining segments are mainly HMA/PCC pavements. Therefore, the universal pavement damage cost was obtained using the weighted average value of pavement damage cost on different highway types and pavement types. The average value of pavement damage cost equates to 15.7 cents/ESAL-mile.

The PDC caused by an individual truck can be estimated using Equation 6.1. In this case, the PDC caused by an individual truck with a combination of different axles is equivalent to the linear combination of the damage caused by each axle. The extra PDC caused by an overweight truck is due to the tonnage exceeding legal weight limits. The pavement cost difference between the overweight truck and the legal-weight truck can be calculated to determine a permit fee, as shown in Equation 6.1. The critical loading condition was considered to compute the extra ESALs induced by overweight. The excessive weight exceeding legal weight limits was added on the critical trailing axles but not on the steering axle.

$$\text{Permit fee per mile} = [(\textstyle\sum_{\mathrm{i}=1}^{\mathrm{k}} ESALs_{\mathrm{i}})_{\text{overweight}} - (\sum_{\mathrm{i}=1}^{\mathrm{k}} ESALs_{\mathrm{i}})_{\text{legal}}] \times \text{PDC} \quad (6.1)$$

where

k = number of axle groups for the truck

PDC = pavement damage cost (15.7 cents/ESAL-mile from this study)

$ESALs_i$ = number of equivalent single-axle loads for each axle group that can be calculated using the load-equivalency factors defined in the AASHTO 1993 pavement design method:

$$\text{ESALs} = 8\text{E-}6 \times L1^{4.0393} + 9\text{E-}7 \times L2^{3.972} + 3\text{E-}7 \times L3^{3.8914} + 2\text{E-}7 \times L4^{3.7678}$$

where

L1, L2, L3, ad L4 = axle load of single, tandem, tridem, and quad axles

For example, the number of ESALs from a six-axle OW truck with a GVW of 84 kips and steering, tandem, and tridem axle loads weighing 12 kips, 30 kips, and 42 kips, respectively, equals 1.47 (ESALs = 8E-6 * $12^{4.0393}$ + 9E-7 * $30^{3.972}$ + 3E-7 * $42^{3.8914}$). The corresponding number of ESALs from a legally loaded six-axle truck with a GVW of 80 kips and steering, tandem, and tridem axle loads weighing 12 kips, 26 kips, and 42 kips equals, respectively, equals 1.18 (ESALs = 8E-6 * $12^{4.0393}$ + 9E-7 * $26^{3.972}$ + 3E-7 * $42^{3.8914}$). Therefore, the extra pavement use induced by the OW truck is 0.29 ESALs. The permit fee associated with pavement damage can be calculated as the cost resulting from the extra pavement use, which in this example equals $0.0455/mi.

6.2. PERMIT FEE RELATED TO BRIDGE DAMAGE

The framework that quantifies the damage caused by excessive loads is presented in Chapter 4. The framework is novel in that it computes the fee on the basis of bridge strength rather than on just the applied load. The average cost is calculated as \$3.93E-6/Δkip * ft^2 (see Chapter 4).

The ideal implementation of this framework requires that bridge information be obtained for bridges on a route. Bridge strength is then obtained from the NBI, and the damaging load (i.e., the difference between bridge inventory strength and the gross weight of an OS/OW vehicle) is calculated. Finally, this value is multiplied by the bridge deck area, also obtained from the NBI. Given that IDOT already has a GIS-based online tool for issuing permits, integration of the developed framework would be relatively straightforward.

The developed framework, however, can also be converted into a table to conform to the existing permit fee calculation scheme by averaging the cost over all bridges. To achieve that, the average deck area and the number of bridges per mile are needed. Average deck area is reported by IDOT as 10,679 ft^2 (state-owned bridges only). The centerline mileage and number of bridges were provided by IDOT as 15,969 miles and 7847 bridges (1856 parallel structured), respectively. To avoid double counting the parallel structures, they were divided by two and subtracted, resulting in a modified number of bridges of 6919 (7847 – 1856 * 0.5). Based on the aforementioned inputs, the average per-mile cost of damage is calculated as \$0.0182/mi * Δkip (3.93E-6 * 10,679 * 6919/15,969), where Δkip should be calculated by subtracting the gross weight of an overweight vehicle from the average inventory rating of state-owned bridges extracted from NBI as 80.38 kips for 2015 (the most recent data available).

6.3. PERMIT FEE RELATED TO SAFETY

On the basis of the information in Chapter 5, the following unit costs were derived to reflect the impacts of OW truck safety impacts on Illinois roadways. Unit costs may vary depending on road type and crash severity type (Table 6.1).

The permit fee could be derived directly based on the mileage of each roadway peer group along an OW truck route, using the unit safety costs from Table 6.1.

Alternatively, for standardization, the proposed permit fee weighs the unit safety cost of each road type with respect to the total segment length of each road type in Illinois. The underlying assumption is that the truck route composition in terms of peer group mileage is expected to be similar to the composition of the Illinois roadway network.

Table 6.1: Crash Costs per Unit Length for Each Crash Severity

Peer group	K-crashes ($/mi)	A- and B-crashes ($/mi)	Total miles in Illinois
Rural undivided highway	0.02	0	9,290
Rural divided highway	0.10	0	1,638
Urban undivided highway	0.46	0.04	2,963
Urban divided highway	0.02	0	2,195

Let C_p^K and C_p^{A+B} denote the unit costs of K-crashes and A+B crashes, respectively, for each peer group $p \in P$. The total miles of each peer group, denoted as $L_p, \forall p \in P$, can be found in Table 6.2. Thus, the proposed permit fee, in dollars per mile, is derived as follows:

$$C_u = \left(\frac{1}{\sum_{p \in P} L_p}\right) \sum_{p \in P} \left(C_p^K + C_p^{A+B}\right) \cdot L_p \tag{6.3}$$

$$C_u = \$0.117/mi$$

Thus, the average per-mile fee that would reflect the impact of OW trucks on the safety of Illinois roadways is estimated to be about $0.12 per mile.

6.4. INTEGRATION OF VARIOUS PERMIT FEES

The recommended permit fee is calculated based on the aggregation of the impacts OW trucks impose on pavements, bridges, and roadway safety. These events are considered independent of each other, and therefore the permit fee is a summation of the three individual fees calculated in previous sections. It should be noted that these three fees are represented as functions of different quantities. In particular, the pavement fee is calculated based on a formula that depends on the mileage, axles, and weight information. The bridge fee formula depends on the weight and number of bridges. The safety fee formula, on the other hand, depends only on the mileage.

The total permit fee formula is a function of mileage, axles, and weight information. For the bridge fee formula, in order to convert the per-bridge unit into per-mile, we calculate the average number of bridges per mile in Illinois. Based on the modified total number of IDOT owned bridges (6919) and the total centerline mileage in the Illinois network (15,969, owned by IDOT), the average ratio of 0.43 bridge per mile was considered. Truck axle weight information was used for pavement damage assessment, truck gross weight information was used for bridge damage assessment, while all OW truck traffic was aggregated to estimate safety impacts.

The suggested OW permit fee structure is as follows:

$$\text{OW Fee (\$)} = \text{miles} * (0.117 + 0.157\Delta\text{ESAL} + 0.0182\Delta\text{kips}) \tag{6.4}$$

where

$$\Delta ESAL = (\sum_{\mathrm{i}=1}^{\mathrm{k}} ESALs_{\mathrm{i}})_{\mathrm{overweight}} - (\sum_{\mathrm{i}=1}^{\mathrm{k}} ESALs_{\mathrm{i}})_{\mathrm{legal}}$$

$\Delta\mathrm{kips} = W_{ow} - M_{inventory}$

ESALs = 8E-6 x $L1^{4.0393}$+ 9E-7 x $L2^{3.972}$ + 3E-7 x $L3^{3.8914}$ + 2E-7 x $L4^{3.7678}$

k = number of axle groups for the truck

L1, L2, L3, and L4 = axle load of single, tandem, tridem and quad axles

W_{ow} = Gross weight of an overweight vehicle

$M_{inventory}$ = Average inventory rating of stated owned bridges (80.38 kips from NBI)

6.4.1 Scenario-Based Evaluation of Recommended Fee

This section evaluates the recommended fee (Equation 6.4) by comparing it with current permit fees in Illinois and its neighboring states. The scenario-based approach was followed for evaluating the recommended fee because it was suggested in Bilal et al. (2010) as the ideal way for comparing the permit fee across states.

Four scenarios were used for the comparison. The first one was provided by IDOT (referred as "IDOT"), and the others were adapted from the literature (Adams et al. 2014). The detailed information about the scenarios are provided in Table 6.2.

The results of the comparison are given in Tables 6.3 through 6.6 for each scenario. The second column, "Recom.," gives the OW fee values based on the recommended OW fee formula (Equation 6.4). The third column "IL," gives the values for existing fees in the State of Illinois, which are calculated using 625 Illinois Compiled Statutes (ILCS) 5/15-307. Most of the fees in 625 ILCS 5/15-307 were established in 1981. In 1981, the Engineering News Record (ENR) Construction Cost Index was 3535. However, in 2016, the ENR Construction Cost Index was established as 10,403. This leads to an inflation ratio of 2.94 (10,403/3535). Using this ratio, the current fee in Illinois was adjusted and is presented in the column titled "IL (Adj)." The remaining columns show corresponding states' fees (detailed fee structures for each states are given in Table C.1).

It is important to note that the recommended fee considers only the damage cost of OW vehicles. The recommended OW fee does not include any processing fees (such as the $50 base fee specified in 625 ILCS 5/15-307(g)), investigative fees (such as the engineering inspection or field investigation fee specified in 625 ILCS 5/15-311), police escort fees (such as those specified in 625 ILCS 5/15-312), etc. Additionally, oversize fees (such as the fees specified in 625 ILCS 5/15-304 and 305) are excluded due to a lack of traffic data on vehicle size/dimension. Finally, fees associated with an overweight axle(s) on a legal gross weight vehicle (such as those specified in 625 ILCS 5/15-306) were also not evaluated in this study.

Table 6.2: Information About Scenarios

	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Load Description	**IDOT**	**Steel Bridge Girder**	**Combine Harvester**	**Generator**
Gross Weight (lb)	174000	112000	90000	132000
Axle 1 (lb)	14000	12000	10000	12000
Axle 2 (lb)	20000	20000	16000	24000
Axle 3 (lb)	20000	20000	16000	24000
Axle 4 (lb)	20000	20000	16000	24000
Axle 5 (lb)	20000	20000	16000	24000
Axle 6 (lb)	20000	20000	16000	24000
Axle 7 (lb)	20000			
Axle 8 (lb)	20000			
Axle 9 (lb)	20000			
Axle Spacing (1 to 2) (in)	250	234	197	193
Axle Spacing (2 to 3) (in)	53	54	55	54
Axle Spacing (3 to 4) (in)	102	113	55	456
Axle Spacing (4 to 5) (in)	51	54	504	54
Axle Spacing (5 to 6) (in)	454	54	55	54
Axle Spacing (6 to 7) (in)	51			
Axle Spacing (7 to 8) (in)	51			
Axle Spacing (8 to 9) (in)	51			

Table 6.3: Comparison Results for Scenario 1

Miles	Recom.	IL	IL (Adj)	IN	MO	MI	KY	WI
45	$133	$124	$365	$75	$628	$50	$60	$179
90	$265	$198	$582	$120	$828	$50	$60	$179
135	$398	$272	$800	$165	$828	$50	$60	$179
183	$539	$351	$1,032	$213	$828	$50	$60	$179
225	$663	$420	$1,235	$255	$1,128	$50	$60	$179
270	$795	$494	$1,452	$300	$1,128	$50	$60	$179
315	$928	$568	$1,670	$345	$1,128	$50	$60	$179
360	$1,060	$642	$1,887	$390	$1,128	$50	$60	$179
405	$1,193	$716	$2,105	$435	$1,128	$50	$60	$179
450	$1,325	$790	$2,323	$480	$1,128	$50	$60	$179

Table 6.4: Comparison Results for Scenario 2

Miles	Recom.	IL	IL (Adj)	IN	MO	MI	KY	WI
45	$57	$75	$221	$47	$79	$50	$60	$55
90	$113	$100	$294	$74	$79	$50	$60	$55
135	$170	$126	$370	$101	$79	$50	$60	$55
183	$230	$152	$447	$130	$79	$50	$60	$55
225	$283	$176	$517	$155	$79	$50	$60	$55
270	$340	$201	$591	$182	$79	$50	$60	$55
315	$396	$226	$664	$209	$79	$50	$60	$55
360	$453	$252	$741	$236	$79	$50	$60	$55
405	$510	$277	$814	$263	$79	$50	$60	$55
450	$566	$302	$888	$290	$79	$50	$60	$55

Table 6.5: Comparison Results for Scenario 3

Miles	Recom.	IL	IL (Adj)	IN	MO	MI	KY	WI
45	$19	$30	$88	$36	$35	$50	$60	$20
90	$38	$55	$162	$52	$35	$50	$60	$20
135	$57	$80	$235	$67	$35	$50	$60	$20
183	$77	$130	$382	$84	$35	$50	$60	$20
225	$95	$130	$382	$99	$35	$50	$60	$20
270	$114	$155	$455	$115	$35	$50	$60	$20
315	$133	$180	$529	$130	$35	$50	$60	$20
360	$152	$205	$603	$146	$35	$50	$60	$20
405	$171	$230	$676	$162	$35	$50	$60	$20
450	$190	$255	$750	$178	$35	$50	$60	$20

Table 6.6: Comparison Results for Scenario 4

Miles	Recom.	IL	IL (Adj)	IN	MO	MI	KY	WI
45	$107	$91	$268	$57	$119	$50	$60	$75
90	$214	$132	$388	$84	$119	$50	$60	$75
135	$320	$173	$509	$111	$119	$50	$60	$75
183	$434	$217	$638	$140	$119	$50	$60	$75
225	$534	$255	$750	$165	$119	$50	$60	$75
270	$641	$296	$870	$192	$119	$50	$60	$75
315	$747	$337	$991	$219	$119	$50	$60	$75
360	$854	$378	$1,111	$246	$119	$50	$60	$75
405	$961	$419	$1,232	$273	$119	$50	$60	$75
450	$1,068	$460	$1,352	$300	$119	$50	$60	$75

CHAPTER 7: SUMMARY, CONCLUSION, AND RECOMMENDATIONS

7.1 SUMMARY

In this study, overweight (OW) vehicles were evaluated based on their impacts on pavements, bridges, and roadway safety to determine updated permit fees for these vehicles. In quantifying these impacts, the most recent databases on infrastructure condition and state-of-the-art prediction/ classification algorithms were used.

To quantify the impact on pavements, performance deterioration models were developed using the following:

- Condition rating survey (CRS) data extracted from the Illinois Roadway Information System (IRIS) database
- Detailed traffic data obtained from both the IRIS database and weigh-in-motion (WIM) sites located along some of the interstate highways

Life-cycle cost analyses (LCCA) were performed to estimate pavement damage cost. The permit fee for specific overweight axles and trucks was determined based on axle-load configuration and excessive load magnitude.

In assessing the impact on bridges, Gaussian mixture models were developed for weight distribution on each bridge. The applied load (i.e., gross weight of vehicles) and bridge load-carrying capacity were considered for quantifying the impact of OW vehicles on bridges. Along with applied load information and National Bridge Inventory (NBI) data, prediction models were developed that use bridge characteristics (e.g., age, location, inventory ration) as input and bridge condition as output. These models were used to determine bridge life reduction per damaging load, which is defined as load greater than the load level that can safely utilize an existing structure for an indefinite period of time.

To calculate the impact on safety, customized statistical tools were developed to quantify the relationships between total vehicle traffic, OW truck traffic, and crash frequency. The costs related to crashes by severity type were obtained from IDOT, and a simple methodology was developed to translate Illinois-specific safety performance functions (SPFs) into marginal safety costs caused by OW truck traffic per unit of travel length. Costs vary with respect to crash severity, traffic exposure, and roadway type, while averages are computed across Illinois roadways.

Through these assessments, a combined permit fee (presented in the form of OW vehicle per mile traveled) was introduced based on the increase in pavement and bridge damages as well as the increase in crashes. (i.e., the permit fee is based on bridge and pavement damage as well as safety impact.) Truck axle weight information was used for pavement damage assessment and truck gross weight information was used for bridge damage assessment, while all OW truck traffic was aggregated to estimate safety impacts.

7.2 CONCLUSIONS

- **Pavements**
    - Pavement life was found to be affected by truck traffic, freezing index, and existing pavement condition before rehabilitation.
    - Models were developed to estimate the life of HMA surface pavements and Portland cement concrete (PCC) pavements.
    - The estimated pavement damage cost varies significantly depending on truck traffic volume and highway type. Truck loading would cause greater pavement damage cost on a non-interstate highway with low truck traffic volume.
    - The overall permit fee for each category was estimated based on excessive weights with the assumptions of the representative truck traffic volume, the vehicle miles traveled on interstate highways, and the relative percentage of pavement types in the Illinois highway network.
- **Bridges**
    - Bridge data for Illinois were filtered and cleaned, and related parameters were selected for model development.
    - Data-driven models were developed to predict bridge service life considering bridge age, bridge strength, daily expected applied load, and location of the bridge.
    - Quantification of damaging load that shortens bridge life was conducted using bridge life-cycle cost analysis.
- **Safety**
    - A set of customized Illinois-specific safety performance functions (SPFs) were developed to assess the impact of overweight trucks on safety on Illinois' highway network.
    - This empirical study developed state-of-the-art methodologies for data processing, network OW truck flow modeling, and statistical regression analysis for SPF development. Detailed SPF results and marginal safety cost estimates were averaged across the State of Illinois.
    - It is recommended that a safety-related fee be implemented based on the miles traveled.

7.3 RECOMMENDATIONS

- It is recommended that the developed algorithms be integrated into the currently existing GIS-based permit system used by IDOT. This integration will provide specific and accurate inputs to the developed models and algorithms (e.g., OW truck weight and configuration, route plan, bridges and roadway segments traveled, and traffic exposure), which will consequently generate a more accurate route-based OW permit fee.
- A combined fee (Equation 6.4) that covers truck weight, axle weight, and mileage and their impact on pavements, bridges, and safety was developed. The use of this fee is recommended as an alternative to the previously suggested GIS-based fee calculation.

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APPENDIX A: TRUCK SIZE AND WEIGHT LIMITS

The federal government took the lead in regulating commercial motor vehicle weights and dimensions on the interstate and other national highways that carry the majority of truck traffic. This appendix presents the development of vehicle size and weight permit regulations in the United States, as well as European truck size and weight regulations.

A.1 U.S. TRUCK SIZE AND WEIGHT REGULATIONS

The U.S. federal interest in preserving the highway infrastructure dates back to the enactment of the Federal-Aid Highway Act of 1956, which established size and weight standards for trucks that used the national system of interstate and defense highways (FHWA 2015). In 1974, federal weight ceiling limits were increased to help offset a large increase in fuel prices, yet some states refrained from adopting the higher limits. The Surface Transportation Assistance Act (STAA) of 1982, per Section 133(a), Section 127 of Title 23 of the United States Code, stated that no funds were to be apportioned to any state that does not allow vehicles with a weight of 20,000 lb (9,072 kg) carried on one axle, or with a tandem-axle weight of 34,000 lb (15,422 kg), or a gross weight of at least 80,000 lb (36,287 kg) for vehicle combinations of five axles or more. In 1983, the overall maximum gross weight on a group of two or more consecutive axles was restricted by the following bridge formula:

$$W = 500\left(\frac{LN}{N-1} + 12N + 36\right) \tag{A.1}$$

where

W = overall gross weight of any group of two or more consecutive axles to the nearest 500 lb

L = distance in feet between the extreme of any group of two or more consecutive axles

N = number of axles in the studied group

Although the STAA allowed twin trailers, Section 1023 of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 limited the increased use of double-trailer combinations (longer-combination vehicles) with a gross weight higher than 80,000 lb (36,287 kg). The term “longer-combination vehicles” means any combination of a truck tractor and two or more trailers or semi-trailers, with a GVW greater than 80,000 lb, that operates on the interstate system.

Since 1991, there have been no major changes in vehicle weight and size restrictions except by the states of Alaska, Ohio, and Wyoming. In addition to the vehicles allowed under Paragraph A of Section 1023 of ISTEA, the State of Wyoming allowed the operation of additional configurations authorized by state law no later than November 3, 1992. Those additional vehicle configurations had to comply with the single-axle, tandem-axle, and bridge formula limits, and not exceed 117,000 lb GVW. In Ohio, in addition to vehicles that the state continued to allow to operate under ISTEA, the state allowed a longer combination of vehicles with three cargo-carrying units up to 28.5 ft each to be operated within its boundaries on a 1 mi segment of Route 7.

Several research projects have been carried out since 1991 to investigate the impact of OS/OW vehicles on the transportation infrastructure. Some of the major efforts that were undertaken in the United States were the Turner Proposal, the U.S. Department of Transportation (US DOT) Comprehensive Truck Size and Weight Study, and the Western Uniformity Scenario.

A.1.1 The Turner Proposal

In 1989, formal federal highway administrator Francis Turner suggested a new approach, which was evaluated by the Transportation Research Board (TRB), to regulate the sizes and weights of trucks using U.S. roads. The proposal promised to reduce road wear caused by truck loading while improving the productivity of freight transport. The approach projected there would be trucks with lower axle and axle group weights, but more axles than existing vehicular loading, and greater allowable gross weights. The scope of the study included reducing the legal single-axle loading to a maximum of 15,000 lb and tandem axles to 25,000 lb, allowing greater vehicle lengths, and raising the maximum gross weights to as much as 112,000 lb. The report anticipated that these limits would reduce the cost of shipping freight without compromising safety. It further anticipated that the total cost of maintaining the road network would be reduced, although pavement-wear savings would be partially offset by higher bridge costs.

The major large truck that was considered in the study was a 5-axle tractor/semi-trailer with a maximum weight of 80,000 lb and length of 50 to 65 ft. The most common multi-trailer combination had a maximum weight of 80,000 lb, two 28 ft trailers, five axles, and an overall length of 70 ft. The vehicular configurations considered in the Turner Proposal report included a wide range of possible values of length limits, overall weights, and axle weights to obtain the most desirable performance in terms of pavement wear, bridge cost, safety, and productivity. The following configurations are the prototypes that were considered in the proposal:

- 11-axle A-train double-trailer combination with a maximum weight of up to 141,000 lb (63,957 kg)
- 9-axle B-train double-trailer combination with a maximum weight of 114,000 lb (51,710 kg), two 33 ft (10 m) trailers and overall length of 81 ft (24.7 m)
- 9-axle double-trailer combination with a maximum weight of 114,000 lb (51,710 kg), two 33 ft (10 m) trailers and overall length of 81 ft (24.7 m)
- 7-axle tractor/semi-trailer with a maximum weight of 91,000 lb (41,277 kg), trailer length of 48 ft (14.6 m), and overall length of 60 ft (18.2 m)

The 9-axle A-train double-trailer combination was the most desirable to motor carriers. It was anticipated that the adoption of the proposal would result in a 23% reduction in existing truck miles within 5 to 10 years. In addition, the proposed configurations were expected to offer a slight reduction in truck crashes and a small decline in truck interference with regular traffic flow because the total annual miles of combination-truck travel would decline.

The major costs to the transportation agencies were bridge reconstruction and maintenance. The report suggested replacement of 7,000 interstate and primary highway bridges at an estimated cost of $2.8 billion, with an additional $4.1 billion to replace bridges on the non-primary system. In addition, there would be an additional cost of $110 million per year for new bridge construction and a $28 million annual cost once the proposed-configuration truck traffic reached its long-term level. The estimated pavement-wear savings would be $729 million annually, while the maintenance cost of the roadway system would decrease by $326 million.

The recommendations of the study pertaining to Turner's trucks (bullet list above) were made regarding vehicle restrictions including truck weight, dimensions, equipment, and route restrictions, as well as the aforementioned estimates of impacts on the nationwide introduction of Turner's trucks. Table A.1 presents the suggested vehicle restrictions.

Table A.1: Turner Study's Suggested Vehicle Restrictions

Criteria	Restriction
Weight Limits	
Single Axle	15,000 lb maximum
Tandem Axle	25,000 lb maximum
Tri-Axle	40,000 lb maximum
Gross Combination	Not specifically limited
Length Limits	
Single Trailer	48 ft maximum
Double Trailers	33 ft maximum each
Overall	85 ft maximum
Width Limits	102 in
Configuration Restrictions	Double-tank trailers permitted only in B-train or double-drawbar-dolly configurations
Other Equipment Restrictions	
Tires	Loading not to exceed 600 lb/in tire width
Brakes	Anti-lock braking system on drive axles and dolly
Route Restrictions	Permitted on all roads except route segments blocked by bridges failing to meet specified load capacity criteria

A.1.2 US DOT Comprehensive Truck Size and Weight Study

This study (U.S. Department of Transportation 2000) focused primarily on development and testing of analytical tools to estimate potential diversion of traffic from one type of truck to another, or diversion between truck and rail, if truck size and weight limits were changed. This study also made improvements over previous studies by considering inventory and other logistics costs to shippers in making transportation decisions.

Impacts of proposed size and weight changes considered to be most critical were safety, productivity, infrastructure (pavement, bridges, and geometrics), traffic congestion, environment, and railroads.

The impacts of different trucks on those criteria were assessed for five truck size and weight scenarios:

- Uniformity: Imposed federal weight limits on all non-network highways and removed grandfathered vehicles under provisions in current federal law. This resulted in a gross weight cap of 80,000 lb on all national network routes, and long combination vehicles (LCV) became impractical. LCVs can be a combination of multiple trailers on tractor trucks, compared with standard 5-axle semi-trailer trucks with one trailer.
- North America trade: Increased allowable tridem axle loads to be more consistent with limits in Canada and Mexico. Tridem loads of 44,000 and 51,000 lb were considered.
- LCVs nationwide: Allowed LCVs on a nationwide network, with the largest LCVs restricted to a designated network; however, triple combinations and doubles with 33 ft trailers were allowed more flexibility.
- H.R. 551: Three provisions related to federal truck size and weight limits in this scenario would phase out trailers longer than 53 ft, freeze state grandfathered rights, and freeze weight limits on non-interstate portions of the national highway system.
- Triples nationwide: Allow triple-trailer combinations to operate nationwide at a gross weight up to 132,000 lb—the same as in the LCV nationwide scenario.

The Comprehensive Truck Size and Weight Study showed significant productivity gain for each scenario that allowed heavier vehicle weight, with the greatest gain generated by LCVs. Increasing the semi-trailer length in some states has resulted in increased cubic capacity. The standard semi-trailer length increased from 45 ft to 53 ft, and some states allowed semi-trailers up to 60 ft long. Cost recovery, however, was a significant issue when considering an increase in truck size and weight limits, which has undoubtedly increased the cost to maintain the infrastructure. Some states capture a large share of these costs throughout permit fees, but other states undercharge for these increases and barely covered their administrative costs.

The USDOT completed another study in 2015 that is directed by the Moving Ahead for Progress in the 21st Century Act (MAP-21). The purpose of the study was to investigate the impact of operation of large trucks on bridges, pavements, roadway safety, fuel efficiency, the environment, and enforcement. Six scenarios were developed to model the impact of increasing the size and weight limits (Figure A.1.). It was concluded that the results of this study could not be used to make general decisions for the entire country because of limitations in the models and data. Therefore, this study did not produce any explicit recommendation about changing federal limits for size or weight limits on vehicles. This study, however, successfully identified the needs and required improvements in data collection to more accurately assess the effect of increasing the size/weight limits of vehicles.

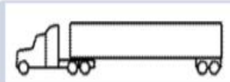
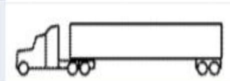
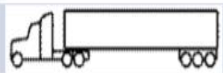
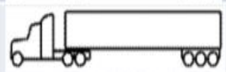
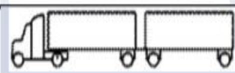
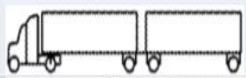
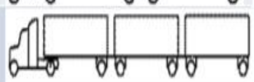
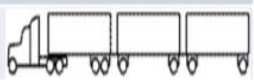
Scenario	Configuration	Depiction of Vehicle	# Trailers or Semitrailers	# Axles	Gross Vehicle Weight (pounds)	Roadway Networks
Control Single	5-axle vehicle tractor, 53 foot semitrailer (3-S2)		1	5	80,000	Currently operating on the entire Interstate System and National Network, including most of the National Highway System
1	5-axle vehicle tractor, 53 foot semitrailer (3-S2)		1	5	88,000	Modeled to use same networks as above
2	6-axle vehicle tractor, 53 foot semitrailer (3-S3)		1	6	91,000	Modeled to use same networks as above
3	6-axle vehicle tractor, 53 foot semitrailer (3-S3)		1	6	97,000	Modeled to use same networks as above
Control Double	Tractor plus two 28 or 28 ½ foot trailers (2-S1-2)		2	5	80,000 maximum allowable weight 71,700 actual weight used for analysis[2]	Modeled to use same networks as above
4	Tractor plus twin 33 foot trailers (2-S1-2)		2	5	80,000	Modeled to use same networks as above
5	Tractor plus three 28 or 28 ½ foot trailers (2-S1-2-2)		3	7	105,500	Modeled to use a 74,500 mile roadway system including the Interstate System, approved routes in 17 Western States allowing triples, and certain four-lane roads in the Eastern United States.
6	Tractor plus three 28 or 28 ½ foot trailers (3-S2-2-2)		3	9	129,000	Modeled to use same networks as above

Figure A.1: Scenarios Analyzed

A.1.3 Western Uniformity Scenario

Following the Comprehensive Truck Size and Weight Study, the Western Governors' Association requested that US DOT analyze a policy option that would allow the western states of Colorado, Idaho, Kansas, Montana, Nebraska, Nevada, North Dakota, Oklahoma, Oregon, South Dakota, Utah, Washington, and Wyoming, to harmonize LCV weights and dimensions at levels that meet existing federal axle-load limits and the federal bridge formula, and to be consistent with the guidelines of the Western Association of State Highway and Transportation Officials (WASHTO).

Several substantial improvements were made to the Comprehensive Truck Size and Weight Study, in the Western Uniformity Scenario analysis, including truck and rail data, as well as methods used to analyze pavement, bridge, and safety impacts. The Western Uniformity Scenario study used the freight analysis framework commodity-flow data. The analysis included the following vehicles: 5-axle tractor semi-trailers, twin 28.5 ft trailers, and five LCVs—Rocky Mountain doubles, turnpike doubles, triples, a 10-axle resource-hauling double, and 8-axle B-trains. The following criteria were used:

- 20,000 lb for a single axle on the interstate system
- 34,000 lb for a tandem axle on the interstate system
- Application of the bridge formula for other axle groups, up to the maximum 80,000 lb GVW on the interstate system

- Vehicle width of 102 in on the national network
- Minimum semi-trailer length of 48 ft in a semi-trailer combination on the national network, and minimum 28 ft length for trailers in a twin-trailer combination on the national network
- Grandfathered rights under which certain LCVs are allowed to operate in each scenario state
- LCVs that were permitted by state law but subject to the LCV freeze

The study found several benefits from allowing more widespread use of LCVs, including a reduction in fuel consumption, emissions, and noise-related costs.

A.2 EUROPEAN TRUCK SIZE AND WEIGHT REGULATIONS

Tables A.2 presents the permissible maximum dimensions for trucks in Europe, while Table A.3 shows the permissible maximum weights in Europe, according to the International Transport Forum (2013).

Table A.2: Permissible Maximum Dimensions for Trucks in Europe (International Transport Forum 2013)

			Length		
Country	**Height (ft)**	**Width (ft)**	**Lorry/Trailer (ft)**	**Road Train (ft)**	**Articulated Vehicle (ft)**
Belgium	13.12	8.37	40	61.5	54
Denmark	13.12	8.37	40	61.5	54
Finland	13.78	8.53	40	82.8	54
France	ND	8.37	40	61.5	54
Germany	13.12	8.37	40	61.5	54
Ireland	15.25	8.37	40	61.5	54
Italy	13.12	8.37	40	61.5	54
Netherlands	13.12	8.37	40	61.5	54
Norway	ND	8.37	40	64.0	54
Russia	13.12	8.37	40	65.6	54
Spain	13.12	8.37	40	61.5	54
Sweden	ND	8.37	40	82.8	54
Turkey	13.12	8.37	40	61.5	54
UK	ND	8.37	40	61.5	54

ND = Not defined

Table A.3: Permissible Maximum Weights for Trucks in Europe (International Transport Forum 2013)

Country	Lorry 2 axles (lb)	Lorry 3 axles (lb)	Road Train 4 axles (lb)	Road Train 5+ axles (lb)	Articulated Vehicle 5+ axles (lb)
Belgium	41,887	57,320	85,980	97,000	97,000
Denmark	39,683	57,320	83,775	92,600/ 119,050(6)	92,600/ 119,050(6)
Finland	39,683	57,320	79,366	97,000/ 132,280(7)	92,600/ 105,822
France	41,887	57,320	83,775	88,185/ 97,000(8)	88,185/ 97,000(8)
Germany	39,683	57,320	79,366	88,185	88,185
Ireland	39,683	57,320	79,366	97,000	97,000
Italy	39,683	57,320	88,185	97,000	97,000
Netherlands	47,400	47,400/ 67,241(1)	88,185	110,231	110,231
Norway	41,887	57,320	85,980	101,412/ 123,460(9)	94,800/ 110,231(11)
Russia	39,683	55,115/ 61,729(2)	79,366/ 70,547(5)	88,185	88,185
Spain	39,683	55,115/ 57,320(3)	79,366	88,185	97,000(12)/ 92,600(13)
Sweden	39,683	57,320	83,775	105,822/ 132,277(10)	105,822/ 132,277(14)
Turkey	39,683	55,115/ 57,320(4)	83,775	88,185	88,185/ 97,000(15)
UK	39,683	57,320	83,775	88,185	88,185/ 97,000(15)

(1) Depending on the distance between the axles, number of driven axles, type of suspension, and single- or double-mounted tires
(2) Only for 3-axle road train
(3) When the driving axle is fitted and has pneumatic suspension or recognized as equivalent to EU level
(4) With the conditions laid down in regulation for type approval
(5) If 4-axle, single-unit lorry
(6) 6-axle = 105,822 lb, 7-axle = 119,050 lb
(7) 5-axle = 97,000 lb, 6-axle = 123,460 lb, 7-axle = 132,280 lb
(8) 97,000 lb is applicable under special conditions concerning axle weight, tridem weight, and suspension type (cf. http://www.developpement-durable.gouv.fr/-Reglementation,1432-.html)
(9) 5-axle (2 + 3) = 103,617 lb, 5-axle (3 + 2) = 101,412 lb, 6-axle = 110,231 lb, 7-axle = 123,460 lb
(10) 5-axle = 105,822 lb, 6-axle = 127,868 lb, 7-axle =132,277 lb
(11) 5-axle = 94,800 lb, 6-axle = 110,231 lb
(12) 3-axle motor vehicle with 2- or 3-axle semi-trailer carrying a 40 ft ISO container as a combined transport operation
(13) 2-axle motor vehicle with 3-axle semi-trailer carrying a 40 ft ISO container as a combined transport operation
(14) 5-axle = 105,822 lb, 6-axle = 127,868 lb, 7-axle = 132,277 lb
(15) For vehicles engaged in combined transport

REFERENCES FOR APPENDIX A

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APPENDIX B: BASIC FEE CATEGORIES AND OVERWEIGHT PERMIT REGULATION STUDIES

Permit fees to recover damage costs from overweight trucks can be classified into five basic categories: flat, weight based, distance based, axle based, or a combination. This appendix introduces the different categories and their implementations in the states.

B.1 FLAT FEE

A flat user fee is the simplest method for permitting offices and the trucking industry to manage and administer. This fee method has been used by multiple states, which issue single-use permits with charges ranging between $5 and $135, with a median of $25, as shown in Figure B.1 (J.J. Keller and Associates 2011). In most states that issue single-use permits, truckers must perform the trip within a specified period of time, usually in 3 to 5 days. Permits can be issued on a daily, monthly, or annual basis to reduce corresponding application and processing time as well as the fee charge. This fee structure, however, does not account for truck weight and travel distance (Chowdhury et al. 2013).

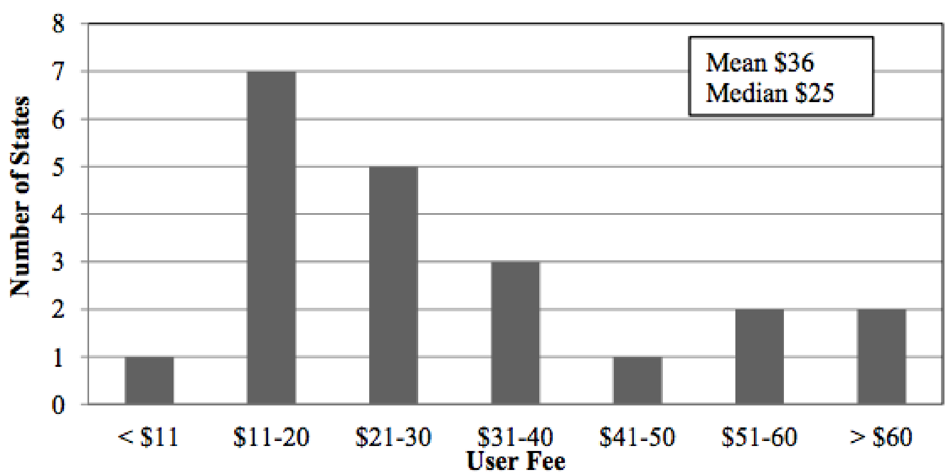


Figure B.1: States issuing single-trip permits with flat user fee (J.J. Keller and Associates 2011).

Flat fees have been used to cover the administrative costs of issuing permits with a contribution to highway maintenance. The permits have allowed state DOTs to track the extent of overweight shipping on roadways, which is important for estimating acceleration in deterioration of the transportation infrastructure. This facilitates better maintenance scheduling and inventory tracking.

B.2 WEIGHT-BASED FEE

Gross vehicle weight (GVW) is related to the impact of truckloads on the deterioration of pavements and bridges. While the legal GVW is limited to 80,000 lb, states have been allowing heavier loads on their network to encourage economic prosperity (Chowdhury et al. 2013). States administering single-trip weight-based permits in 2011 charged $0.1 to $20 per ton of excess load, as shown in Figure B.2.

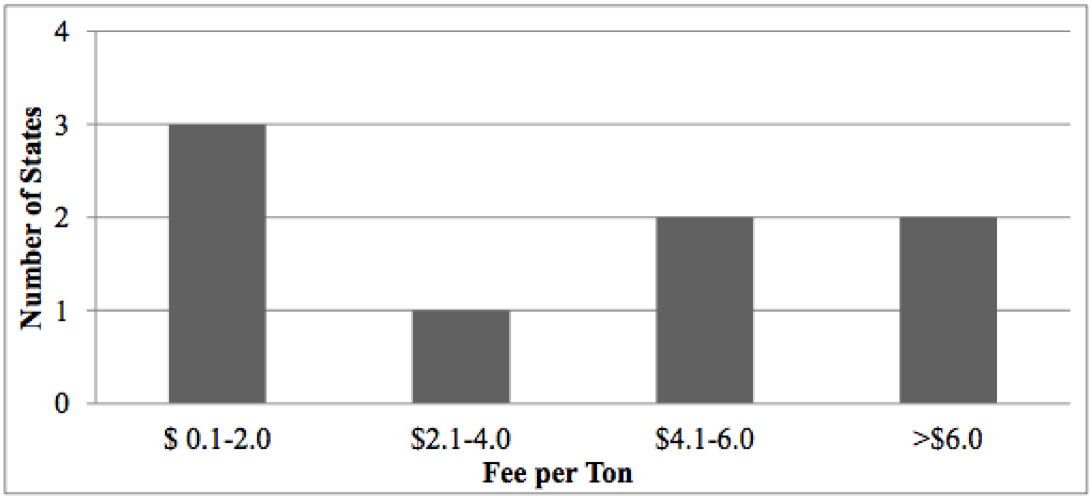


Figure B.2: States issuing single-trip permits with a weight-based user fee (J.J. Keller and Associates 2011).

The most commonly permitted weights in the United States for a 5-axle semi-trailer range from 100,000 to 110,000 lb, with a maximum of 132,000 lb.

B.3 DISTANCE-BASED FEE

While weight permits account for the vehicular loading applied to the infrastructure, they do not account for the extent of exposure. Charging for distance allows consideration of the length of the roadway and overweight vehicle impacts (Chowdhury et al. 2013).

B.4 AXLE-BASED FEE

Besides the maximum allowable GVW, some states consider the number of axles, or, implicitly, the weight per axle, as a threshold to characterize overweight vehicles.

Illinois established a relatively comprehensive overweight permit fee system that takes GVW, axle weight, and distance into account (Chowdhury et al. 2013).

B.5 OVERWEIGHT PERMIT FEE REGULATION STUDIES

Whitford and Moffett (1995) examined the existing annual permit system in Indiana. The study investigated permits for Michigan truck trains, which allow non-conforming vehicles to use the “extra-heavy-duty highways.” The researchers also explored the feasibility of using annual permits. A serious loss of revenue caused by Michigan truck train annual permits was noticed.

Hajek et al. (1998) developed a methodology to evaluate the changes of pavement cost resulting from regulation changes related to truck weights and dimensions. The ESAL-based cost functions, which determined less than 6% discount rates under a 60-year analysis period for new pavements and in-service pavements indicated that life-cycle pavement costs were in an exponential relationship with truck volumes. Marginal cost functions were obtained by deducting one year's ESAL cost function from its continuous year's ESAL cost function. An exponential increase in marginal costs for low-volume roads was noted. After quantifying the damage costs of proportionally rearranged traffic streams in Ontario, it was found that highway type is the main factor accounting for marginal pavement damage costs.

Hewitt et al. (1999) explored a procedure to quantify pavement damage and economic impacts of regulation changes related to truck weights in Montana. Instead of using an existing vehicle fleet, a new traffic stream was created based on several sources of information from the Montana Motor Carriers Association to estimate the changes in truck traffic volume and VMT for the Montana highway network. The AASHTO design method and equivalent uniform annual cost (EUAC) were used to calculate pavement performance and costs, and the ESAL cost changes were plotted in term of percentages. The transportation costs of the 12 selected commercial industries were evaluated under the assumption of hauling the same amount of freight. A major purpose of the Hewitt et al. study was to run input–output (I–O) models of the Montana economy, in which infrastructure and productivity costs were inputs and gross state product was the output. The system developed by Regional Economic Modeling Inc. could be applied to all regions in the United States to determine broad economic trends. However, because the result was based on selected commercial industries, it was not applicable to general cases.

Roberts and Djakfar (1999) conducted a preliminary assessment of impacts of increasing the GVW from the existing legal limit to 100 kips on trucks hauling sugarcane, rice, timber, and cotton. Issued by the Louisiana Department of Transportation and Development, the agronomic/ horticultural permit, the cotton module permit, and the harvest season or natural forest products permit were included in the study. As shown in the flowchart (Figure B.3), it was found that researchers spent considerable time communicating with the related departments to determine roadways for each commodity because additional pavement damage depends on trip length, which in turn is determined by locations of each crop and their transportation routes. The truck weight scenario and payload per truck would affect pavement costs. Researchers found that smaller impacts resulted from increasing vehicle's GVW if the pavement was designed to haul a larger number of ESALs.

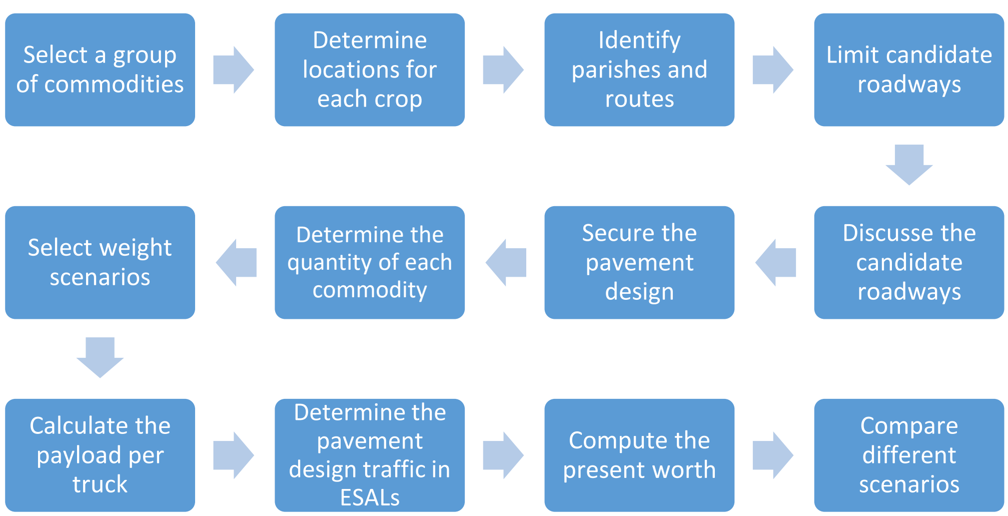


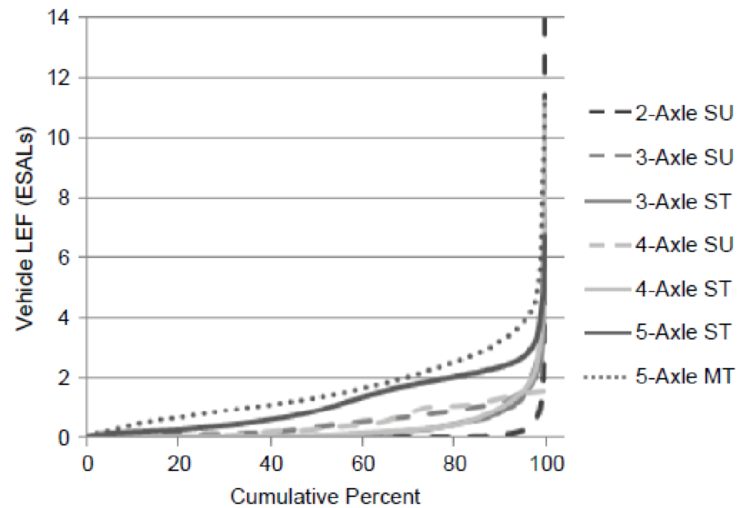
Figure B.3: Research flowchart used by Roberts and Djakfar (1999).

Luskin et al. (2002) discussed the economic inefficiency of the fee structure for HB 2060 permits, which are the annual divisible-load permits legislated by the Texas government in 1989 for trucks to operate above the general weight limits. Owing to the lack of detailed data on travel information, extreme-case scenarios for pavement damage (i.e., the worst-case scenarios and the best-case scenarios) were confined in the analysis. A truck traveling only on relatively light-duty roads was the subject in the worst-case scenario, while a truck traveling only on the relatively heavy-duty roads was the subject in the best-case scenario. Five-axle truck/trailer combinations were the predominant configuration among the trucks with the HB 2060 permit. The researchers selected a typical road for each traveling type and endeavored to maximize the difference of GVW with and without the permit for the worst-case scenario while minimizing the difference for the best-case scenario.

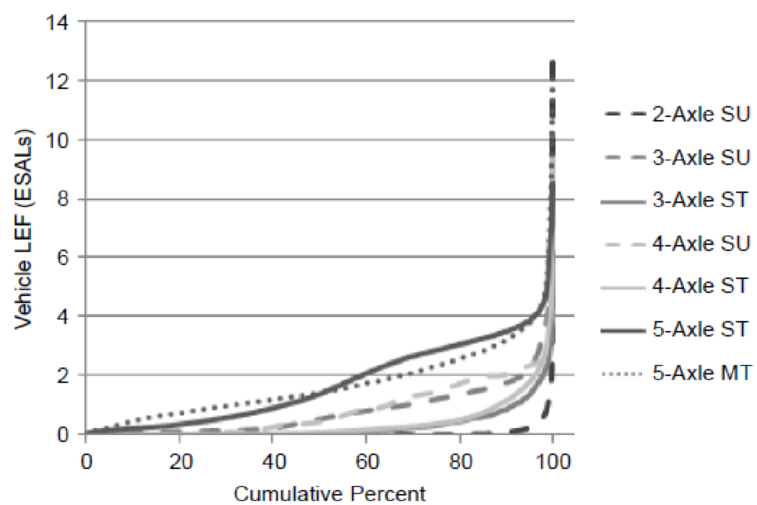
Fekpe et al. (2006) provided a conceptual framework for a federally supervised (but state-administered) performance-based oversize and overweight permitting program. Performance standards and a framework, which were the two essential elements of a performance-based system suitable for application in the United States, were emphasized in the study. The researchers identified the three major building blocks—administration, enforcement, and evaluation—and defined the components of each building block for the performance-based program. The evaluation block played a unique role in the framework because the performance could be continuously monitored and the results could be used for revising performance measures. To improve highway safety, an overall assessment of the system performance was provided based on the feedback from the evaluation and enforcement systems. However, periodic reassessments of permitted vehicles were needed to make the enforcement system more comprehensive.

Fortowsky and Humphreys (2006) developed two methodologies to estimate freight changes and pavement impacts from freight truck diversion caused by changes in truck weight limits on interstate highways. The purpose of the first methodology was to evaluate the changes in truck freight if I-95 was open to heavy trucks. The current case and exemption case, where truck traffic was rerouted on interstate highways, were assumed in the study. The subtraction of the current cost from the exemption case cost was the total safety, pavement, and bridge cost difference. To estimate truck VMTs and ESALs, a representative ratio of 5-axle and 6-axle trucks was developed to convert freight tonnage. Through multiplying ESAL factors by truck VMT for each truck type, ESAL miles were calculated for each pavement segment. Because trucks were rerouted on interstate highways, a mapping and traffic network analysis using the TransCAD model was an important step in the study. The purpose of the second methodology was to calculate road cost per ESAL by road type. The historical annual costs of pavement maintenance for each functional system were provided by the Maine Department of Transportation. Dividing expenditures by ESALs calculated with the first methodology, the researchers obtained road cost per ESAL. It was found that pavement cost savings were underestimated if state GVW limits were allowed on all of I-95.

Conway and Walton (2008) developed a methodology for testing whether the existing class-based toll structures were "fair" between various commercial truck classes and for optimizing toll rates in each truck class in order to fully recover the total cost of pavement consumed by trucks. They found that because of the disproportionality of the cost-recovery optimization strategy, a majority of vehicles would overpay for their actual pavement cost. The calculation of toll rates was supposed to be based on actual load-equivalency factor (LEF) values—instead of vehicle classes—because "fairness" within vehicle classes could not be achieved by the calculation of toll rates on the basis of optimal LEFs. Direction, bridge impacts, and space consumption were identified as additional factors that should be taken into consideration to improve the methodology. In addition, the study results indicated that real weight data were useless for testing and improving toll equity. Figure B.4 shows the approximated cumulative distribution function for southbound trucks.



(a)



(b)

Figure B.4: Approximated cumulative distribution function for southbound trucks: (a) flexible and (b) rigid (Conway and Walton 2008).

Bilal et al. (2010) gathered and summarized detailed permitting fee structures in Indiana and its seven neighboring states. Multiple-trip permit expenditures and single-trip permit expenditures summed over 1 year were evaluated. A company with 200 trucks of various dimensions was assumed for the evaluation. Additional steps in the research were to (1) calculate the total annual permit fee to be paid by another hypothetical company on the basis of the current single-trip fee structure and (2) determine how much a hypothetical trucker should pay in a given year on the basis of damage done to the pavement. Researchers assumed that a hypothetical truck with a GVW of 134,000 lb would travel 10,000 miles per year. To find a relationship between weight per axle and unit pavement cost, the cost data in Table B.1 were used to plot nomographs and establish models for urban interstate and rural interstate highways.

Table B.1: Weight per Axle vs. Unit Pavement Cost, Urban and Rural Interstates (Bilal et al. 2010)

Number of Axles	GVW (lbs)	Weight Per Axle (lbs)	Unit Pavement Cost ($/1000 Miles)	
			Rural	Urban
(a)	*(b)*	*(b)÷(a)*		
4	40,000	10000	10.0	31.0
4	60,000	15000	56.0	181.0
5	60,000	12000	33.0	105.0
5	80,000	16000	127.0	409.0

Bilal et al. (2010) also explored the relationship between unit pavement cost and weight per axle (Figure B.5). Using the formula, the researchers estimated the unit pavement cost per 1,000 mi for each set of axle numbers from 6 to 10 and for urban and rural interstate highways. Then the pavement cost could be computed for different mileages. From the processed data, pavement cost nomographs (Figures B.5 through B.7), which reflected the relationship between mileage and unit pavement cost and the relationship between mileage and the number of axles, were plotted. Finally, the researchers arrived at three conclusions: (1) increasing the number of axles led to a decrease in load per axle; (2) if mileage increased, pavement cost for a given number of axles increased linearly; and (3) increasing the number of axles for a given traveled distance drastically decreased the pavement damage cost.

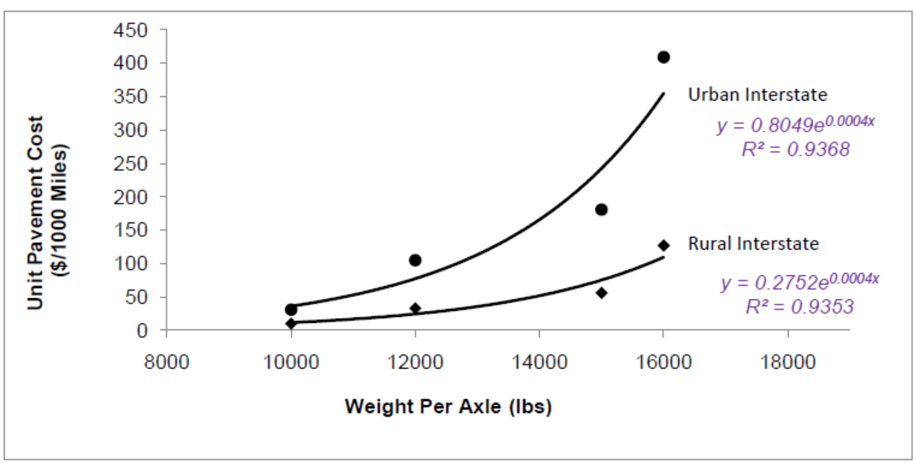


Figure B.5: Weights per axle vs. unit pavement cost, urban and rural interstates for 134,000 lb GVW truck (Bilal et al. 2010).

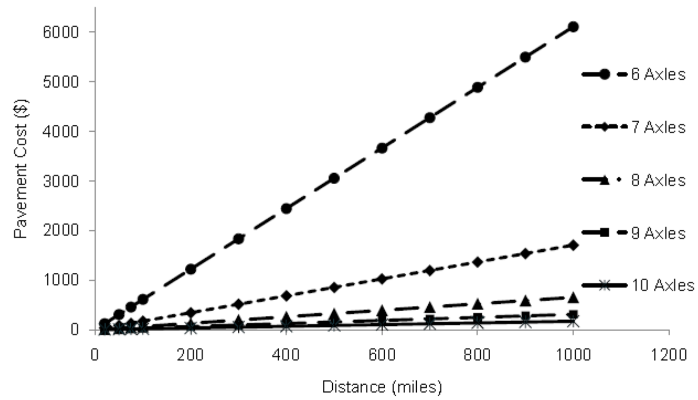


Figure B.6: Unit pavement cost vs. truck miles traveled for urban interstate highways (Bilal et al. 2010).

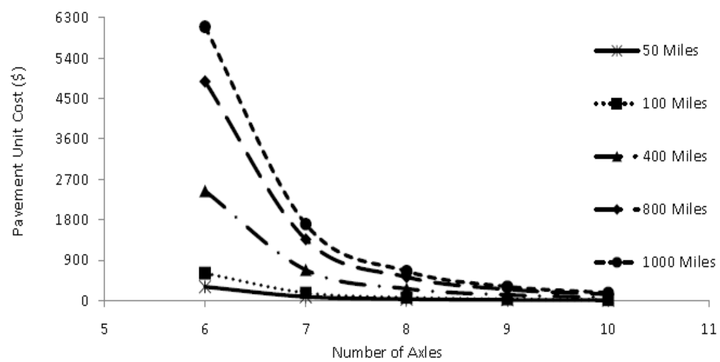


Figure B.7: Unit pavement cost vs. number of axles for urban interstate highways (Bilal et al. 2010).

Bereni (2012) discussed and compared heavy-vehicle policies in New Zealand, Switzerland, Austria, Germany, the Czech Republic, the Slovak Republic, and France. A heavy-vehicle fee (HVF) was supposed to be paid by all heavy trucks hauling 3.5 tons or more. In Germany, the weight limit for heavy vehicles was increased to 12 tons. The researchers analyzed and compared the procedures for how HVFs were implemented in each country. Switzerland, Germany, and Austria conducted extensive investigations of road cost bases and of the method used to allocate the road cost to heavy and light vehicles. Type of road, distance, number of axles, and emission classes were the four contributing factors used to determine the amount of the fee. In Austria, the HVF was in the range of AUD0.20 and 0.40 per km for a 5-axle truck (2-axle prime mover, 3-axle trailer, GVM of 40 tons), but Switzerland charged AUD0.92 per km.

Dey et al. (2013) used a multi-objective analysis approach to simultaneously accommodate overweight freight truck mobility and to select an optimal permit fee. The objectives in the bi-objective model were minimization of unpaid damage associated with overweight trucks and minimization of the overweight damage fee. A series of parameters and formulas was used to develop the relationship between the two objectives. The flat damage fee, the axle-based damage fee, the weight-based damage fee, and the weight- and distance-based damage fee were the four fee structures considered in the analysis. Overweight freight trip demand elasticities of –0.5, –1.0, and –1.5 were assumed in order to evaluate the sensitivity of overweight demand to the permit fee. The ESALs were estimated based on the assumption of a standard flexible pavement section with a structural number of 5 and terminal serviceability index of 2.5. Minimum damage cost to all vehicles, additional damage cost related to all truck traffic, and additional damage cost related to overweight trucks only were the three scenarios used to estimate pavement costs. The elasticity value reflected the sensitivity of overweight demand to the permit fee. Figures B.8 and B.9 show the weighted axle-based damage fee calculated using the proposed multi-objective analysis approach.

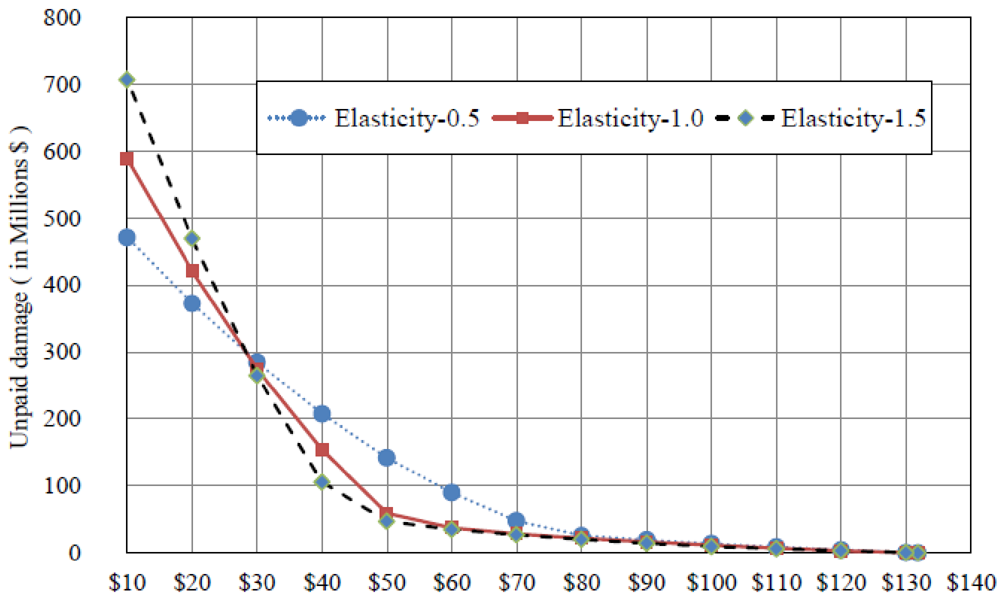


Figure B.8: Weighted axle-based damage fee (Dey et al. 2013).

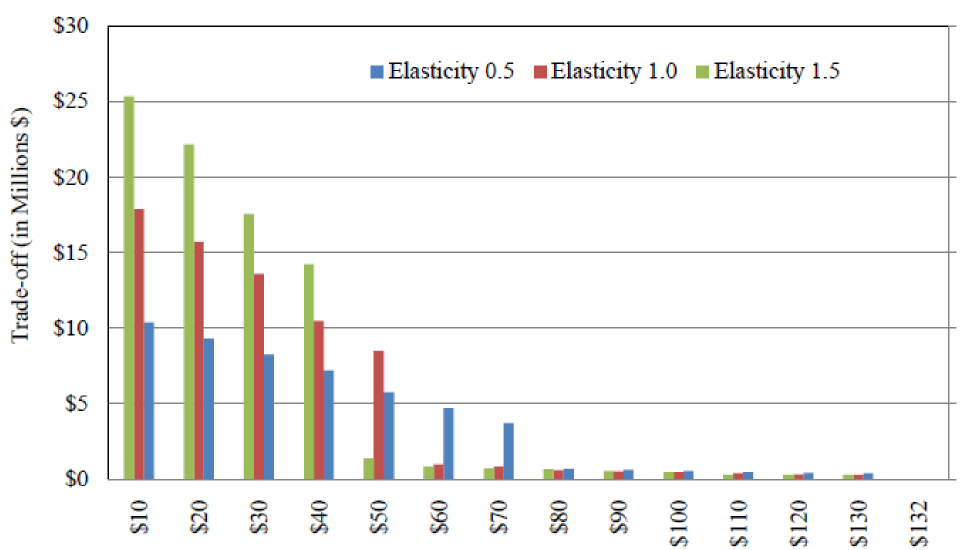


Figure B.9: Weighted axle-based damage fee (Dey et al. 2013).

Chowdhury et al. (2013) performed a study to estimate pavement deterioration caused by overweight trucks and the adequacy of standard permitting practices in state agencies. The additional pavement damage costs that were related to overweight trucks consisted of two primary factors: truck freight on South Carolina highways and the sum of ESALs produced by each individual truck. Researchers obtained AADTT estimates for roads of different functional classes in South Carolina through the TRANSEARCH database and studied the truck type distribution (from two axles to 8 axles) through WIM data from the St. George weigh station on I-95. The average trip length of each truck class was estimated on the basis of annual mileage reported in the 2002 South Carolina Economic Census data. The trucks were assumed to operate five days per week and travel once a day. An analysis was performed for flexible pavements in three different GVW groups: 80% of the South Carolina Department of Transportation (SCDOT) legal weight limits, SCDOT maximum weight limits, and maximum considered truck weight. Three traffic scenarios were created: no trucks in traffic (minimum design scenario); traffic that includes trucks with none having weights that exceed legal weight limits; and traffic that includes trucks, of which 8.3% were overweight. The researchers created truck configurations and calculated the ESAL factors for each individual truck. Then the replacement costs for each pavement design scenario were estimated to calculate permit fees. Four basic types (flat fees, weight-based fees, distance-based fees, and axle-based fees) and two types of overweight permit fee (annual permit and combined weight and distance) were included in the study.

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APPENDIX C: COMPARISON OF PRACTICES IN ILLINOIS AND NEIGHBORING STATES

Table C.1 presents a comparison of the different permitting practices in Illinois and its neighboring states. Most states have adopted a fee structure that combines a flat fee with another fee method, such as distance-based or GVW-based.

Table C.1: Comparison of Different Permitting Practices in Illinois and Its Neighboring States (Bilal et al. 2010)

State	Fee	Remarks
IL	**OS Only** $30 for 0-90 mi, $50 for 181-270 mi, $60 over 270 mi, for 100 to 120 ft long, 14 to 18 ft wide, 15 to 16 ft high **OS Only** $50 for 0-90 mi, $100 for 181-270 mi, $125 over 270 mi, for over 120 ft long, over 18 ft wide, over 16 ft high **OS/OW Less Than 12 Ft Wide** 6-axle 100,000 lb = $55 for 181-225 mi, $115 for 451-495 mi 6-axle 120,000 lb = $130 for 181-225 mi, $280 for 451-495 mi 5-axle 100,000 lb = $130 for 181-225 mi, $280 for 451-495 mi	Both flat fee and distance based. Separate fee structure for OS only and OS/OW Add $40 district fee plus $1 online transmission fee
IN	OS = $20 up to 95 ft in length, 12 ft 4 in in width and legal height OS = $30 up to 96 to 110 ft length, 12 ft 5 in to 16 ft wide, 13 ft 7 in to 15 ft height OS = $40, over 110 ft length, 16 ft wide, 15 ft tall and 80,000 lb OW = $20 + $0.35 per mile (up to 108,000 lb) OW = $20 + $0.60 per mile (108,001 to 150,000 lb) OW = $20 + $1.00 per mile (over 150,000 lb) OS/OW = Greater of the OS or OW calculated above	Both flat fee and distance based. Separate fee structure for OS only, OW only, OS/OW and superloads over 120,000 lb charged $10 executive fee
IA	$10	Flat fee
MO	OS Only = $15 OS/OW = $15 plus $20 per each 10,000 lb in excess of legal gross weight OS = $15 plus $250 movement feasibility fee for greater than 16 ft wide, greater than 16 ft high, or 150 ft long OW = $15 plus $20 for greater than 160,000 lb per each 10,000 lb in excess of legal gross weight. Additional bridge and roadway analysis fee of $425 for each vehicle that moves 0-50 miles, $625 for 51 to 200 miles, and $925 for over 200 miles	Separate fee structure for OS only, OW only, and OS/OW
OH	OS: 1-way = $65, 2-way = $100 OS/OW: 1-way = $135, 2-way = $200 Steel Coil: 1-way = $65, 2-way = N/A Supersize: 1-way = $135+TM, 2-way = $200+TM for 14 ft wide and greater than 14 ft 6 in height Superload OS/OW: 1-way = $135+TM, 2-way = $200+TM for greater than 120,000 lb gross vehicle weight, 14 ft wide, and 14 ft 6 in height TM = Ton-Mile = [(GVW-120,000)/2000 X $0.04	Separate flat fee structure for OS only, OS/OW, supersize, and superloads
WI	OS = $15 for greater than 40 ft for single vehicle and 65 ft for combination vehicles OS = $20 for greater than 102 in wide and/or greater than 162 in in height	Add $10 district fee, $10 bridge fee, $1 only permit order fee, and $10 pavement damage fee for vehicle greater than 16 ft wide and/or greater than 270 kips gross vehicle weight for vehicles greater than 16 ft wide and/or greater than 270 kips GVW
MI	OS Only = $15 OS/OW = $50	Separate flat fees for OS only and OS/OW
KY	$60	Flat fee

C.1 ILLINOIS TOLLWAY OVERWEIGHT AND OVERSIZE LIMITATIONS AND FEES

All vehicles using the tollway network are required to obtain an IDOT permit (illinoistollway.com 2015).

C.1.1 Legal Weight Limits

Single axle: not more than 20,000 lb

Tandem axle:

2-axle tandems (axle spacing 4 to 8 ft): not more than 34,000 lb

3-axle tandems (outside axles 8 to 10 ft): not more than 42,000 lb

Table C.2: Illinois Tollway's Legal Gross Vehicle Weight Limits (illinoistollway.com 2015)

Number of Axles	Maximum Weight	Minimum Outside Axle Spacing
2	36,000 lb	10 ft
3	60,000 lb	32 ft
4	80,000 lb	57 ft
5	80,000 lb	51 ft
6	80,000 lb	43 ft

C.1.2 Overweight Limits

Single axle: between 20,000 and 25,000 lb

Tandem axle:

2-axle tandems (axle spacing 4 to 8 ft): between 34,000 and 48,000 lb

3-axle tandems (outside axles 8 to 10 ft): between 42,000 and 60,000 lb

Table C.3: Illinois Tollway's Overweight Gross Vehicle Weight Limits (illinoistollway.com 2015)

Number of Axles	Maximum Weight	Minimum Outside Axle Spacing
2	36,001 to 48,000 lb	10 ft
3	68,000 lb	14 ft
4	76,000 lb	36 ft
5	80,001 to 100,000 lb	40 ft
6	80,001 to 120,000 lb	44 ft

Table C.4: Illinois Tollway's Fees for Overweight Vehicles (illinoistollway.com 2015)

Fees	$35	$150
2-axle tandems (4 to 8 ft)	34,001 to 44,200 lb	44,201 to 48,000 lb
3-axle tandems (8 ft)	42,001 to 54,600 lb	54,601 to 60,000 lb
3-axle tandems (9 ft)	42,501 to 55,250 lb	55,251 to 60,000 lb
3-axle tandems (10 ft)	43,501 to 56,550 lb	56,551 to 60,000 lb
5 or more axles	80,001 to 104,000 lb	104,001 to 120,000 lb

Table C.5: Illinois Tollway's Fees for Oversize Vehicles (illinoistollway.com 2015)

Criteria	Dimension	Permit Fee
Over width	More than 8 ft 6 in to 12 ft	$15
Over length	Vehicles exceeding 100 ft or trailers exceeding 80 ft	$15
Over height	More than 13 ft 6 in to 14 ft 6 in	$15

C.1.2 Superload Weight and Size Limits

Vehicles are considered superloads if:

- Any single axle exceeds 25,000 lb
- The GVW is greater than 120,000 lb
- Any tractor tandem is greater than 48,000 lb
- Any trailer tandem is greater than 60,000 lb
- Any trailer has two or more tandems
- The width is greater than 12 ft
- The length is greater than 145 ft
- The height is greater than 14 ft 6 in

Superload trucks are charged a flat fee of $150.

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APPENDIX D: PAVEMENTS

D.1 PAVEMENT ANALYSIS METHOD

D.1.1 Empirical Pavement Design Method

Empirical pavement design methods were developed based on data collected from field experience and experiments. On the basis of known pavement distresses under specific traffic loadings, the relationship between the observed pavement performance and pavement design can be determined. By taking the effects of drainage and environmental factors into consideration and including rehabilitation design, the AASHTO 1993 Guide enhanced the pavement design method for flexible and rigid pavement.

D.1.2 Mechanistic–Empirical Pavement Design Guide

The mechanistic–empirical (M–E) approach is a recently developed compound method for designing new construction and rehabilitation pavement structures. In the Mechanistic–Empirical Pavement Design Guide (MEPDG), mechanistic theory is used to predict pavement deterioration, and empirical test results are applied to establish an appropriate correlation between pavement damage and pavement performance or service life.

A hierarchical approach to the design inputs is applied to MEPDG. There are three levels of design input. Laboratory-measured material properties and project-specific traffic data are required for Level 1 input. Level 2 input is obtained through empirical correlations with other parameters. Compared with Level 1 and Level 2 input, Level 3 input supplies the lowest accurate analysis results because default values with minimal material testing and data collection are used in the AASHTOWare software program.

MEPDG input comprises traffic, material property, pavement structure, and climate input. Traffic, material property, and pavement structures can be designated by users. In AASHTOWare software, there is a library of weather data from about 800 weather stations all over the United States. Once a climate site is chosen, environmental conditions are automatically input. The Enhanced Integrated Climatic Model (EICM) is used to predict environmental conditions. EICM is a mechanistic model that reflects daily and seasonal variations of temperature and moisture in pavement structures caused by environmental factors at the project site.

D.2 PAVEMENT DETERIORATION MODEL

D.2.1 Empirical Models

D.2.1.1 AASHTO Design Method

A load-equivalency factor (LEF) is defined as the ratio between pavement damage caused by one single pass of the axle under consideration and the pavement damage caused by one single pass of a standard 18 kip single-axle load with dual tires (one ESAL). The calculated LEFs can be used to determine the equivalent number of ESALs for each specific axle that will provide the basis for allocation of pavement damage cost:

$$LEF = \frac{1/N}{1/N_{ESAL}} = \frac{N_{ESAL}}{N}$$

where

LEF = load-equivalency factor

N_{ESAL} = allowable number of load repetitions to failure under loading of a standard 18 kip single-axle load with dual tires

N = allowable number of load repetitions to failure under the loading of an axle with different load magnitudes and configurations

The pavement performance concept of the Present Serviceability Index (PSI) was developed during the AASHO Road Test in the 1960s. PSI is a combination of values defined by the following equation:

$$PSI = 5.03 - 1.91\log(1 + SV) - 0.01(C + P)^{0.5} - 1.38RD^2$$

where

SV = longitudinal cracking in the wheel path

C = cracked area

P = patched area

RD = average rut depth for both wheel paths

The modified AASHTO Guide design equation from Huang (2004) for flexible pavements is:

$$\log\left(\frac{W_{tx}}{W_{t18}}\right) = 4.79\log(18+1) - 4.79\log(L_x + L_2) + 4.33\log L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}}$$

$$G_t = \log\left(\frac{4.2 - p_t}{4.2 - 1.5}\right),\ \beta_x = 0.40 + \frac{0.081(L_x + L_2)^{3.23}}{(SN+1)^{5.19} L_2^{3.23}}$$

where

W_{tx} = number of applications of a given axle

W_{t18} = number of standard axle passes (single 18 kip axle)

L_x = load (in kips) of axle group

L_2 = axle code (1 for single axle, 2 for tandem axles, 3 for tridem axles, and 4 for quad axles)

β_{18} = value of β_x when L_x = 18 and L_2 = 1

p_t = terminal serviceability

SN = structural number, SN = $a_1D_1 + a_2D_2m_2 + a_3D_3m_3$

a_1, a_2, a_3 = layer coefficients for the surface, base, and subbase

D_1, D_2, D_3 = thickness of the surface, base, and subbase

m_2, m_3 = drainage coefficients for the surface, base, and subbase course

The data from the Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) in-service pavement test sections (244 flexible, 120 rigid) were used to evaluate AASHTO design equations (SHRP 1994). The general form of the developed formula is:

$$Z = N^{D} 10^{E}$$

where

- Z = total change in distress during a specified period of time
- N = cumulative ESALs in 1000s
- $D = d_0 + d_1X_1 + d_2X_2 + \ldots + d_nX_n$
- $E = e_0 + e_1X_1 + e_2X_2 + \ldots + e_nX_n$
- $X_1, X_2 \ldots X_n$ = pavement design, construction standard, and climate-related parameters

The equations above can be transferred to predict the required thickness of layers:

$$X_T = \frac{\left(\log_{10}\left(\frac{Z}{N^D}\right)\right) - E_X}{E_T}$$

where

- X_T = thickness of base or hot-mix asphalt concrete
- $E_X = e_0 + e_1X_1 + e_2X_2 + \ldots + e_nX_n - e_TX_T$
- E_T = coefficient of the term C_iX_i that includes the layer thickness of interest X_T

D.2.1.2 Other Empirical Models

George (2000) investigated power regressions for weighted alligator cracking of medium and high severity (WCAMH), rutting, International Roughness Index (IRI), and pavement condition rating (PCR) of original flexible pavement, using data from the Pavement Management System (PMS).

$$\text{WCAMH} = \text{A}_0 AGE^{A_1} CESAL^{A_2} MSN^{A_3}$$

$$Rutting = A_4 AGE_{A5} CESAL_{A5}$$

$$IRI = (A_7 + AGE^{A_8}(1 + CESAL^{A_9}))MSN^{A_{10}}$$

$$PCR = (A_{11} - AGE^{A_{12}}(1 + CESAL^{A_{13}}))MSN^{A_{14}}$$

where

- $A_0, A_1, A_2 \ldots A_{14}$ = coefficients
- AGE = pavement age
- CESAL = cumulative 18 kip ESAL applied to the pavement (in the heavily trafficked lane), in millions
- MSN = modified structural number

To study the relationship between pavement deterioration (fatigue cracking and rutting) and truck repetitions, Chatti et al. (2004) used data from the General Pavement Study (GPS-1) in the LTTP to conduct simple linear regression analysis. The standardized slope value was used to determine the importance of differing independent variables (truck configurations and axle configurations).

Salama et al. (2006) selected pavement segments in Michigan with similar pavement age, type, cross-sectional design, and traffic loading, and the performance data of the Distress Index (DI), rutting, and Ride Quality Index (RQI) of the slow lane, as surveyed by the Michigan Department of Transportation. Axle configuration, truck configuration, and FHWA truck class were the three independent variables considered in the simple, multiple, and stepwise linear regressions. The simple linear univariate regression was applied to study the impacts of axle or truck configuration on pavement distress. The multiple linear regression involved all the specific variables. On the basis of the regression parameter (β), coefficient of determination (R^2), and test statistic (ρ-value), the effects of different axle and truck configurations were compared. The step regression was used to examine the results of simple and multiple linear regressions.

Prozzi and Madanat (2002) developed a joint pavement performance model by combining experiment data from the AASHO Road Test dataset and field data collected from the Minnesota Road Research Project (MnRoad). The specification for the roughness is expressed by:

$$r_{it} = \theta_1 e^{\theta_2 H_{1i}} + \sum_{l=1}^{t} \theta_3 (1 + EAT_i)^{\theta_9} e^{\theta_{10} G_l} N_{i,l-1}^{\theta_{11}} \Delta N_{i,l}$$

where

r_{it} = roughness (in m/km, IRI)

EAT = equivalent asphalt thickness (relative to the asphalt concrete used during the AASHO test)

H_j = layer thickness

G = frost gradient

θ_j = parameters to be estimated

Li and Sinha (2000) created the concept of the Modified Damage Index (MDI) and developed load and non-load factor functions using the data on the relative change in IRI in one life cycle, regional and climatic features, subgrade materials, cumulative traffic loading in one life cycle (EASLs), design and construction standards, and pavement age. The general form of MDI is:

$$MDI_t = \beta_0 + \sum_{j=1}^{L} \beta_j X_j + \varepsilon_2$$

9

where

MDI_t = base 10 logarithm of modified damage index in year t

X_j = set of parameters representing climate, subgrade materials, annual ESALs, pavement age, design, and construction features

β = model coefficient

ε_2 = error term

D.2.2 Mechanistic–Empirical (M–E) Based Models

D.2.2.1 Mechanistic–Empirical Pavement Design Guide

Fatigue Cracking (ft/mi)

The equation used for fatigue cracking in MEPDG is:

$$N_f = 0.00432\, k_1' C(\varepsilon_t)^{-3.291}(E)^{-1.281}$$

where

N_f = number of repetitions to fatigue cracking

k_1' = correction parameter for different asphalt layer thickness effects

C = laboratory to field adjustment factor

ε_t = tensile strain at the critical location

E = stiffness of the material

The final equation used for top-down fatigue cracking in MEPDG is:

$$FC_{Top} = \left(\frac{1000}{1+e^{(7.0-3.5\log 10(D\cdot 10))}}\right)(10.56)$$

where

FC_{top} = top-down fatigue cracking (ft/mi)

D = top-down fatigue damage

Rutting in Asphalt Mixtures

The equation used for rutting in asphalt mixtures in MEPDG is:

$$\frac{\varepsilon_p}{\varepsilon_r} = k_1 10^{-3.3541} T^{1.5606} N^{0.4791}$$

where

ε_p = accumulated plastic strain at N repetitions of load (in/in)

ε_r = resilient strain of the asphalt material as a function of mix properties, temperature, and loading rate (in/in)

k_1 = function of total asphalt layer thicknesses (in) and depths (in) to computational point, to correct for the confining pressure at different depths

T = temperature (°F)

N = number of load repetitions

$$k_1 = (C_1 + C_2 \cdot depth) \times 0.328196^{\ depth}$$

$$C_1 = -0.1039\, h_{AC}^2 + 2.4868\, h_{AC} - 17.342$$

$$C_2 = 0.0172\, h_{AC}^2 - 1.7331\, h_{AC} + 27.428$$

where

depth = rut depth on asphalt layer (in)

h_{AC} = total thickness of the asphalt layers (in)

Rutting in Unbound Materials (in)

The equation used for rutting in unbound materials in MEPDG is:

$$\delta_a(N) = \beta_{s1}\left(\frac{\varepsilon_0}{\varepsilon_r}\right)e^{\left(\frac{\rho}{N}\right)^{\beta}}\varepsilon_v h$$

where

δ_a = permanent deformation for the layer/sublayer (in)

N = number of traffic repetitions

β_{CAL} = national calibration factor: 2.23 for granular layers; 1.35 for subgrades

ε_0 , β, and ρ = material properties

ε_r = resilient strain imposed in laboratory test to obtain the above-listed material properties, ε*0*, β, and ρ (in/in)

ε_v = average vertical resilient strain in the layer/sublayer as obtained for the primary response model (in/in)

h = thickness of the layer/sublayer (in)

Reflective Cracking (%)

The equation used for reflective cracking in MEPDG is:

$$RC = \frac{100}{1 + e^{(3.5+0.75(Heff))+dt(-0.688584-3.37302(Heff)^{-0.915469})}}$$

where

RC = present of cracks reflected (%)

t = time (year)

d = calibration parameter

D.2.2.2 Modified Texas M–E

Fatigue Cracking

The equation used for fatigue cracking in the Texas M–E Design Guide:

$$N_f = k_i N_i + k_p N_p$$

$$N_i = k_1 \left(\frac{1}{\varepsilon} \right)^{k_2}$$

$k_1 = 10^{6.97001 - 3.20145 k_2 - 0.83661 \log E}$, $k_2 = n$

$$N_p = \int_{c_0}^{h} \frac{1}{k_b A K_I^n + k_s A K_{II}^n} dc$$

where

- N_f = fatigue life
- N_i = crack initiation life
- N_p = crack propagation life
- k_i, k_p, k_b, k_s = calibration factors
- ε = maximum tensile strain at the bottom of asphalt layer
- E = dynamic modulus
- A, n = fracture properties, determined from OT testing
- K_I, K_{II} = stress intensity factors (SIF) caused by bending and shearing stresses

Rutting in Asphalt Mixtures

The VESYS layer-rutting model developed by FHWA in the 1970s is used in the Texas M-E Design Guide:

$$RD_{AC} = k_{AC} \int \Delta U \mu N^{-\alpha}$$

where

- RD_{AC} = rut depth in the AC layer
- k_{AC} = calibration factor
- ΔU = deflection difference between the AC layer top and layer bottom

N = load repetitions

α and μ = rutting properties of the AC layer, determined by lab test

Monthly accumulation of RD_{AC} is shown as follows:

$$RD_1 = k_{AC} \Delta U_1 \mu_1 \frac{N_1^{1-\alpha_1}}{1-\alpha_1}, \; N_{eq1} = \left(\frac{RD_1 (1-\alpha_2)}{\Delta U_2 \mu_2 k_{AC}} \right)^{1-\alpha_2}$$

$$RD_2 = k_{AC} \Delta U_2 \mu_2 \frac{(N_{eq1} + N_2)^{1-\alpha_2}}{1-\alpha_2}, \; N_{eq2} = \left(\frac{RD_2 (1-\alpha_3)}{\Delta U_3 \mu_3 k_{AC}} \right)^{1-\alpha_3}$$

$$RD_3 = k_{AC} \Delta U_3 \mu_3 \frac{(N_{eq2} + N_3)^{1-\alpha_3}}{1-\alpha_3}$$

where

RD_1, RD_2, and RD_3 = rut depths at the end of the first month, second month, and third month

N_1, N_2, and N_3 = monthly ESALs of the first month, second month, and third month

α1, μ1, α2, μ2, α3, and μ3 = rutting properties of the first month, second month, and third month

The equation used for calibration factor is:

$$k_{AC} = f_1(T) f_2(E) f_3(h_{AC})$$

where

$f_1(T)$ = pavement temperature correction factor

$f_2(E)$ = modulus correction factor to alternatively consider non-linear stress dependency of rutting development

$f_3(h_{AC})$ = asphalt layer thickness correction factor

$$f_1(T) = 0.191112 + \frac{3.643124}{1 + e^{18.3009 0.204437T}}$$

$$f_2(E) = 0.37264 + \frac{1.18771}{1 + e^{-8.90208 + 0.09879E}}$$

$$\begin{aligned} f_3(h_{AC}) = {} & (0.01445272\, h_1^3 - 0.12471319\, h_1^2 + 0.22193794\, h_1 + 1.37640722) \times \\ & (0.00567302\, h_2^3 + 0.07104301\, h_2^2 - 0.49592553\, h_2 + 2.12378879) \times \\ & \left(0.00199314 + \frac{0.54035153}{1 + e^{(2.61478586 + 0.584941481 \times (h_1 + h_2))}} \right) \end{aligned}$$

where

T = asphalt layer temperature (°F)

E = asphalt layer dynamic modulus value (ksi) at 54°C and 10 Hz

h_1 and h_2 = representative layer thicknesses

Rutting in Unbounded Materials

$$RD_{granular\_base} = k_{\text{granular\_base}} \int \Delta U \mu N^{-\alpha}$$

$$RD_{subgrade} = k_{\text{subgrade}} \int \Delta U \mu N^{-\alpha}$$

where

$Rd_{granular\_base}$ and $RD_{subgrade}$ = rut depth of granular base and subgrade

$k_{granular\_base}$ and $k_{subgrade}$ = calibration factors

$$k_{granular\_base} = 1.0871 + \frac{9.4879}{e^{06483 h_{AC}}}$$

$$k_{subgrade} = \frac{8.9494}{e^{(4.346+0.2304 h_{AC})}}$$

where

h_{AC} = the asphalt layer thickness (in)

D.2.2.3 Modified California M–E

Ullidtz et al. (2008) and Ullidtz et al. (2010) used CalME to predict reflection cracking and rutting in asphalt for new pavement and overlay design.

Reflection Cracking

The equation used to calculate the tensile strain at the bottom of the overlay is as follows, assuming a dual wheel on a single axle:

$$\varepsilon = \alpha \times E_{an}^{\beta_1} \times E_{bn}^{\beta_2} \times (a1 + b1 + In(LS_n)) \times \exp(b2 + H_{an}) \times (1 + b3 \times H_{un}) \times (1 + b_4 \times E_{un}) \times \sigma_n \; E_{an} = \frac{E_a}{E_s},$$

$$E_{bn} = \frac{E_b}{E_s},\; E_{un} = \frac{E_u}{E_s},\; \sigma_n = \frac{\sigma_o}{E_s},\; LS_n = \frac{LS}{a},\; H_{an} = \frac{H_a}{a},\; H_{un} = \frac{H_u}{a}$$

where

E_a and H_a = overlay modulus and thickness, respectively

E_u and H_u = underlayer modulus and thickness, respectively

E_b = modulus of the base/subbase

E_s = modulus of the subgrade

LS = crack spacing

σ_o = tire pressure

a = radius of the loaded area for one wheel, with constants as follows:

α = 342,650

β1 = −0.73722

β2 = −0.2645

β3 = −1.16472

a1 = 0.88432

b1 = 0.15272

b2 = −0.21632

b3 = −0.061

b4 = 0.018752

$$\log(E) = \delta + \frac{\alpha \times (1-\omega)}{1+\exp(\beta + \gamma \log(tr))}$$

where

δ, α, β, and γ = constants

tr = reduced time (sec)

ω = damage, calculated from

$$\omega = A \times (MN)^{\alpha} \times \left(\frac{\mu\varepsilon}{200\,\mu strain}\right)^{\beta} \times \left(\frac{E}{3000\,MPa}\right)^{\gamma} \exp(\delta t)$$

where

E = modulus

MN = number of load repetitions (millions) ($N/10^6$)

μ_{ref} = strain at the bottom of the asphalt layer

t = temperature (°C)

200 µstrain and 3,000 MPa = reference constants

A, α, β, γ, and δ = constants

The equation used for rutting in the asphalt layers in CalME is:

$$d_p = K \sum h_i \gamma_i^i$$

where

K = calibration factor determined from heavy-vehicle simulator (HVS) testing

h_i = thickness of layer i

γ_i^i = inelastic (permanent) shear strain in layer i, determined from

$$\gamma^i = \exp\left(A + \alpha \times \left[1 - \exp\left(\frac{-In(i)}{\gamma}\right) \times \left(\frac{1 + In(i)}{\gamma}\right)\right]\right) \times \exp\left(\frac{\beta \times \tau}{\tau_{ref}}\right) \times \gamma^e$$

where

γ^e = elastic (resilient) shear strain

τ = shear stress

N = number of load repetitions

τ_{ref} = reference shear stress (0.1 MPa)

A, α, β, and γ = constants determined from the repeated simple shear tests at constant height:

$$\gamma^e = \frac{\tau}{2 \times G_i} = \frac{\tau}{E_i / (1 + v_i)}$$

where

G_i = the shear modulus of layer i

E_i = the modulus of layer i

ν_i = Poisson's ratio for layer i

<u>Rutting of Asphalt and Unbound Layers</u>

The equation used for rutting of asphalt and unbound layers in CalME is:

$$dp = A \times (MN)^{\alpha} \times \left(\frac{\mu\varepsilon}{1000\ \mu strain}\right)^{\beta} \times \left(\frac{E}{40\,MPa}\right)^{\gamma}$$

D.2.2.4 Modified Illinois M–E

<u>Flexible Pavement</u>

The IDOT M–E pavement design procedure is established on the basis of resilient soil and material testing procedures, the ILLI-PAVE structural model, and a design algorithm from an extensive ILLI-PAVE database.

The asphalt fatigue algorithm developed for typical IDOT HMA mixtures currently used by IDOT in the design of full-depth HMA pavements is as follows:

$$N_f = 8.78 \times 10^{-8} \left(\frac{1}{\varepsilon}\right)^{3.5}$$

where

N_f = number of load cycles to failure

ε = tensile strain at the outer fiber of the hot-mix asphalt (HMA)

Jointed Plain Concrete Pavement

The jointed plain concrete pavement (JPCP) thickness method employs the ACPA fatigue algorithm to compute fatigue damage. According to Bordelon et al. (2009), the equation used for fatigue cracking of JPCP is:

$$\log N = \left[-\frac{SR^{-10.24} \log(R^*)}{0.0112} \right]^{0.217}$$

$$R^* = 1 - \frac{(1 - R')*P_{cr}}{0.5}$$

where

R'= desired reliability of having a given percent of cracked slabs at fatigue damage of 1.0

P_{cr} = a given percent of cracked slabs

R^*= effective reliability or the probability of survival of the pavement

SR = stress ratio

N = allowable load repetitions

D.3 IMPACT OF OVERWEIGHT TRUCKS ON PAVEMENT DETERIORATION

Martin (2002) estimated attributed load-related road wear costs on the basis of two approaches to approximate the marginal cost of road wear for the thin bituminous-surfaced arterial roads in Australia. One approach was based on the statistical relationship between maintenance cost and road use. Martin established a simple linear regression of the annual average maintenance expenditure against the parameter for the road-use variable, both of which were collected through measuring AADT per lane and heavy-vehicle classifications.

The second approach, in which pavement deterioration models were applied, provided a more accurate assessment (Figure D.1). Road roughness was selected to predict pavement condition degradation, and the accumulative roughness consisted of the linear increase of non-load road wear cost and load-related road wear cost (Figure D.2). Attributable road wear cost was assumed to be related to environmental factors: the average of the annual mean maximum and minimum air temperatures, the mean monthly precipitation, and the Thornthwaite index (Thornthwaite 1948) based on soil and climatic conditions.

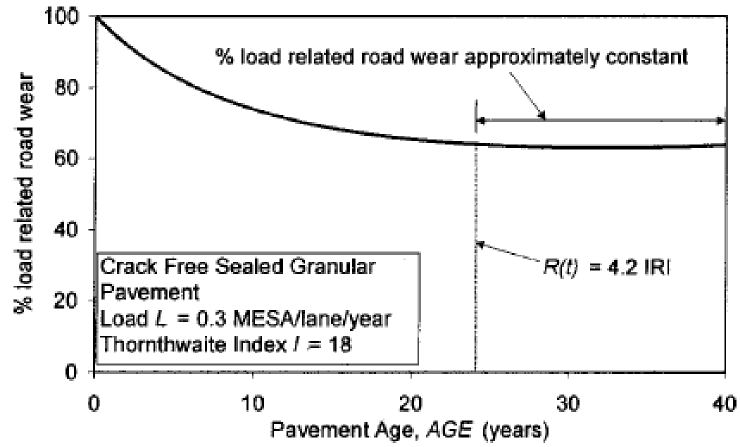

Figure D.1: Estimates for percentage of attributable road wear cost for crack-free sealed granular pavements versus pavement age (Martin 2002).

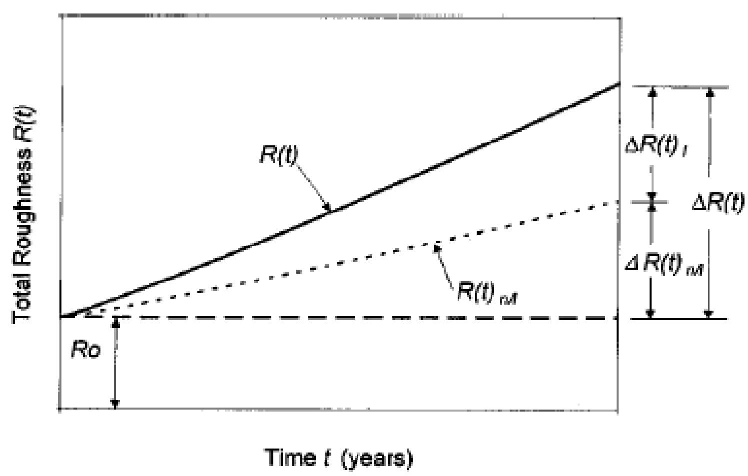

Figure D.2: Simple roughness progression model (Martin 2002).

Boilé et al. (2003) performed a study on impacts of buses on New Jersey's highway infrastructure. The proposed approach and data requirements applied in that study were similar to previous studies. However, buses stopped frequently at bus stops, which was a unique characteristic. Bus axle loads were supplied by vehicle manufacturers for crash load condition, which represented fully loaded buses.

On the basis of the AASHTO procedure, the load of an individual axle was estimated in order to calculate the total ESALs of a whole bus for one pass. It was noted that on New Jersey highways, buses could cause pavement damage comparable to that caused by trucks. Researchers were able to compare and determine which bus type had a more negative influence on pavement. Seventy percent of New Jersey highways were assumed to be flexible pavements, and VMT was the indicator of how much of the infrastructure was used by buses. It was found that buses were responsible for 2.4% of pavement maintenance and rehabilitation costs. A regression model was developed to study the relationship between the asphalt layer coefficient and resilient modulus of asphalt. Figure D.3 shows the regression model of the unique stop-and-go traffic condition. The moduli for stop-and-go and in slow- and normal-condition traffic loading are 300,000, 500,000, and 700,000 psi, respectively. The stop-and-go condition had a greater effect on pavement deterioration than the other two conditions. DARWin software, based on the AASHTO 1993 Design Guide, was used to conduct sensitivity analysis for the hypothetical pavement sections.

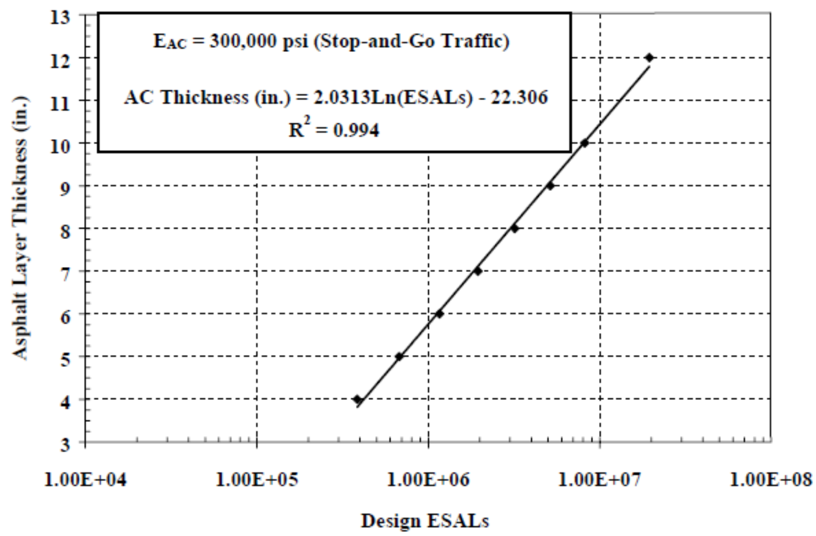

Figure D.3: Design ESALs versus asphalt thicknesses for stop-and-go traffic (Boilé et al. 2003).

Bruzelius (2004) summarized four different approaches to measure the marginal costs of road use: direct approach, indirect approach, full cost-allocation approach (also known as the club and equity approach), and econometric approach. The HDM-4 model (by the World Bank) was the most common and was used in the direct approach. That method is not new and is seldom applied in reality. Both the indirect approach and full cost-allocation approach were established on the foundation of Newbery's fundamental theorem. In that theory, the marginal cost was related proportionally to the average maintenance cost of road use. The Road User Charges version 3.00 (RUC30) model, which was developed by World Bank, is widely used to quantify marginal pavement costs. Because of the difficulties in generating required data and obtaining historic data, examples of an econometric approach, such as the methodology developed by Hajek et al. (1998), were few. Using one or two of the approaches above, the estimates of Swedish, U.S. federal, British, EU, and German studies were included in an international survey report.

Fernando and Oh (2004) developed a methodology for estimating the capacity of existing highways in Texas to carry the traffic loading over a specific analysis period, using the Overweight Truck Route Analysis (OTRA) program, which could also calculate how thick the asphalt concrete (AC) overlay must be to carry the predicted amount of truck axle loads over the pavement design life in accordance with the user-prescribed reliability level. The Asphalt Institute (1982) performance models for fatigue cracking and rutting were used to estimate the allowable number of load repetitions. The researchers assumed four cases of legal vs. overweight distribution (80/20 split, 70/30 split, 60/40 split, and 50/50 split) and four pavement structures (AC over weak base over weak subgrade, AC over weak base over stiff subgrade, AC over stabilized base over weak subgrade, and AC over stabilized base over stiff subgrade). Figure D.4 depicts the effect of truck distribution on predicted pavement service life.

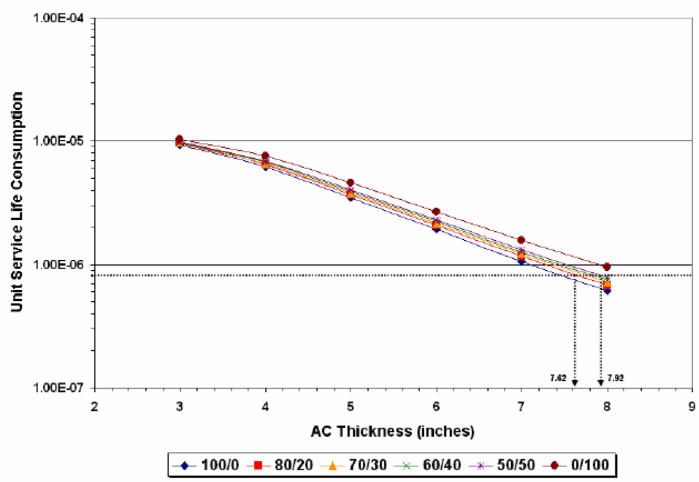

Figure D.4: Effect of truck distribution on the predicted unit service life consumption based on fatigue cracking, AC over weak base over weak subgrade (Fernando and Oh 2004).

Roberts et al. (2005) investigated a methodology to study the impact of overweight hauling timber, lignite coal, and coke fuel on Louisiana highways, using the AASHTO pavement design method. The total tonnage of each commodity transported over the road between the production point and user point in the year 2003 was estimated, and three axle-load scenarios of vehicles carrying each commodity were simulated for each truck type. After the average payload of each scenario had been determined, the number of trucks was estimated and added to the traffic steam. An 8-year overlay design period was used to calculate and compare periodic rehabilitation costs in the entire analysis period. The researchers studied the three scenarios for each commodity to decide whether the extra pavement damage costs caused by overloading could be recovered by the permit fee. It was found that for lignite coal, the triple axle on the trailer with an additional 8,000 lb caused less pavement damage than the tandem axle on the trailer. For timber hauling, there was a gap between the annual damage cost and revenue from overweight permit fees.

Straus and Semmens (2006) conducted a study to estimate the costs of overweight vehicles traveling on Arizona highways. The researchers prepared specific questions and surveyed several states to gather information about overweight permit schedules. Unlike the other studies, the comparisons were made within the states by percentage because the additional weight exceeding the legal weight was the vital factor for estimating the costs of overweight trucks. The researchers concluded that WIM data were inadequate for estimating pavement damage cost because the WIM data were usually sparse and inconsistent. The researchers instead estimated the percentage of overweight vehicles on the basis of existing reports. They recommended a study for future research: determining which types of vehicles were subject to the most overweight violations.

Following Burmister's elastic layered theory (Burmister 1958), Sadeghi and Fathali (2007) established a modeling procedure to predict flexible pavement deterioration caused by overweight traffic. The KENLAYER software was used to simulate the elastic multilayer pavement system. Fatigue cracking (based on the horizontal tensile strain at the bottom of the HMA) and rutting (based on the vertical compressive strain on the top of the subgrade) were taken into consideration to calculate the allowable number of load repetitions. It was found that tensile strain was the critical factor in most cases.

On the basis of a group of reference values, sensitivity analysis was conducted on four contributing factors related to pavement damage: asphalt layer thickness, pavement temperature, subgrade condition, and vehicle speed. The final formula assumed a linear relationship with the deterioration formula from the reference case and affected by changes in the four sensitivity parameters. Pavement deterioration was expressed in terms of the original load repetitions and load cycles after increasing axle loads.

Overweight ticketing was determined using pavement operational life, deterioration, length. The parameters of average asphalt layer thickness, pavement temperature, and subgrade condition (California bearing ratio [CBR]) could be imported as defaults. Vehicle speed, vehicle type, and length of the road traversed by a truck were recorded by a digital truck scale, which was linked to the software. The comparison between the existing fine policy and the overweight ticketing modeled in Sadeghi and Fathali's study indicated that revenue collected through fines was adequate. Figure D.5 illustrates CBR's influence on allowable repetition for standard axles.

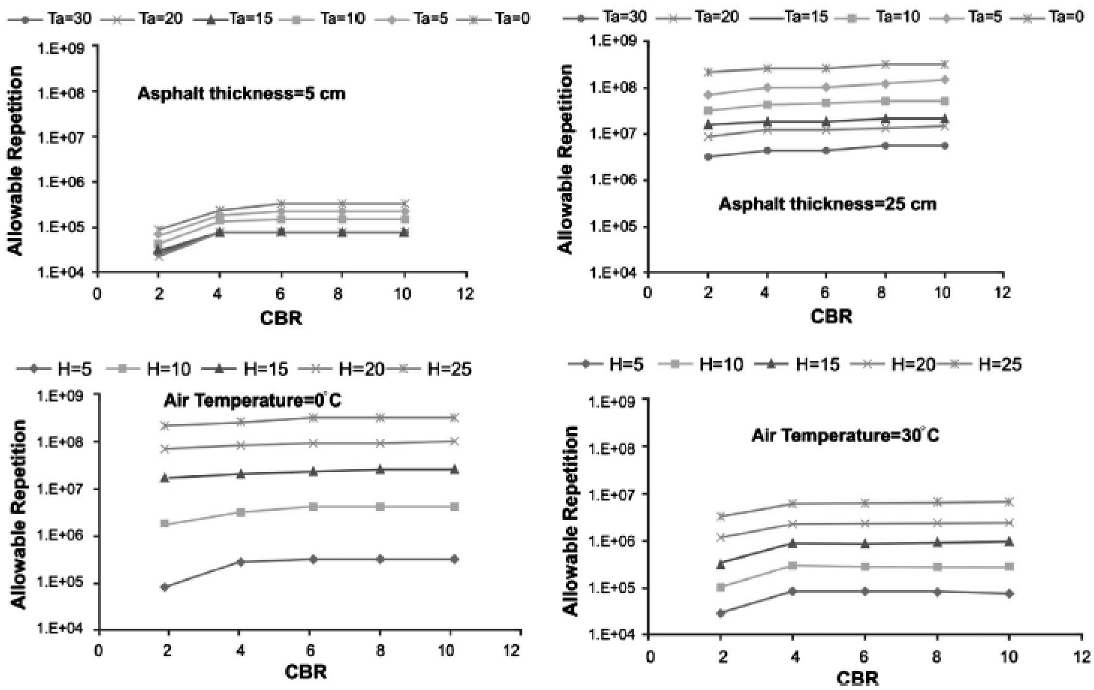

Figure D.5: Subgrade CBR versus allowable repetition for standard axles (Sadeghi and Fathali 2007).

Twenty-seven states participated in a survey conducted by Timm et al. (2007). Permit fee criteria, infrastructure damage assessment techniques, and legal weight limits for those states were summarized and grouped on the basis of the survey results. The researchers emphasized the methodology, including the MEPDG framework, LCCA framework, and three loading scenarios: (1) shifting the entire loading spectra toward heavier loads; (2) specific overloaded axles, which included constant volume–increased weight (CV–IW) and decreased volume–constant weight (DV–CW); and (3) altering the axle configuration on trucks with specific axle weights. Using MEPDG, the researchers obtained the baseline and plots after shifting these three load spectra.

Figures D.6 and D.7 present the loading spectra shifting toward heavier loads. In the pavement damage analysis, pavement thicknesses and pavement life were re-evaluated using the increased load spectra after the baseline was determined for both flexible and rigid pavements. The layered elastic pavement analysis software WESLEA was used to find the maximum horizontal tensile strain at the bottom of the HMA layer for each loading case in flexible pavements. In the cost analysis, a 60-year period with at least two rehabilitation cycles could be considered for every baseline case. An interest rate of 4% was assumed in the LCCA. The analysis compared the cost difference between entire load shifting and specific overloaded axle shifting. Different percentages of the four traffic levels were assumed to transfer tandem axles to tridem axles.

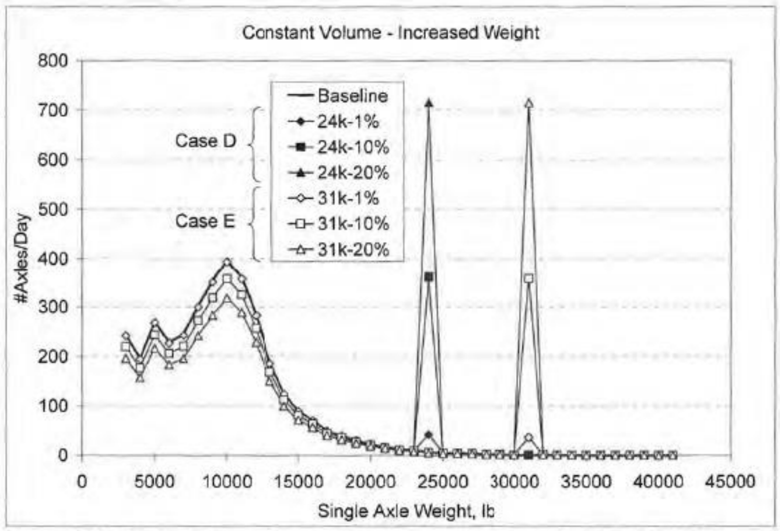

Figure D.6: Single-axle load spectra:
constant volume–increased weight (Timm et al. 2007)

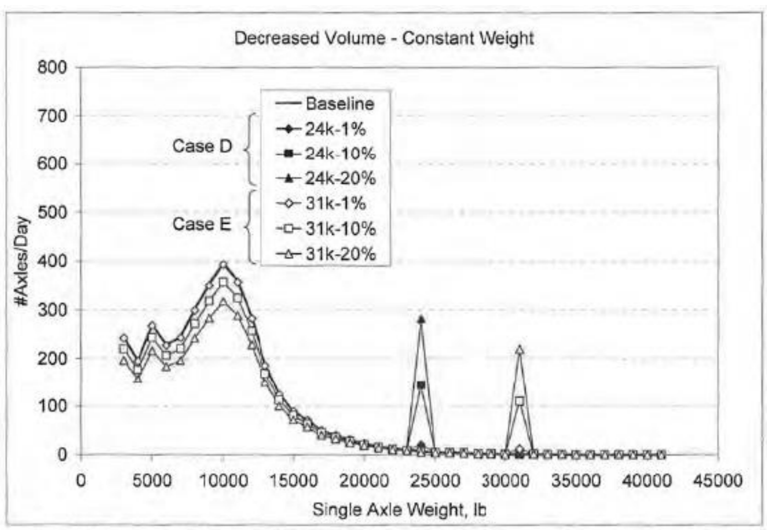

Figure D.7: Single-axle load spectra:
decreased volume–constant weight (Timm et al. 2007).

Saber et al. (2008) investigated the economic impact of heavy sugarcane trucks on Louisiana highways, using the 1986 AASHTO Design Guide. The study assumed two truck classifications (FHWA Type 9 and Type 10) and GVWs of 80, 100, and 120 kips for carrying the commodity. The 1986 AASHTO Design Guide was applied to estimate the overlay thickness of pavement under heavy loading for a 20-year analysis period. The Louisiana Department of Transportation and Development collected ADT and vehicle count data on the 271 control sections from January through March 2006. Using historical data, researchers computed ESALs for current pavement condition and obtained pavement capacity under the assumed traffic loading. It was found that the GVW for FHWA Type 9 sugarcane trucks was supposed to be decreased from 100 to 80 kips but not be increased from 100 to 120 kips. Once the overweight limits were increased, more highway funding was required for the extra pavement damage caused by heavy sugarcane trucks.

The Ohio Department of Transportation (2009) performed a simplified highway cost-allocation study to determine the impacts of overweight trucks. In their study, a 20-year design life was assumed for new pavement construction and a 12-year life for pavement overlays. To obtain total annual costs, the department calculated expenditures for multi-lane and two-lane systems separately for typical pavement cross-sections designated on the basis of highway functional classes. Then they removed overweight trucks from total traffic and performed the same expenditure calculations. The cost allocators depended on two widely used allocators: VMT and ESALs. Using total annual costs and cost allocators, the department compared the costs of including and excluding overweight traffic. It was noted that more 14,500 lane-miles of pavement could be designed thinner if no overweight trucks existed and that the overweight weight trucks were responsible for about $122 million per year.

Using an life-cycle cost (LCC) approach, Liedtke and Scholz (2009) presented a forward-looking method for infrastructure cost calculation. The road infrastructure network was subdivided based on individual structures, and its asset value was computed separately. Each policy decision was undertaken with a view toward minimizing LCC by considering traffic forecasts and technological development. To address the increasing replacement costs over time, economic depreciation was used to distribute the cost fairly between user generations.

Ahmed (2012) conducted marginal pavement damage cost estimation and found it in the range of $0.0033 per ESAL-mile on interstate highways to $0.1157 per ESAL-mile on non-national highway systems. Unlike the methodology used in previous research, various overlay materials types and thicknesses applied at fixed intervals (Figure D.8) were evaluated for flexible and rigid pavements, which was more practical and realistic. There were three different highway types: (1) interstate highways, (2) non-interstate highway system, and (3) non-national highway system. To make the study more comprehensive, truck traffic volumes in four subcategories (very high, high, medium, and low) for each highway type were evaluated separately. The compound annual traffic growth rates were estimated by formulation or assigned by the Indiana Department of Transportation.

Ahmed (2012) used ordinary least-squares (OLS) regression to explore pavement performance models, which had been widely used in other research. Because of heterogeneity, however—which might be caused by unobserved factors—the random parameter regression model was confirmed to be a better option than the OLS model. Over an infinite analysis period, five pavement age groups, including new and old pavements, were considered in pavement life-cycle maintenance, rehabilitation, and reconstruction (MR&R) profile. The present worth of MR&R cost for partial cycle and full cycle was converted to an equivalent uniform annual cost (EUAC). The MR&R EUAC was related to road use (ESALs), pavement types, and pavement age. Four scenarios were simulated to study the impacts of non-consideration of reconstruction or maintenance costs on marginal pavement damage costs. It was found that the only realistic option was to incorporate a highway agency's MR&R strategy into the marginal pavement damage cost estimation and consider all pavement repair costs. The pavement life-cycle length and the interest rate used in the analysis were the factors that had the most significant influence on marginal pavement damage costs. It was also found that the length of the rest period and effectiveness of rehabilitation treatments could affect the accuracy of marginal pavement damage cost estimates.

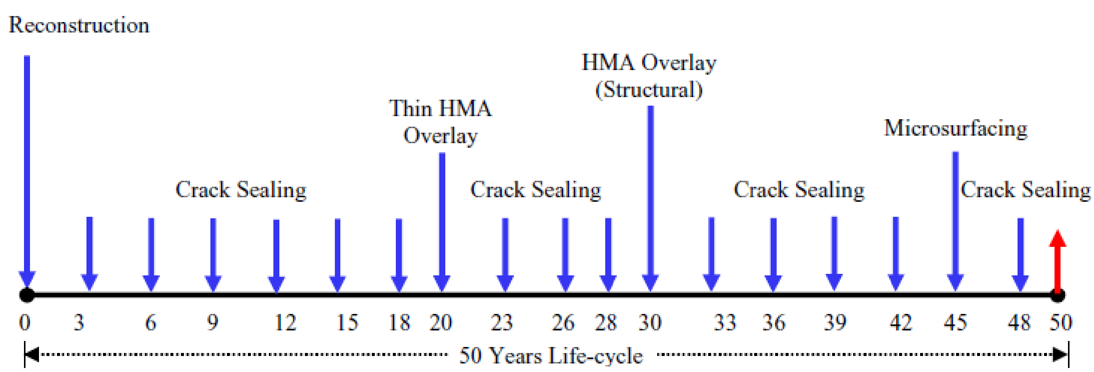

Figure D.8: Typical pavement life-cycle MR&R profile (Ahmed 2012).

Wang et al. (2015) evaluated the impact of overweight traffic on pavement life using an M-E analysis approach. Field performance data at the sites where WIM data had been collected were analyzed to estimate pavement service life under field conditions. The pavement structures considered in the analysis included flexible pavement and composite pavement with different combinations of layer thickness. Different distribution patterns were observed between the overweight and non-overweight traffic in terms of truck classes and axle-load spectra. The reduction ratio of pavement life was used to normalize the effect of an overweight truck under different conditions. A linear relationship was found between the overweight percentage and the reduction ratio of pavement life, regardless of the variation in traffic loading and pavement structure (Figure D.9). In general, it showed that a 1% increase in overweight trucks could cause a 1.8% reduction of pavement life.

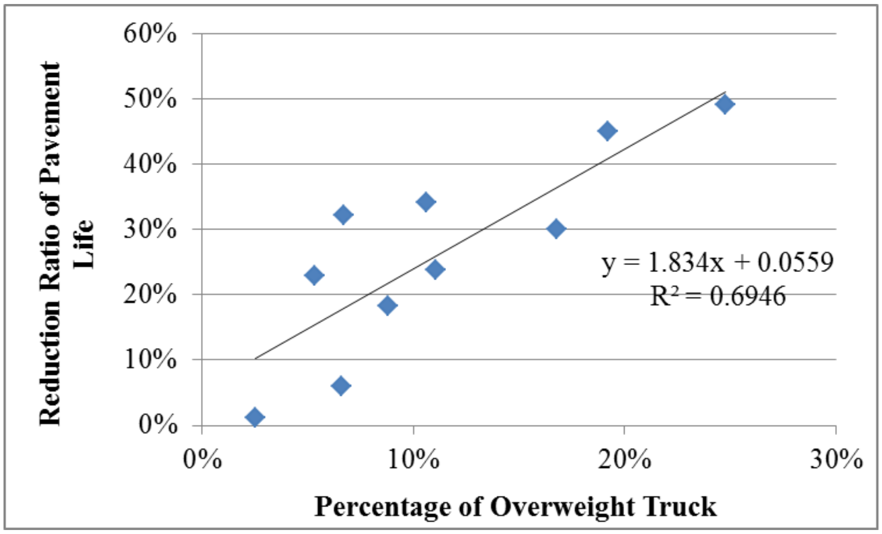

Figure D.9: Reduction ratio of pavement life with overweight truck percentage.

Latifi (2014) selected four Wisconsin state trunk highways to conduct both field investigation and AASHTOWare MEPDG analysis. In the field investigation, overweight truck traffic volume was measured over a 6-hour period, and a visual distress survey of a 150 ft section was conducted. The overweight truck traffic count was compared with information available in the Wisconsin Department of Transportation (WisDOT) database. The visual distress survey indicated generally severe pavement damage and stress across the sampled sections. The axle-load spectra (ALS) developed by WisDOT was considered a baseline traffic input in MEPDG, and the ALS for OS/OW traffic was obtained from single-trip permit records to merge the baseline and OS/OW traffic. Pavement deterioration levels predicted by MEPDG were consistent with the levels found in the field investigation, and a relative distress increase of 0.5% to 4% caused by OS/OW traffic was noted. Figure D.10 shows the pavement layer-rutting difference over pavement life resulting from normal traffic and from normal traffic with OS/OW single-permit truckloads.

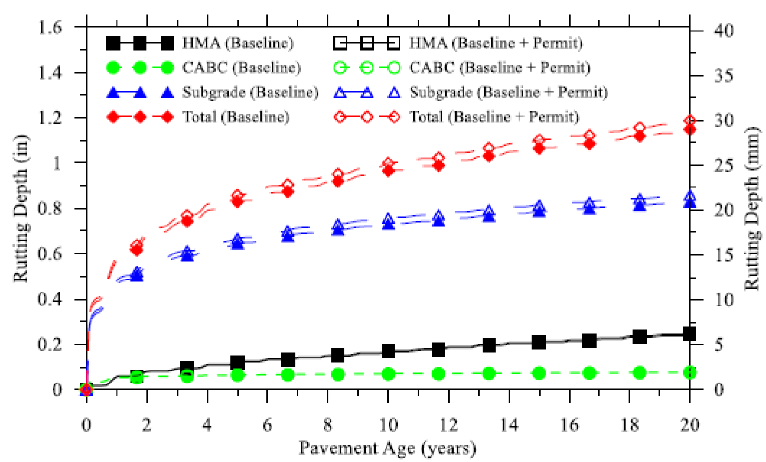

Figure D.10: Pavement layer-rutting over pavement life resulting from normal traffic and normal traffic plus OS/OW single-trip permit truck loads for Wisconsin State Highway 140, with 50% reliability (Latifi 2014).

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APPENDIX E: BRIDGES

E.1 BRIDGE DESIGN AND ANALYSIS

Over the years, design procedures have been developed to provide a satisfactory margin of safety. These procedures were based on the engineer's confidence in the analysis of the load effects and the strength of the materials being provided. As analysis techniques improved and quality control of materials became better, the design procedure changed.

E.1.1 Variability of Loads

The earliest design procedures, such as the allowable stress design (ASD), were developed with a primary focus on metallic structures. Structural steels were observed to behave linearly up to a relatively well-defined yield point that was safely below the ultimate strength of the material. Nevertheless, such methods did not distinguish between different types of loads (including dead, live, and wind loads) nor did they recognize different levels of uncertainty. Thus, the safety factor is applied to the resistance side of the design inequality, and the load side is not factored. The safety factor is determined using the following equation:

$$\frac{R}{F} \geq effect\ of\ loads, Q \qquad \text{(E.1)}$$

where

R = resistance

F = safety factor

In the ASD, fixed values of design loads are selected for the design code, and the varying degree of predictability of the different load types is not considered. In addition, because the safety factor is chosen based on experience and judgment in the ASD, the safety factor is applied only to resistance, and loads are considered to be deterministic. The resistance concepts are based on the elastic behavior of isotropic, homogenous materials. Also, ASD does not embody a reasonable measure of strength, which is a more fundamental measure of resistance than allowable stress is. What is needed to overcome these deficiencies is a method that is based on the strength of the material, considers variability not only in resistance but also in the effect of loads, and provides a measure of safety related to the probability of failure.

E.1.2 Load and Resistance Factor Design (LRFD)

To account for variability in load, the resistance in Equation E.1 is multiplied by a statistically based resistance factor Φ, whose value is usually less than 1. The load side is multiplied by a statistically based load factor γ, whose value is usually greater than 1. Because the load effect at a particular limit state involves a combination of different load types (Q_i) that have different degrees of predictability, the load effect side is represented by a summation of $\gamma_i Q_i$ values. If the nominal resistance is given by R_n, the safety criterion is the following:

$$\phi R_n \geq effect\ of\ \sum \gamma_i Q_i \quad \text{(E.2)}$$

The resistance factor Φ for a particular limit state must account for a plurality of uncertainties. The load factor γ_i, chosen for a particular load type, must consider the uncertainties in magnitude of loads, arrangement of loads, and possible combination of loads. In selecting resistance factors and load factors for bridges, probability theory has been applied to sample data on the strength of materials, and statistics on weights of materials and vehicular loads.

The load and resistance factor design (LRFD) accounts for variability in both resistance and load, achieves fairly uniform levels of safety for different limit states and bridge types without involving complex probability or statistical analysis, and provides a rational and consistent method of design. However, it requires availability of sufficient statistical data and probabilistic design algorithms to make adjustments in resistance factors to meet individual situations.

The basic design expression in the LRFD bridge specifications is given by:

$$\eta \sum \gamma_i Q_i \leq \Phi R_n \quad \text{(E.3)}$$

where

Q_i = force effect

R_n = nominal resistance

γ_i = statistically based load factor applied to the force effects

Φ = statistically based resistance factor applied to the nominal resistance

η = load modification factor

For all non-strength limit states, Φ =1.0

The load modification factor takes into account the ductility, redundancy, and operational importance of the bridge. It is given by the following equation:

$$\eta = \eta_D \eta_R \eta_I \geq 0.95 \quad \text{(E.4)}$$

where

ηD = ductility factor

ηR = redundancy factor

ηI = operational importance

Brittle behavior is to be avoided because it implies a sudden loss of load-carrying capacity when the elastic limit is exceeded. The values to be used for the strength limit state ductility factor are:

ηD = 1.05 for non-ductile components and connections

ηD = 0.95 for ductile components and connections

Redundancy in the bridge system will increase its margin of safety, and this is reflected in the strength limit state redundancy factors given as:

ηR = 1.05 for non-redundant members

ηR = 0.95 for redundant members

In the event of earthquakes, it is important that all bridges remain open. Therefore, the following requirement applies to extreme-event limit states:

$\eta_I \geq 1.05$ for a bridge of operational importance

$\eta_I \geq 0.95$ for unimportant bridge

In the context of reliability analysis, load and resistance factors are selected to ensure that each possible limit state is reached only with an acceptably small probability of failure. The probability of failure can be determined if the mean and standard deviation of the resistance and load distribution functions are known. As long as the resistance R is greater than the effects of the load Q, there is a margin of safety for the limit state under consideration. A quantitative measure of safety is the probability of survival given by:

$$p_s = P(R > Q) \tag{E.5}$$

where the right-hand side represents the probability that R is greater than Q. The probability of failure will be the complement ($1 - p_s$).

E.2 BRIDGE RATING PRACTICES AND POLICIES

Bridge load rating in the United States is guided by the AASHTO Manual for Condition Evaluation of Bridges (MCEB). The safe load-carrying capacity refers to a load level that the bridge can safely carry—but not its ultimate capacity. The bridge load-rating process does not involve experimental testing of material or physical testing of the entire bridge; instead, it is mathematically modeled and analyzed (NCHRP 2006). The following load-rating factor (LRF) is the result of bridge load rating for a bridge component with respect to a specific failure mode (bending moment, shear, etc.), according to the MCEB:

$$LRF = \frac{R - A_1 \times DL}{A_2 \times LL} \tag{E.6}$$

where

R = bridge component's resistance for that particular failure mode

DL = total dead load effects in that component

LL = total live (vehicular) load effects in that component

A_1 and A_2 = dead- and live-load factors that cover possible uncertainties involved in estimating DL and LL

This formula is applied to all of the critical cross-sections of the bridge component, and the lowest LRF is taken as the LRF for that bridge component. In addition, the same process is applied to all components of concern, and the lowest of the resulting LRFs for those components and failure mode is typically taken as the bridge's LRF.

The bridge is said to be capable of carrying the design standard vehicular load if the LRF of the bridge is found to be 1.0 or higher. This indicates that the bridge has a safe load-carrying capacity equal to the standard load's tonnage multiplied by the LRF. For example, if a bridge is found to have an LRF of 1.20 for the AASHTO standard HS20 live load, which has a GVW of 72,000 lb as shown in Figure E.1, it is said to have a capacity of 1.2 x 72,000 lb = 86,400 lb. Note that the load-carrying capacity in tonnage depends on the reference standard vehicle load used because the vehicular live-load effects used in Equation E.1 are not proportional between different standard loads. Figure E.1 shows the maximum load effects (bending moments and shear forces) of simply supported bridge spans for AASHTO HS20 loads (GVW of 72,000 lb), while Figure E.2 shows that for AASHTO H20 loads (GVW of 40,000 lb). The ratio of the two maximum load effects is not always 40/72, which indicates that different permit vehicles may induce different load effects in a bridge's component.

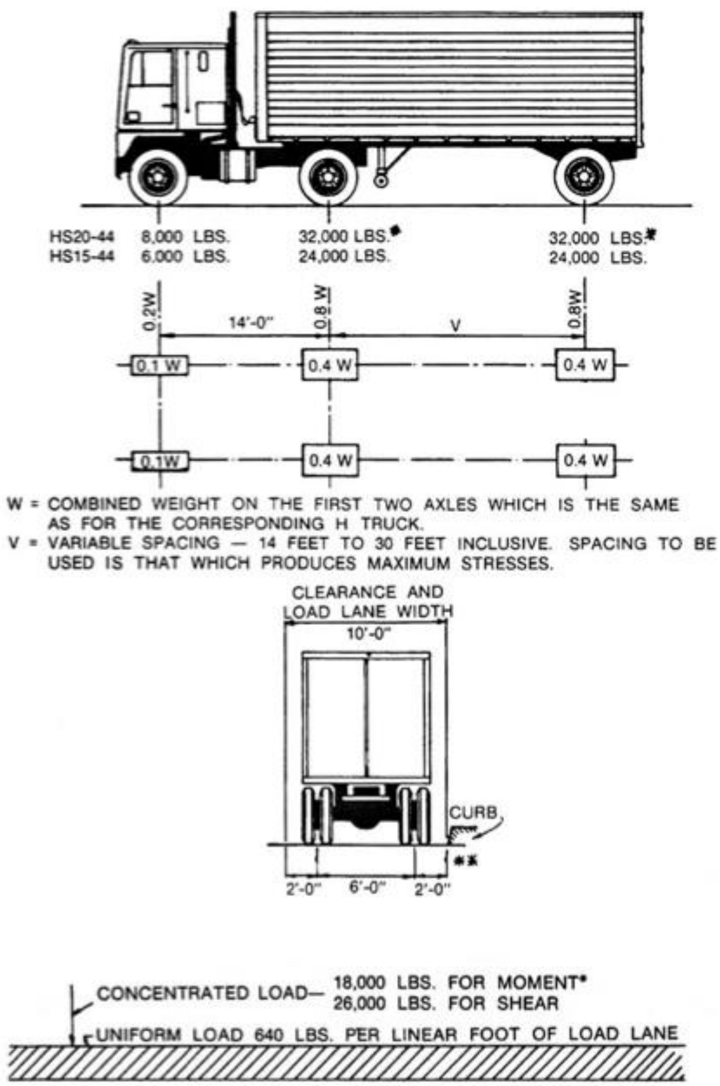

Figure E.1: AASHTO standard HS20 vehicle live load (AASHTO 2011).

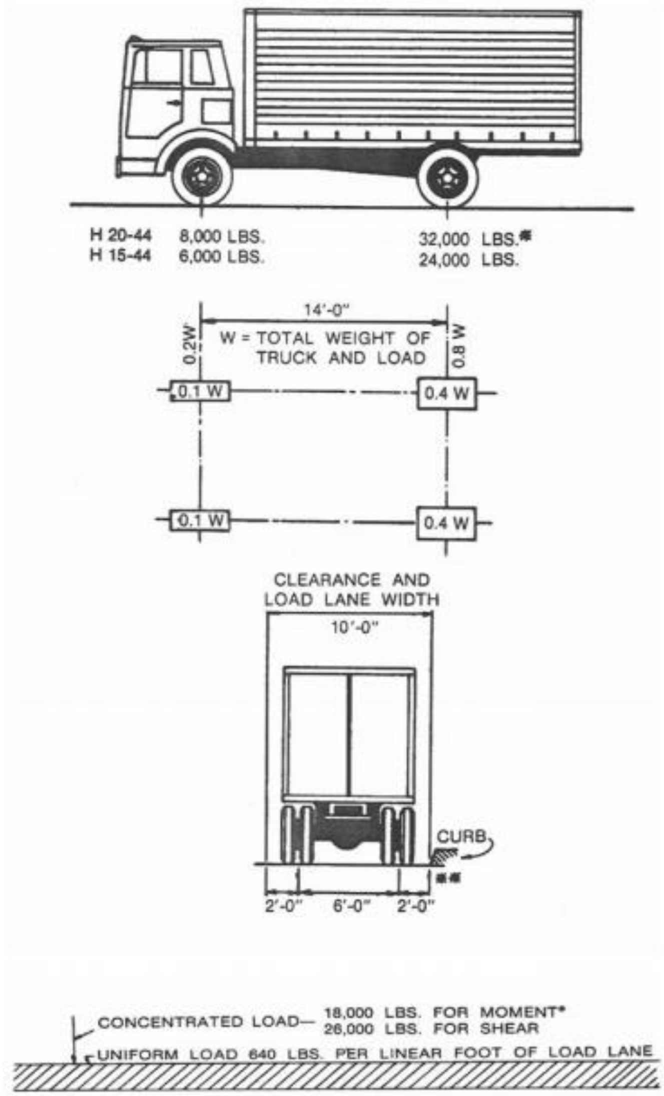

Figure E.2: AASHTO standard H20 vehicle live load (AASHTO 2011).

In Equation E.6, A_2 is the live-load factor. In the MCEB, this factor is prescribed as 1.3 for the operating rating and 2.17 for the inventory rating. The operating rating refers to the maximum load level the bridge is allowed to carry, while the inventory rating is a load level the bridge is allowed to carry without a time limit. There have been discussions on whether using these live-load factors for permit review is appropriate because the bridge evaluation for permits focuses on the particular permit vehicle and the load-rating process considers general truck traffic loads—and those two groups have different probabilities of occurrences. The AASHTO Guide Manual for Condition Evaluation and Load Resistance Factor Evaluation (LRFD) of Highway Bridges 2003 has adopted a probabilistic concept of prescribing different load factors for the standard bridge load rating and the bridge evaluation permit review (AASHTO 2011).

Because bridge load rating uses standard vehicle loads, such as the AASHTO HS and H loads, the bridge rating system's results are directly useful only when the permit vehicle's configuration is close to the standard load used. If that is not the case, the typical approach for reviewing the permit is to load the bridge with the permit vehicle and then determine whether the bridge can sustain the load, which is referred to as bridge evaluation for permit review. The AASHTO MCEB has been used extensively in guiding bridge evaluation for permit review (NCHRP 2006).

E.3 BRIDGE DETERIORATION MODELS

A study in Minnesota investigated the fatigue damage caused by increased truck weight on steel and prestressed concrete bridges. Five steel bridges and three prestressed concrete bridges on Minnesota highways were selected for instrumentation and loading. To validate the results, the selected bridges were modeled using SAP2000 software. It was found that for prestressed concrete bridges, a 10% to 20% increase in allowable GVW did not have a significant impact on the fatigue life of bridges because of a very small increase in the stress range (Altay et al. 2003). For most modern steel bridges, a 20% increase in truck weight would not cause critical fatigue issues. However, for steel bridges with very high traffic volumes and very poor fatigue details, fatigue might be a safety concern (Altay et al. 2003).

It has been a difficult task to assign maximum allowable loads on bridges for different truck types. State and local agencies use the federal bridge formula to determine the maximum allowable load on bridges. While many bridge studies and models exist, researchers cannot generalize many findings because specific bridge conditions, traffic patterns, truck fleets, and environmental conditions are among many variables that keep changing. Helgason et al. (1976) found that factors including stress range, yielding stress, and rebar size and shape affect the fatigue strength. The stress range was the most critical in determining the fatigue life of the rebar. The fatigue life of the rebar can be estimated using the following equation:

$$\log N = 4.419 - 0.0392\sigma - 0.013\sigma_{min} + 0.0079G + 7.8059D_{nom} - 8.4155D_{nom}^2 + 2.799D_{nom}^3 \tag{E.7}$$

where

N = fatigue life in number of stress cycles

σ_{min} = minimum stress during stress cycle (ksi)

G = rebar yield strength (ksi)

D_{nom} = nominal rebar diameter (in)

Figure E.3 shows a typical rebar fatigue curve in terms of the stress range (endurance limit) (S) and the number of cycles (N). A rebar is expected to be able to sustain an unlimited number of cycles if its stress range is below the endurance limit of 20 ksi.

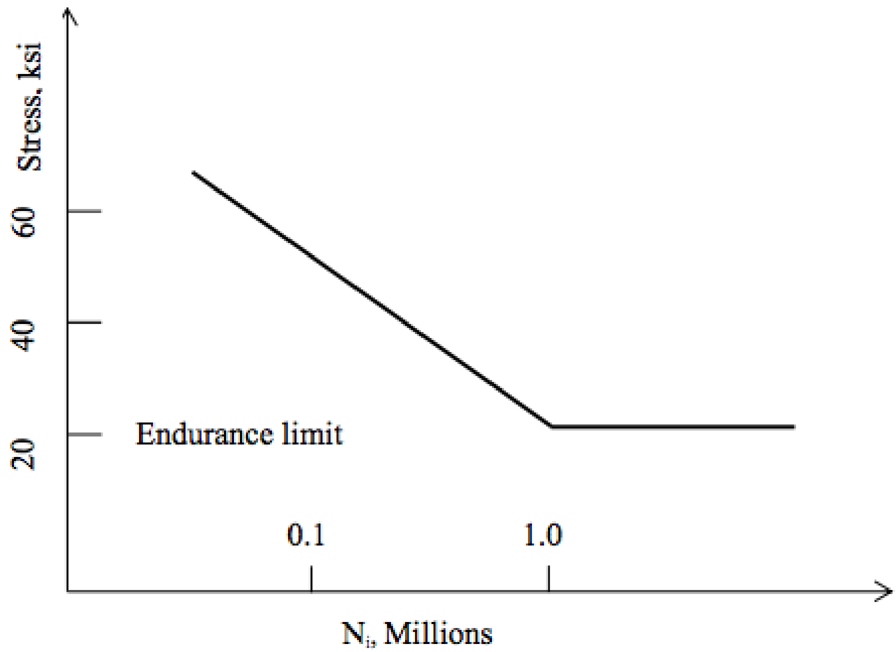

Figure E.3: Rebar S-N curve (Helgason et al. 1976).

Overman et al. (1984) investigated the fatigue behavior of pretensioned concrete girders. That study included full-scale fatigue tests of flexural prestressed concrete girders. The researchers found that among the different fatigue failure mechanisms of prestressed concrete girders, the most common fatigue failure was fatigue fracture of prestressing strands. To estimate the fatigue life of the prestressing strands, the following equation by Paulson et al. (1983) can be used:

$$\text{Log } N = 11 - 3.5 \log \sigma \qquad \text{(E.8)}$$

where

N = fatigue life in number of stress cycles

σ = prestressing strand stress range (ksi)

Chowdhury et al. (2013) conducted a study in South Carolina to investigate the impact of overweight and oversize trucks on bridges. The bridge damage methodology is presented in Figure E.4. The first step was to develop a series of truck models to represent the truck population in South Carolina. The truck models, with different truck weights and axle configurations, were developed based on truck gross weight and axle configuration distribution, as well as truck limits in South Carolina.

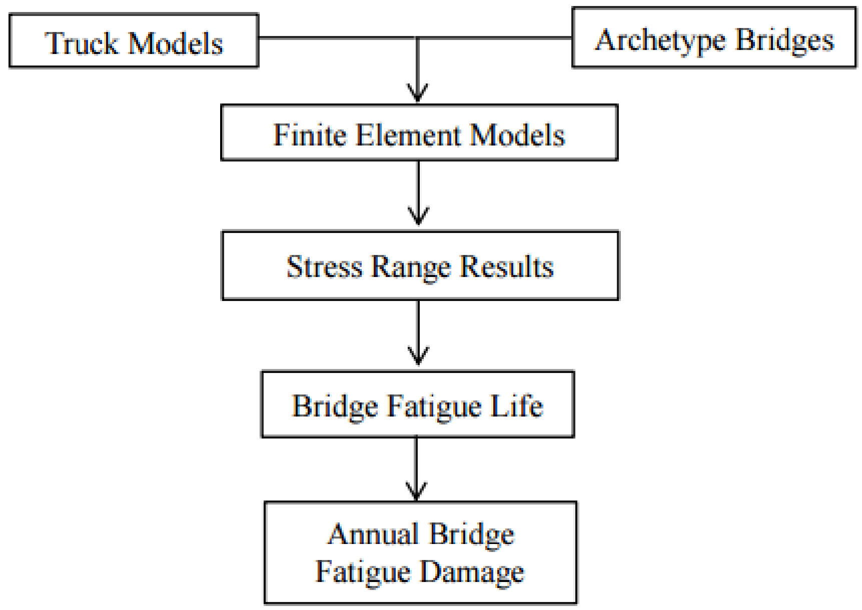

Figure E.4: Bridge damage modeling method (Chowdhury et al. 2013).

Because of the large number of bridges in South Carolina (9,271), it was not feasible to create a finite element model for each bridge. The second step, therefore, was to develop archetype bridges to represent groups of bridges that share common features and structural characteristics. Bridge information, such as material, span length, count, location, etc., was obtained from the National Bridge Inventory database. For the project, four types of archetype bridges were modeled to quantify bridge damage caused by trucks: a reinforced concrete slab bridge with a span of 33 ft, a prestressed concrete beam bridge with a span of less than 66 ft, a prestressed concrete beam bridge with a span of 66 to 115 ft, and a prestressed concrete beam bridge with a span 115 to 148 ft. The finite element models for all archetype bridges were built using LS-DYNA software. The stress ranges for each analysis were recorded, and the bridge fatigue lives for the archetype bridges were calculated using Equation E.7.

The annual bridge damage caused by a truck model is defined as the annual consumed fatigue life by a particular truck model (N_{Ci}) divided by the bridge fatigue life of that truck model (N_i). The total bridge fatigue damage (D) is the sum of fatigue damage from all truck models, as presented in Equation E.9.

$$Fatigue\ Damage\ (D) \sum \left(\frac{N_{ci,1}}{N_{i,1}} + \frac{N_{ci,2}}{N_{i,2}} + \frac{N_{ci,3}}{N_{i,3}} + \cdots + \frac{N_{ci,n}}{N_{i,n}} \right) = \qquad \text{(E.9)}$$

where

N_{ci} = number of loading cycles consumed for the i^{th} truck model with GVW levels 1 to n

N_i = allowable number of cycles for the i^{th} truck model with GVW levels 1 to n

i = truck type

Note that the bridge fatigue damage (D) is a unitless quantity, where D equal to 0 corresponds to no damage and D equal to 1 means that the particular bridge is in complete damage condition under repetitive fatigue loading (Chowdhury et al. 2013).

AASHTO LRFD specifications provide a design fatigue truck with a GVW of 54 kips, a front axle spacing of 14 ft, and a rear axle spacing of 30 ft (AASHTO 2011). AASHTO LRFD states that the maximum design ADT under normal conditions is limited to around 20,000 vehicles per lane. This maximum design ADT can be used to estimate the single-lane average daily truck traffic ($ADTT_{SL}$) by multiplying it by the percentage of truck traffic shown in Table E.1 (AASHTO 2011).

Table E.1: Distribution of Truck Traffic (AASHTO 2012)

Highway Classification	Percentage of Trucks in Traffic
Rural Interstate	20
Urban Interstate	15
Other Rural	15
Other Urban	10

E.4 BRIDGE DAMAGE COST ESTIMATION AND ALLOCATION

An Indiana study investigated the costs of bridge damage attributable to overweight trucks. According to Indiana Department of Transportation jurisdictions, a truck is considered to be overweight when the GVW exceeds 80,000 lb. The damage was studied based on the axle spacing and number of axles. As presented earlier, the AASHTO design vehicles are used in bridge designs to simulate the most severe live loads. However, the design vehicles are different from those observed operating on the roads. To assign the cost increment to each vehicle weight group, a quantitative correlation between the AASHTO design vehicles and the FHWA vehicles operating on the highway was established. To do that, the researchers used the equivalent load approach proposed by Tee et al. (1986). The approach relates the amount of moment produced by AASHTO design vehicles to that of FHWA vehicles. The researchers found that the moments produced depend on the axle spacing, the axle-load distribution of the vehicles, and the bridge span type (simple or continuous). The model that links the AASHTO design vehicles to the FHWA vehicles is presented in Figure E.4. Seven main steps were used to develop the modified equivalent-vehicle (MEV) model approach.

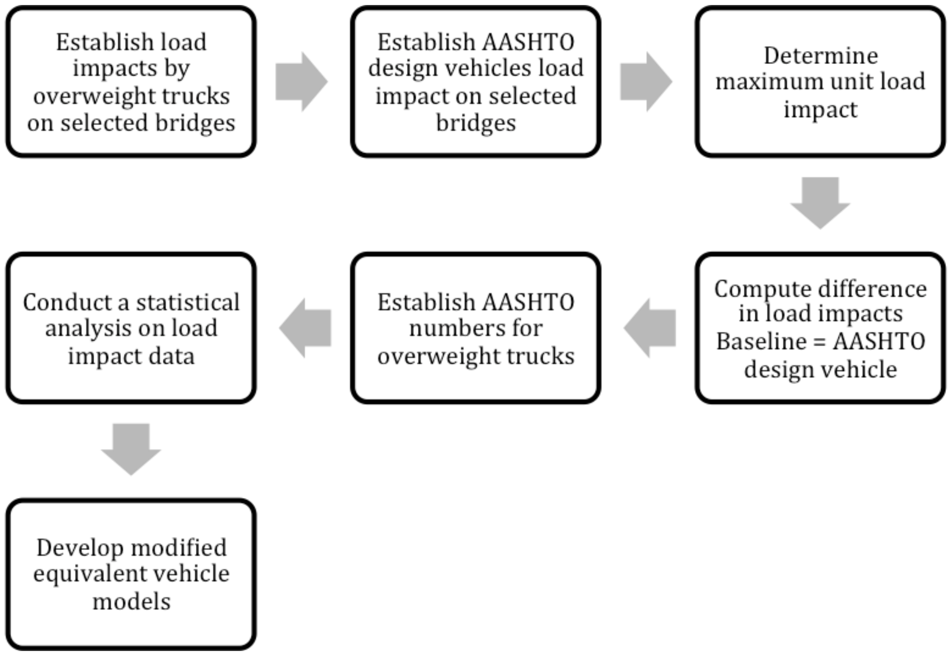

Figure E.4: The MEV methodology.

After the critical or maximum moments produced by overweight trucks and the critical moments produced by AASHTO standard vehicles (HS15 and HS20) have been computed, the unit design critical moment is established. This is calculated using the following equation:

$$UDM_{HSI} = \frac{CM_{HS20} - CM_{HS15}}{L_{HS20} - L_{HS15}} \qquad \text{(E.10)}$$

where

UDM_{HS1} = unit design critical moment

CM_{HS20} = computed critical moment produced by load HS20

CM_{HS15} = computed critical moment produced byHS15

$L_{HS20} – L_{HS15}$ = 5, which is the numerical difference between 20 and 15

The next step is to establish the difference between the critical moment of an FHWA vehicle and that of an HS20. The critical moment produced by HS20 was used as the baseline in computing the difference. In the following step, each AASHTO code was correlated to each FHWA weight group class. A multivariate non-linear regression was then conducted using the generated data from steps one through five (Figure E.4). Three independent variables were found to be statistically significant, at the 99% confidence level, in estimating the equivalent-vehicle model. Equation E.11 presents the MEV model.

$$MEV = \alpha \left(\frac{GVW}{AAS}\right)^{\beta} AAL^{\gamma} \qquad \text{(E.11)}$$

where

MEV = modified equivalent-vehicle (AASHTO) loading

GVW = gross vehicle weight (lb)

AAS = average axle spacing (in)

AAL = average axle load (lb)

α, β, and γ = estimated parameters for the modified equivalent-vehicle model as presented in Table E.2

Table E.2: Statistical Results for the MEV Model

Parameter	Coefficient	t-Statistic	Adjusted R^2
α	0.0057	2.121	0.923
β	0.3130	6.804	
γ	0.6400	4.211	

For estimating the bridge damage repair cost caused by OW/OS trucks, the input variables included highway type, bridge material type, bridge length, bridge deck width, bridge age group, equivalent uniform average annual traffic volumes, and bridge life-cycle cost per unit of bridge length (Anwaar et al. 2012).

In another study, conducted by Chotickai and Bowman (2006), researchers evaluated steel bridge fatigue damage caused by overweight vehicles along a high traffic-volume highway in northern Indiana. Truck weight distribution was obtained using WIM data. Several truck classes were used in the study, including FHWA Class 9 and 13 trucks, which were found to be the most common truck types in Indiana. The maximum weights that were considered for these types were 150,000 and 200,000 lb, respectively. The average truck gross weight for all trucks in both directions on the considered highway was 52,368 lb; vehicle Class 9 and 13 trucks had average gross weights of 54,356 and 119,459 lb, respectively. Strain gages were installed to obtain the strain range and estimate fatigue damage. The researchers found that fatigue failure for the bridges in the study was not a concern because overweight trucks made up less than 1% of the entire truck population in Indiana.

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APPENDIX F: SAFETY

F.1 STATISTICAL ANALYSIS FOR ROADWAY SAFETY

Roadway safety is a major concern in transportation. According to the NHTSA's Fatality Analysis Reporting System (2013), 33,651 people died in crashes on roadways in 2012 nationwide. Of those crashes, 918 occurred in Illinois. To better understand how risk exposure variables such as traffic, vehicle type, or crash condition affect crash frequency (number of crashes per unit of time per unit roadway length) and severity, statistical models must be developed and data-driven analysis must be conducted. The models should provide accurate predictions of the number of crashes at a particular location as a function of traffic volume, location geometry, and driving conditions.

F.1.1 Multivariate Models

Regression models that relate crash frequency to traffic and other location characteristics are often referred to as multivariate models (Persaud et al. 2001). Accident prediction models are intended to estimate the relative safety of a location by determining a safety performance function (SPF). According to the FHWA (2013), SPFs are "equations used to predict the average number of crashes per year at a location as a function of exposure and, in some cases, roadway or intersection characteristics (e.g., number of lanes, traffic control, or median type)."

Thus, such functions help to quantify the expected number of crashes for various design variables and condition settings (Tegge 2008; Hauer 2004; Bauer and Harwood 2000; Miaou 1994; Lord 2000; Vogt and Bared 1998). For instance, Tegge (2008) developed SPFs for various roadway segments and intersections as a function of traffic. That study was conducted for fatal, A-injury, B-injury, and combined fatal and injury crashes. Bauer and Harwood (2000) focused specifically on intersections. In their model, AADT accounts for the most of the variability, while other variables such as the number of lanes, access control, or grade account for only a small amount of the variability in the data. Miaou (1994) studied truck safety and assessed the relationship between truck accident frequency and road geometry (e.g., curve, grade). To estimate the expected accident frequency of an infrastructure element, Hauer's model variables (2004) included traffic flow, shoulder type, grade, and number of driveways. Persaud and Dzbik (1993) developed SPFs for freeways as a function of ADT on a macroscopic scale, as well as hourly volume on a microscopic scale. Vogt and Bared (1998) designed accident models for two-lane rural segments and intersections, with numerous variables for both segments and intersections, such as curve degree, grade, lane width, posted speed, and channelization on the main road.

The SPFs also must be determined by a model equation. In the most generic form, the model equation would be:

$$\mathrm{Y} = \mathrm{f}(\mathrm{X}_1, \mathrm{X}_2, \ldots \mathrm{X}_\mathrm{n}, \beta_1 \beta_2 \ldots \beta_\mathrm{m}), \qquad \text{(F.1)}$$

where

Y = the expected crash frequency

Xi = covariates of the model

β_i = parameters of the function f

The choice of f is arbitrary, but one often-used model equation has the following form (Hauer 2004; Vogt and Bared 1998):

$$Y\alpha_0 \cdot \exp(\beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_n X_n) \qquad \text{(F.2)}$$

or a slightly modified version of it (Tegge 2008; Bauer and Harwood 2000; Persaud et al. 2001; Hauer and Persaud 1996):

$$Y = \alpha_0 \cdot (\mathrm{AADT})^{\beta_1} \cdot \exp(\beta_2 X_2 + \cdots + \beta_n X_n) \qquad \text{(F.3)}$$

To model the SPF, Hauer (2004) split it into two parts: a multiplicative portion and an additive portion. The former accounts for factors that apply to stretches of road (e.g., traffic flow, lane width, shoulder type), and the latter accounts for factors that apply locally (e.g., number of driveways, short bridges). SPFs are commonly split into two categories. Level 1 SPFs are models based only on traffic volumes for both segments and intersection. Studies have found that ADT has the largest impact on the variability of the accident data (Tegge 2008; Bauer and Harwood 2000). Level 2 SPFs incorporate other variables to model crash frequency, using Equation F.3.

Accident prediction models were also used to estimate frequency of accidents involving pedestrians or cyclists (Brüde and Larsson 1993) or to estimate vehicle roadside encroachment (Miaou et al. 1996).

Alternative methods consist in ratio computation (Campbell et al. 1988; Stein and Jones 1988; Fancher and Campbell 1995). Campbell et al. (1988) analyzed the impact of truck configuration, gross weight, area type, and road type on truck crash incidence. Aggregate rates were first computed as the ratio of the total number of vehicles of a given category involved in accidents to the total miles traveled. To remove travel category influence from the comparison, adjusted rates were computed based on aggregate rates. Also, to investigate truck configuration influence on crashes, Stein and Jones (1988) derived involvement ratios as the ratio of the percentage of trucks with a given characteristic in the crash group to the percentage of trucks in the comparison group.

F.1.2 Log-Linear Analysis

In the literature related to roadway safety, two log-linear regression models are usually used: a Poisson regression model and a negative binomial regression model.

The Poisson distribution is a discrete distribution that expresses the probability of a specified number of events to occur in a given amount of time. Equation F.4 expresses the probability of y_i events occurring in a given amount of time, where λ_i is the expected number of events to occur in this amount of time (Tegge 2008).

$$P(Y = y_i) = \frac{{\lambda_i}^{y_i} e^{-\lambda_i}}{y_i!} = \qquad \text{(F.4)}$$

If Y is the random variable that represents the number of crashes on a given roadway segment or intersection, then Y takes on the form described in Equation F.2 and takes the value y_i with probability $P(Y = y_i)$.

Poisson distribution is appropriate for applications that involve counting the number of times a random event occurs in a given amount of time. However, recent research has shown that the use of a Poisson regression model is often inaccurate (Miaou 1994; Persaud et al. 2001; Hauer 2004). The assumption of a Poisson distribution implies that the mean of the random variable must be equal to the variance. Experience with the data shows variations and overdispersion that contradict this latter assumption (Dean and Lawless 1989; Miaou and Lum 1993, as cited in Persaud et al. 2001). Overdispersion occurs when the variance is greater than the mean and could lead to biased model coefficients. Alternatively, Hauer (2004) proposed a model in which the count of accidents is modeled as θY, where Y is the random variable described in Equation F.1 and θ is another variable following a gamma distribution.

Thus, to deal with the overdispersion problem, the other commonly used regression model is the negative binomial (NB) model. The NB distribution allows the variation to be larger than the mean, and it describes the occurrence of discrete events. In effect, the NB model allows for variation caused by variables that were not included in the model (Tegge 2008). Equation F.5 expresses the probability of a random variable that follows a negative binomial distribution to take the value y_i:

$$P(Y = y_i)\frac{\Gamma\left(y_i + \frac{1}{k}\right)}{y_i!\,\Gamma\left(\frac{1}{k}\right)} = \left(\frac{k\lambda_i}{1 + k\lambda_i}\right)^{y_i}\left(\frac{1}{1 + k\lambda_i}\right)^{1/k} \tag{F.5}$$

The variance can be expressed as $\lambda_i + k(\lambda_i)^2$, where λ_i is the mean of Y. Hence, this regression model assumes that the crash frequency Y follows a negative binomial distribution with parameters λ_i and k. Note that when the overdispersion parameter k approaches zero, the NB model approaches a Poisson distribution.

F.1.3 Model Selection

Once the model and the variables have been defined, one should be able to determine whether a variable is significant or not. To do so, several statistical tests are available, such as Pearson chi-square (Dean and Lawless 1989; Bauer and Harwood 2000), t-statistics (Vogt and Bared 1998), and F-statistics (Tegge 2008). Also, to assess the performance of the model, specific criteria were used in the literature: R^2, R^2FT (Fridstrøm et al. 1995; Bauer and Harwood 2000), cumulative residual (Hauer 2004), and the least-squares method (Bauer and Harwood 2000).

Hauer (1996) provided guidelines for statistical testing in roadway safety. The methodology includes the choice of a null hypothesis and an alternative hypothesis, the choice of a level of significance, and a statistical test to define the rejection region. This statistical test will depend on a degree of freedom that is defined by the parameters of the model. For instance, the null hypothesis could test whether a parameter estimate β_i is equal to zero or not. The level of significance value ranges between 0.01 and 0.10.

To assess the goodness-of-fit of the model, the CURE method (cumulative residuals) estimates the difference between the numbers of predicted and recorded crashes with respect to AADT (Hauer and Bamfo 1997; Hauer 2004). If the curve oscillates around zero, it means that the model fits the data. Otherwise, one can consider either modifying the functional model or introducing other explanatory variables. The R^2 statistic is another goodness-of-fit commonly used in the literature. However, Miaou et al. (1996) demonstrated that this indicator does not perform well when the model is not normal linear with additive mean functions and that R^2 might not be close to 1 even if the model is appropriate. Therefore, alternative indicators were developed for accident prediction models. Fridstrøm et al. (1995) proposed different goodness-of-fit measures for Poisson regression models. In particular, the authors studied weighted goodness-of-fit measures R^2FT and R^2PW so that the measure could reach the value of 1 when the model is the best possible.

Finally, the likelihood ratio statistic is a statistical test for comparing two models (Fridstrøm et al. 1995; Tegge 2008). It has a chi-squared distribution, and the degree of freedom is equal to the difference between the degree of freedom of the two models.

F.2 CRASH COST ESTIMATION

To further study the impact of vehicle accidents on roadway safety, the associated cost should be quantified. For that purpose, US DOT and the National Highway Traffic Safety Administration (NHTSA) provide guidelines that will be detailed in this section. With the SPFs determined as in Section F.1, one should be able to link the number of expected crashes for a particular type of accident—fatal injury, A-injury, B-injury, C-injury, and property damage only (PDO)—and the cost inherent in that type of accident. In the year 2000, the total economic cost of vehicle accidents in the United States was estimated to be $230.6 billion (Blincoe et al. 2002), which represents about 2.3% of the U.S. GDP. In 2010, it was estimated to be $277 billion (Blincoe et al. 2014).

The literature typically separates costs into two groups: human capital costs and comprehensive costs. The former represents economic costs associated with motor vehicle crashes and injuries. The latter incorporates the cost associated with the loss of quality of life in addition to human capital costs. Thus, human capital costs place a dollar value on crashes and estimate the economic losses of past crashes, while comprehensive costs estimate the total cost to society of crashes (Hanley 2004).

Human capital costs have two components: direct and indirect costs. Direct costs include those for medical and non-medical services related to the injury and the crash. Direct medical costs encompass services such as ambulance, emergency room, rehabilitation, physical therapy, and prosthetic devices. Direct non-medical costs include, for example, administrative costs, travel delay, property damage, home adaptation, and workplace disruption. Indirect costs are productivity losses such as market wage and benefits, as well as non-market costs such as household work and childcare (Blincoe et al. 2002). Intangible consequences of these events to individuals (pain, suffering, or permanent loss of functional capacities) are not taken into account in the human capital costs unless they result in a permanent productivity loss. Blincoe et al. (2002) also tried to quantify the psychological costs of injuries.

Comprehensive costs encompass the human capital costs previously described and non-monetary losses associated with crashes and injuries, such as pain and suffering. A measure was developed to estimate the dollar cost of the additional component of the comprehensive cost. To estimate the quality-adjusted life years lost, QUALY gives a value from 0 to 1, where 1 represents a year of perfect health and 0 represents death (Blincoe et al. 2002). The value is determined by the duration and severity of the injury and using diagnosis and age group–specific estimates from Miller et al. (2000, as cited in Council et al. 2005). Comprehensive costs can also include the willingness to pay to reduce the risk of loss of life (Hanley 2004). It measures the trade-offs society is willing to make between safety and expense (e.g., smoke detectors). Figure F.1 illustrates the portion of the value for the intangible consequences and the human capital costs for different severities of injury. As severity decreases, the relative value of lost quality of life decreases because injuries are less severe and people with casualties are more likely to recover quickly.

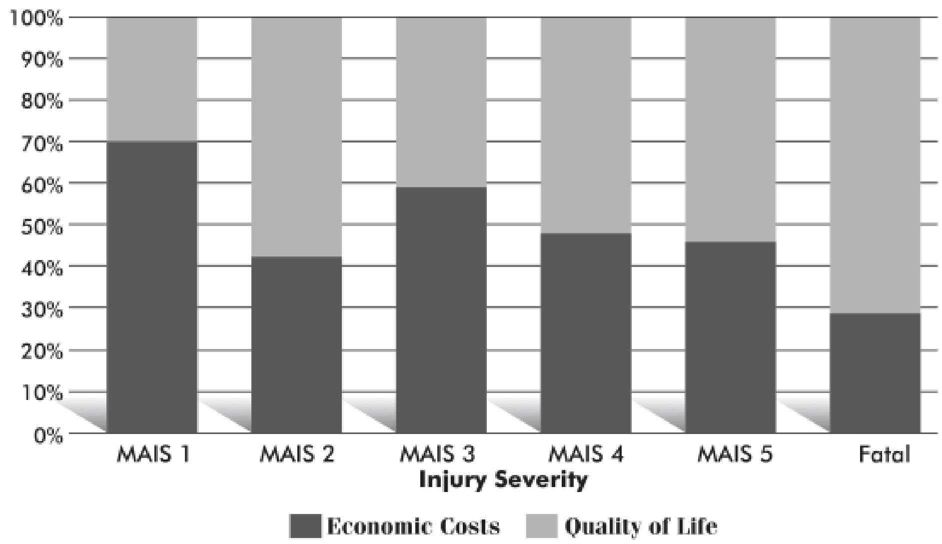

Figure F.1: Distribution of comprehensive costs (Blincoe et al. 2002).

F.2.2 Dollar-Value Estimation

Many researchers have estimated human capital costs and comprehensive costs. Therefore, several values can be found based on injury levels. Both the NHTSA and the National Safety Council (NSC) provide economic costs of motor vehicle crashes. Tables F.1 through F.4 display the 2010 and 2012 cost estimates from NHTSA and NSC, respectively. The 2010 values were adjusted using a GDP deflator (Hanley 2004). Sources include the US DOT, U.S. Environmental Protection Agency (US EPA), U.S. Department of Agriculture (USDA), and others (see Viscusi 1995; Miller 2000; Dionne and Lanoie 2002; and Mrozek and Taylor 2002).

Table F.1: Total Cost per Person for All Injury Levels (NHTSA 2014)

Crash Type	2010 Dollars	2014 Dollars*
Fatal	$1,398,916	$1,490,851
Critical AIS 5	$1,100,597	$1,172,927
Severe AIS 4	$513,949	$547,725

Serious AIS 3	$276,996	$295,000
Moderate AIS 2	$101,961	$108,662
Minor AIS 1	$19,888	$21,195
Property Damage Only	$3,862	$4,116

Table F.2: Total Cost per Person for All Injury Levels (NSC 2012)

Crash Type	2012 Dollars	2014 Dollars*
Fatal	$1,410,000	$1,447,708
Incapacitating Injury (A)	$72,700	$74,644
Non-Incapacitating Injury (B)	$23,400	$24,026
Possible Injury (C)	$13,200	$13,553
Property Damage Only	$8,900	$9,138

Table F.3: Comprehensive Costs of Motor Vehicle Crashes (NHTSA 2014)

Crash Type	2010 Dollars	2014 Dollars*
Fatal	$9,145,998	$9,747,059
Critical AIS 5	$5,679,122	$6,052,345
Severe AIS 4	$2,551,432	$2,719,108
Serious AIS 3	$1,082,693	$1,153,846
Moderate AIS 2	$442,833	$471,935
Minor AIS 1	$43,129	$45,963
Property Damage Only	$3,862	4,116

Table F.4: Comprehensive Costs of Motor Vehicle Crashes (NSC 2012)

Crash type	2012 dollars	2014 dollars
Fatal	$4,538,000	$4,659,362
Incapacitating injury (A)	$230,000	$236,151
Non-Incapacitating Injury (B)	$58,700	$60,270
Possible Injury (C)	$28,000	$28,749
Property Damage Only	$2,500	$2,567

Cost estimates found in the previously mentioned studies are valid only for the year from which they were derived. If human capital costs are required for another year, then it is recommended to adjust the values provided in the tables using a Consumer Price Index (CPI) or gross domestic product (GDP) deflator. If comprehensive costs are required, then a two-step procedure should be performed. Human capital costs should be adjusted as described previously.

F.2.3 State Practices

Highway agencies are responsible for protecting the billions of taxpayer dollars invested in highway infrastructure. As such, states have continued to increase their involvement in transportation system safety by enforcing laws and policies related to motor vehicle safety. State agencies are often interested in the economic and social impact of vehicle accidents so that they can make decisions regarding new or existing roadway safety laws.

Blincoe et al. (2014) analyzed the economic cost attributable to motor vehicle crashes by state in 2010. Results showed that Illinois accounts for the second largest portion of the total cost nationwide. One difficulty in that study came from the fact that states use different reporting practices. The authors estimated human costs for each state based on the guidelines provided by NHTSA and a single reported number of injuries and fatalities in each state. Thus, they were able to overcome the difficulty associated with the diversity of practices among states.

However, Hanley (2004) conducted a study to report the practices used by state DOTs to estimate motor vehicle crash costs. State DOTs make use of those values in various ways. Most states use the values when reporting crash costs, but some of them use the values to identify high-frequency crash locations. Still others use the values to assign funding for safety improvement investment or to estimate the potential benefit of a safety improvement, or, to a lesser extent, for reviewing design exceptions. Results also showed that 22 states use dollar values for fatalities and injury crash costs. Some of them use human capital costs (Minnesota, Indiana, Wisconsin), and many states use the values of comprehensive costs provided by NSC and NHTSA. Hanley recommended using the source whose injury-level reporting matches the one used by the state DOTs.

F.3 SAFETY IMPROVEMENT ANALYSIS

SPFs, as described in a previous section, estimate the expected crash frequency on a roadway segment or an intersection. SPFs can help in assessing the actual safety of a given site and possible benefits from roadway treatments. The evaluation of safety improvement measures is essential in safety analysis. When information is being sought on accident modification factors for the development of countermeasures for road safety, an observational before-and-after study typically is conducted. Therefore, methodologies were developed and can be split into two groups: conventional and empirical Bayes approaches (Hauer 1997). The natural domain of observational studies corresponds to the situation when the roadway segments or intersections that are affected by improvement measures maintain most of their other original attributes. Hauer's work (1997) is meant to provide guidelines for properly conducting and interpreting observational before-and-after studies. The safety of a site during a particular period of time is usually defined by the number of accidents that occur at the site per unit of time. Therefore, observational studies focus on the change in accident frequency at a given site, for various accident severities.

F.3.1 Conventional Before-and-After Evaluation

Conventional before-and-after studies seek to estimate the difference between a prediction of the expected number of crashes without any treatment and the number of crashes with treatment, based on the expected number of crashes before the treatment, along with the variance of those measures. Hauer (1997) analyzed naïve before-and-after studies, improving methods such as accounting for factors or using comparison groups; and the empirical Bayes method, which will be discussed in further detail in the next section.

The naïve before-after study approach uses the “before” accident count of the site to predict the “after” accident count if no treatment had been implemented. Although this approach is frequently used in the professional literature and can achieve an interesting statistical precision, it has important drawbacks. It actually estimates a mix of the effects related to the treatment—variables independent from treatment that may change from “before” to “after”—and is based on possible random peculiarities of accident count before the treatment. To better assess the influence of the roadway treatment, improvements are suggested in exchange for a loss in statistical precision.

To account for the safety effect of changes in other parameters, those parameters can be either explicitly included in the modeling or implicitly divided into comparison groups. The former method derives the estimated number of accidents without any treatment by multiplying the one found with the naïve method by parameter ratios (e.g., to account for changes in the traffic flow). The latter method deals with unrecognized parameters, unmeasured parameters, or those whose influence on safety is obscure. To do so, a comparison group is used. A group of entities is selected (e.g., road segment, intersection) that has not been treated and that has properties similar to the treated entities. Thus, treated entities form the treatment group, and untreated entities form the comparison group. For the outcome, it is hoped that, without treatment, the treated group would have had an “after” accident count proportional to that of the comparison group. This method is known as the C-G method.

Yuan et al. (1999) conducted a before-and-after study in Connecticut to update crash reduction factors for highway treatment. Two different methods of before-and-after analysis were implemented—the first one based on confidence intervals, and the second one based on likelihood. To estimate crash frequency, treatment groups and comparison groups were defined in spite of the complexity of finding locations with similar conditions. As a result, one limitation of the study was the small size of the comparison groups.

Regression-to-mean bias is a common phenomenon that arises in naïve before-and-after studies. Hauer (1997) explains the selection bias and regression-to-mean bias as follows:

> If an entity is treated because its “before” accident count K was abnormally high or unusually low, then the same K cannot possibly be a good estimate of κ [the expected value of K or E{K}]. After all, κ is the expected value of K; that is, the very embodiment of a usual, normal, or average accident count. One cannot estimate what is normal and usual by using accident counts that are abnormal or unusual. To do so would lead to an obvious bias. If the entity has been selected because it had an unusually high number of accidents, K would tend to overestimate κ (p. 178).

Thus, regression to the mean may occur if an entity with an abnormally high or unusually low crash count is selected for treatment. Even if an entity is not selected because of its abnormal level of accident count, K is not a good estimate of κ, and regression to the mean still might occur. The empirical Bayesian analysis presented in the next section is a solution to avoid this bias.

F.3.2 Empirical Bayesian Analysis

Both naïve and comparison group methods assume that the expected “before” accident frequency can be estimated from the observed accident frequency before the treatment. However, this assumption is true only if the crash record of the location has nothing to do with the reasons it was treated (Hauer 1997). Therefore, any naïve observational studies that base their methodology on the expected "before" accident count will be subject to the regression-to-mean phenomenon. Also, there is no exact time for when the "before" period starts. Therefore, the expected "before" accident frequency could be the accident frequency observed 1 year before the beginning of the treatment, or it could be the average for the past 3 years. Neither are unreasonable assumptions. Because the expected number of accidents in a group of entities under a given period of time may change from year to year (owing to changing traffic, weather, vehicles, etc.), each number has to be estimated separately.

The empirical Bayesian (EB) approach is a way to address these problems. The EB method accounts for both the accident record of a specified location and the known information regarding locations with the same characteristics. Thus, a weighted sum of those terms appears as a natural way to derive the expected "before" accident count. Hauer's equation (1997) consists of the following:

$$\mathrm{E}(\kappa|\mathrm{K}) = \alpha\mathrm{E}(\kappa) + (1-\alpha)\mathrm{E}(\mathrm{K}) \qquad \text{(F.6)}$$

where the weight α is given by in Equation F.7 to maximize precision.

$$\alpha = \frac{1}{1 + \frac{\mathrm{VAR}\{\kappa\}}{\mathrm{E}(\kappa)}} \qquad \text{(F.7)}$$

where

- K = accident count during the "before" period on a treatment group of entities
- κ = expected number of accidents to occur on this group of entities

$\mathrm{E}(\kappa)$ and VAR$\{\kappa\}$ can be estimated using the multivariate models described in Section F.1.

There is a large amount of literature about safety analysis using an EB approach (Hauer 2001; Persaud et al. 2001; Yuan et al. 2001; Hauer et al. 2002; Tegge 2008).

Hauer (2001) investigated the influence of the overdispersion parameter k from the negative binomial regression on the relative weights of the EB approach. When k is assumed to be the same across all entities, it results in a bias of the segment length influence and inconsistency when estimating the safety of a road segment. One solution is to replace the overdispersion parameter by a linear function of the overdispersion parameter and the segment length.

Persaud et al. (2001) evaluated the safety effect of roundabout installation to replace traffic signal controls. A before-and-after study was conducted using an EB approach to account for the effect of regression to the mean and the changes in traffic.

Yuan et al. (2001) conducted a before-and-after analysis using an EB approach to estimate crash reduction factors for two-lane rural highways, in addition to the comparison group method. In particular, the authors investigated the effects of intersection approach realignment. However, an analysis of variance proved that the small sample size did not allow accurate estimation of the population mean for all sites treated with realignment.

Hauer et al. (2002) provided guidelines for practical application of the EB method through a handful of examples. It provides another formulation for the EB approach in the case of a negative binomial distribution:

$$\alpha = \frac{1}{1 + \frac{\mu \cdot Y}{\phi}} \tag{F.8}$$

where

μ = accident count for similar entities per year per km

Y = number of years of accident counts used

ϕ = overdispersion parameter estimated per unit length

This formulation is derived from Equation F.7 in the particular case of a negative binomial regression.

Tegge (2008) also investigated the site screening procedure, in which he uses the EB approach to correct for the regression-to-mean bias and displays a formulation similar to Equation F.8. However, the weight is derived from a sequence of expected number of crashes for several years and is applied to Equation F.10:

$$w = \frac{1}{1 + k\sum_{n=1}^{N} P_n} \tag{F.9}$$

where

$1/k$ = overdispersion denoted by ϕ in Equation F.8

Pn = predicted number of crashes for a given roadway in year n

The expected number of crashes (m) can be derived based on the weight w given by Equation F.9, the predicted number of crashes (P) computed with the SPFs previously discussed, and the total number of crashes observed within n years.

$$m = w \cdot P + (1 - w) \cdot \frac{F}{n} \tag{F.10}$$

Note that from Equations F.8 and F.9, the longer the observed accident count is recorded, the smaller the weight factor is—and the closer the expected accident count is to the observed accident count.

F.4 TRUCK-RELATED SAFETY ANALYSIS

F.4.1 Data Collection

One major limitation of the research on size and weight of heavy trucks is the poor quality of crash and exposure data. Most crash data collections are inadequate to study heavier and longer trucks. For instance, Fancher and Campbell (1995) addressed the relationship between truck size and weight and safety. However, there were not enough data to examine the effect of weight in the different configurations. Many sources for crash data related to heavy trucks are available, but they are usually incomplete. Thus, sources such as the Fatal Accident Reporting System (FARS) from NHTSA, the Office of Motor Carrier Safety (OMCS) file, and the National Accident Sampling System (NASS) are not reliable. Even though it contains all accidents, FARS does not contain a detailed description of the trucks—and therefore does not allow a statistical study of their configuration. OMCS has all vehicle information but does not provide a full accident description. NASS has too few large trucks to serve as a basis for astatistical analysis. Therefore, some studies collect data from police-reported accidents (Campbell et al. 1988; Fancher and Campbell 1995; Blower and Matteson 2010). The first two studies used crash data in the period from 1980 through 1984, which are now outdated. The third study collected data from the Kansas Police Accident Report File, which represent about 1,500 crashes.

One of the main data sources for truck safety research is the University of Michigan Transportation Research Institute (UMTRI) (Blower et al. 1993; Forkenbrock and Hanley 2003). Forkenbrock and Hanley collected data from the Trucks Involved in Fatal Accidents (TIFA) file held by the UMTRI for the years 1995 through 1998. Although TIFA data included truck configuration, they focused only on fatalities and did not include mileage. Alternative sources found in the literature include the Commercial Vehicle Enforcement Section of the Washington State Patrol (Stein and Jones 1988) and the Large Truck Crash Causation Study Data from the U.S. Federal Motor Carrier Safety Administration and the NHTSA (Lemp et al. 2011).

Thus, numerous sources are available, but information about truck weight, length, or configuration is often inaccurate or incomplete. No state crash data system includes the operating weight of trucks at the time of the crash, and most of them do not include length of either individual units or combination lengths. Also, most states do not distinguish heavier weight or longer lengths. In addition, data collection about truck weight might be biased. Truck weight data are usually collected at weigh stations. Therefore, the available weight data are likely to be biased toward the legal weight carriers because overweight trucks are more likely to operate on alternative routes to avoid weigh stations (FHWA 2013).

Scopatz (2001) addressed data-quality issues for long combination vehicle (LCV) safety. It was shown that, in general, data do not display the exact configuration of trucks involved in crashes nor exact measures for LCV exposure. Evaluation of the data collection was performed for five states. Results showed that none of those states followed best practice methods well enough to serve as a model for others. For instance, none of the states collect data on all crashes, almost none have an adequate list of vehicle types, and none estimate VMT on a regular basis.

Because of the data issues, two approaches were developed as detailed by FHWA (2013). The first approach identifies critical performance characteristics of heavier and longer trucks. Relationships between those characteristics and truck crashes were previously established. The safety of the truck for a given configuration is then extrapolated from the studied fleet. The second approach simply relies on observational studies. These approaches will be discussed in the next section.

F.4.2 Statistical Studies

Regarding truck safety analysis, there is extensive available literature focusing on truck performance characteristics and configuration.

Different methods have been used to assess the influence of truck configuration, length, and weight. As mentioned in Section F.1, the adjusted rate method was used in the oldest truck safety literature (Campbell et al. 1988; Stein and Jones 1988; Fancher and Campbell 1995). Campbell et al. (1988) sought to determine the effect of different truck dimensions, configurations, travel category, and gross combination weight. Results showed that the relationship between factors is complex. The first finding was that differences in fatal accident occurrences are significant with respect to the type of road and the time. One of the conclusions of the study was that the fatal accident rate for doubles is about 10% lower than that of singles, primarily because doubles use interstate-quality roads for a disproportionate share of their travel. Fancher and Campbell (1995) identified critical performance characteristics, such as rollover threshold, braking efficiency, and rearward amplification. Those characteristics directly affect roadway safety because they directly impact the ability of the trucks to maneuver. Also using involvement ratios, Stein and Jones (1988) found that performance characteristics such as braking efficiency, as well as steering and wheel condition, has the largest influence on truck safety when compared with other parameters (driver age, weight, road type).

Forkenbrock and Hanley (2003) focused on crash involvement and conditions for LCVs. They grouped the variables of interest into three categories: road attributes, prevailing conditions, and other circumstances. Automatic Interaction Detector (AID) and Multiple Classification Analysis (MCA) methods were jointly used to describe the relationship between truck crash occurrence, truck configuration, and variables grouped into those three categories. The AID method partitions the variables into groups on the basis of attributes and conditions associated with crashes, while the MCA method quantifies the influence of those variables on truck crashes.

Blower et al. (1993) also focused on truck configuration, road type, area type, and time-of-day influence on truck accidents. Using a multivariate model and Poisson regression model, the authors showed that, even if doubles tend to be safer than singles on all road types, this difference is not statistically significant. However, it was shown that road type has a large impact on road safety (major roads have a high risk of accidents, while limited-access roads have a low risk), subject to the influence of the operating environment (urban areas lower the risk of accidents, but night driving raises it).

Regarding observational studies, two significant studies reviewed the safety performance of longer and heavier trucks in North America: Woodrooffe (2001) in Canada, and Abdel-Rahim et al. in some western U.S. states (as cited in FHWA 2013).

Woodrooffe’s study (2001) determined road safety performance of LCVs, along with the factors contributing to accidents involving LCVs in Alberta, Canada. Collision exposure was defined as the ratio of the total number of vehicles of a given type involved in collisions and the total distance traveled by that vehicle type. VMT and traffic counts were determined from a survey. Crash rates were estimated from truck involvement in crashes. The study demonstrated that LCVs have the lowest collision rate: one fourth of that for tractor/semi-trailers and about one fifth of that for multi-trailers. It also demonstrated that adverse conditions were present in 42% of LCV crashes, although truck traffic is restricted under those conditions.

Abdel-Rahim et al. performed a similar study in some western U.S. states (as cited in FHWA 2013). The objective of the study was to estimate crash rates for several LCV configurations (e.g., Rocky Mountain doubles, triples, straight trucks with two trailers) and compare those with singles. LCV exposure was estimated using WIM station data and vehicle classification data. In spite of data limitations, singles and doubles were found to have the same severity distribution: about 3% involved a fatality, about one third involved an injury, and the rest involved PDO. Triples were also found to have the highest PDO crash proportion.

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APPENDIX G: COST DATA FOR BRIDGE LIFE-CYCLE COST ANALYSIS

The data used to calculate average new bridge construction cost and major maintenance (e.g., deck replacement) cost are given in Table G.1 and G.2. Before calculating averages, Tuker's outlier test was applied on the data. Upper and lower limits show minimum and maximum values for identifying the outliers on both tables. Tables also show the averages cost with and without outliers.

The data were extracted from 2015 construction cost report provided by IDOT.

Table G.1: New Bridge Construction Cost per Ft^2

New Bridge Construction Cost ($/ft^2)					
343.91	122.47	184.18	243.34	Min	78.08
197.55	118.59	163.82	186.66	1st	128.64
162.08	211.77	408.95	186.66	2nd	163.06
206.77	211.77	216.94	187.06	3rd	202.48
238.78	109.20	216.94	190.59	Max	560.09
190.69	124.85	339.64	181.04	IQR	73.84
388.76	124.56	160.98	201.81	Lower Limit	17.88
179.21	168.93	129.12	176.18	Upper Limit	313.24
163.06	124.71	102.94	128.93	Mean with Outliers	182.44
167.25	141.17	129.47	83.19	Mean w/o Outliers	162.40
167.25	122.08	148.25	191.36		
195.84	78.08	149.13	133.00		
124.02	114.37	201.00	128.21		
145.33	113.40	157.71	114.53		
248.50	112.09	157.71	134.08		
196.83	93.86	111.55	128.73		
229.48	112.09	203.14	263.73		
229.48	162.58	230.20	148.25		
185.15	129.52	139.72	274.20		
146.30	134.92	135.75	191.31		
146.30	125.36	128.54	560.09		
174.27	118.38	213.71	560.09		
313.24					

Table G.2: New Bridge Construction Cost per Ft^2

Major Maintenance Cost ($\$/ft^2$)			
14.15	74.97	Min	14.15
101.17	68.49	1st	73.01
155.17	78.22	2nd	110.38
246.14	119.59	3rd	155.42
156.15	86.73	Max	246.14
43.87	96.28	IQR	82.40
173.86	153.92	Lower Limit	-50.59
59.48	152.96	Upper Limit	279.02
31.38	179.71	Mean with Outliers	112.47
128.13	157.56	Mean w/o Outliers	112.47
74.52	121.81		

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